Research Article

Analysis of Lateral Displacement Law of Deep Foundation Pit Support in Soft Soil Based on Improved MSD Method

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Foundation pit envelope and foundation pit excavation solution design is a multidisciplinary problem that could be linked to a series of safety issues in the geotechnical engineering of an actual construction project. Moreover, the construction of large deep foundation pit in soft soil often faces greater risks and challenges as the support structure deforms more easily and unpredictably. In order to improve the deformation prediction of deep foundation pit engineering precision and efficiency and to ensure that the construction of deep foundation pit engineering is safe and efficient, in this article the traditional MSD (mobilizable strength design) theory research and analysis, combined with the Jinan formation characteristics of a tunnel in Jinan, and new parameters were introduced to the original MSD method theory: the wall itself within the bending strain energy $U$ and support compression elastic potential energy of $V$. A new law of conservation of energy is constructed, and finally, an optimized MSD method has been proposed, this method is shown in the article. Finally, the results of foundation pit deformation calculation were compared among the optimized MSD method, finite element calculation method, and field monitoring data analysis method, so as to demonstrate the reliability of the prediction system of optimized MSD method and finite element analysis method. The results show that, by optimizing the MSD calculation method, the horizontal displacement of the retaining wall varies depending on the excavation depth of the foundation pit and the form of internal support with the overall peak value of displacement between 0–0.2% $H$ ($H$ being the excavation depth); the deformation of retaining wall increases gradually with the increase of excavation depth of the foundation pit, and the peak position of deformation gradually moves down with the excavation of foundation pit. The trend of these changes is consistent with the results of the finite element method and field data analysis method, proving that the optimized MSD method is reliable in predicting the deformation of the foundation pit under specific stratum conditions.

1. Introduction

Constructing a large deep foundation pit in soft soil strata is risky [1–3], and the retaining structure of the foundation pit is prone to deformation [4,5]. Therefore, it is very important to have a deep understanding of the deformation law of the foundation pit in soft soil strata. However, existing methods of foundation pit deformation prediction often produce large errors. For example, the empirical formula method produces errors due to soil layer differences. In addition to errors, functions of BP artificial intelligence methods such as artificial neural network [6] and support vector machine are also limited by the monitoring data and the learning process.

MSD (mobilizable strength design) is a common method for foundation pit deformation prediction proposed by Osman and Bolton [7] based on the plastic deformation mechanism of cantilever foundation pit excavation and the principle of energy conservation in the foundation pit system. The equation for ground subsidence of retaining wall with internal support was derived, and the practicability of the MSD method was verified.
O’Rourke and Clough [8] studied the deformation form of internal support and proposed that the MSD theory is mainly based on three deformation failure modes: the first is cantilever displacement mainly occurring before the internal support or anchor structure is applied; the second is the concave displacement of the retaining structure; and the third is the maximum lateral displacement near the excavation depth. Since then, Bolton et al. [9,10] calculated the deformation of the foundation pit with internal support using the MSD method, which combines two basic theories of deformation increment method and energy conservation principle and includes the anisotropy and undrained shear strength. Bolton [11] applied the MSD method to the cantilever deformation and convex deformation problem in foundation pit engineering. Based on the increment method and energy conservation principle, the dynamic development of nonlinear, anisotropic, and undrained shear strength of the site is considered. According to the plane strain test data, the importance of the safety factor in evaluating the stability of the foundation pit is demonstrated according to the theory of shear strength and ultimate balance. Since then, Bolton [12] applied the MSD method to cantilever deformation and proposed that the strength is closely related to the strain level. Based on the strength exertion theory, the method is used to calculate the deformation of the foundation pit and surrounding strata. Wang et al. [13] studied the work done when bending strain occurs in the retaining wall structure and supplemented the energy conservation system of the MSD theory. Liu et al. [14] further improved the deformation mechanism of the foundation pit and supplemented the theory of MSD foundation pit deformation based on empirical data. By summarizing the measured data of Hangzhou typical long and narrow soft soil foundation pit, Ma et al. [15] concluded the lateral movement law of typical section support of foundation with the MSD method of modified strength analysis and prediction theory. The measured data were analyzed, and the results proved the effectiveness of the MSD method.

MSD method is an effective and practical method to predict the deformation of the foundation pit, yet the energy conservation system of the method still needs perfection, and the formula derived from this method so far is only applicable to the silt layer. For clay foundation pits, the deformation mechanism and calculation parameters are quite different. Therefore, this powder-clay foundation pit is not completely applicable to the conditions in Jinan. In this case, this paper took the actual stratigraphic conditions in Jinan into consideration and introduced new deformation parameters to the traditional MSD theory, namely, the bending strain energy U of the wall itself and the compressive elastic potential energy V of the inner support. A new law of conservation of energy was constructed: ΔP = ΔW + ΔU + ΔV, and an optimized MSD method was proposed. At the same time, finite element analysis was used as an auxiliary approach to further improve the deformation prediction system of soft soil foundation pit [16].

2. Engineering Profile and Geological Conditions

2.1. General Engineering Profile. This paper studied the foundation pit engineering of the working well on the north bank of a tunnel in Jinan (Figure 1). The tunnel in Jinan is a superlarge section shield tunnel located in the northern part of Jinan City. To fit the shield machine, the foundation pit of the north bank starting well was designed to be 34.14 to 50 m wide and 152.2 m long with an excavation depth of 31.2 m.

2.1.1. Exterior-Protected Construction. The retaining structure of the foundation pit was protected by an underground continuous wall with a thickness of 1.2 m and a depth of 47–51.5 m. Underwater reinforced concrete was used to cast the wall, and joints of the wall were connected with H section steel.

2.1.2. Internal Support. The foundation pit was excavated in five steps, in which seven steel bracing concrete supports were installed at the starting well of the large shield and five steel bracing concrete supports were used in the starting section. The design and construction steps of the five supports AA monitoring section are shown in Figure 2 and Table 1.

2.2. Engineering Geological Conditions. The foundation pit project is located in the alluvial plain, and the local microgeomorphology unit is the Yellow River bed. The original terrain is flat, and the terrain is slightly undulating. The drilling shows that the surface layer is partly artificial fill, followed by Quaternary Holocene alluvial, alluvial silty clay, silt, sand layer, and late Mesozoic Yanshanian intrusive gabbro. Therefore, the soil quality is poor. The foundation pit is mainly composed of silty clay and silt.

2.3. Hydrogeological Conditions. The geomorphologic unit of the foundation pit project is the grade I terrace of the Yellow River. The surface water is mainly from the Yellow River, the Queshan Reservoir, and the various fish ponds. At a depth of 1.10–1.70 m, the groundwater is the pore phreatic water of the Quaternary loose overburden. The aquifer is mainly artificial fill, silty clay, silt, and sand layer. Silty clay belongs to the micropermeable layer, silt belongs to the micro-weak permeable layer, and the sand layer belongs to the medium-strong permeable layer.

3. Modified MSD Method for Deformation Prediction

3.1. Deformation Mechanism of Foundation Pits

3.1.1. Deformation Mechanism of Soil around the Pit. Geotechnical engineering scholars Lam and Bolton [17] published the study, in which the deformation of soil around the foundation pit is divided into four regions: rectangular ABCD, sector DCE, sector FHE, and triangular HIF. The deformation mechanism of soil at each depth of the foundation pit is shown in Figure 3. During excavation and internal bracing, the retaining structure produces displacement deformation, accompanied by the settlement of the ground around the pit. In the formation, the soil in the
The yellow river north toll station
North shore working well
The Queshan west village
North shore levee
The south bank of dike
The yellow river
North ring viaduct
Second ring road
Jiluo road
South bank work well

Figure 1: General situation of the starting shaft of a tunnel in Jinan.

Figure 2: Design drawing of AA section.

| Conditions | Construction |
|------------|--------------|
| Condition 1 | Dig to 2.1 m, set the crown beam and the first support (concrete support) at 2.1 m, and pour the retaining wall above |
| Condition 2 | Dig to 7.7 m and set the second support (steel support) at 7.2 m |
| Condition 3 | Dig to 13.5 m and set the third support (concrete support) at 13.5 m |
| Condition 4 | Dig to 20 m and set the fourth support (concrete support) at 20 m |
| Condition 5 | Dig to 24 m and set the fifth support (steel support) at 23.5 m |
| Condition 6 | Dig to 31.2 m |
four regions is deformed following the shape of a cosine function. According to the law of deformation transfer, the horizontal displacement of the retaining wall is equal to the land subsidence caused by the wall at that same position. In Figure 3, the size of the deformation on each arrow is always stable and passed to the rear position.

For rectangular ABCD, the horizontal and vertical deformation calculation formulas are as follows:

\[
\Delta w_x = \frac{\Delta w_{max}}{2} \left[ 1 - \cos\left(\frac{2\pi x}{l}\right) \right],
\]

\[
\Delta w_y = \frac{\Delta w_{max}}{2} \left[ 1 - \cos\left(\frac{2\pi y}{l}\right) \right],
\]

where \(\Delta w_{max}\) is the peak value of the deformation of the retaining wall structure in the engineering, \(\Delta w_x\) is the deformation of the retaining wall in the horizontal direction, \(\Delta w_y\) is the deformation in the vertical direction, and \(l\) is the width of the deformation affected area.

3.1.2. Deformation Mechanism of Foundation Pit Enclosure Structure. According to geological conditions, supporting devices are generally set up at every 3 to 5 m with the progress of excavation. O’Rourke [12] pointed out that, in the single excavation of rock and soil under an inner brace, the retaining wall would produce a horizontal deformation similar to the shape of a cosine function \(\Delta w\), as shown in Figure 4, and the calculation method of \(\Delta w\) is shown in the following equation:

\[
\Delta w = \frac{\Delta w_{max}}{2} \left[ 1 - \cos\left(\frac{2\pi y}{l}\right) \right],
\]

where \(l = \alpha s\),

\[
\Delta w\] is the horizontal displacement change value of the retaining wall structure at a certain position in the area below the current internal brace, \(\Delta w_{max}\) is the maximum horizontal displacement, \(l\) is the width range of the deformation affected area (m), \(s\) is the distance between the current internal brace and the bottom end of the retaining wall, and \(\alpha\) is the coefficient of deformation range. \(\alpha\) is 1 in hard formations or 2 in soft clay. The underground wall in on-site construction is basically in the formation between 1 and 2. At the depth direction, the peak deformation of the underground continuous wall is generally close to the excavation surface of the project [18] and can be considered \(1 < \alpha < 2\) [19].

3.2. Energy Conservation of Foundation Pit in the Improved MSD Theory. The improved MSD theory shows that the shear strength of the soil is functionally related to the shear strain. During the engineering construction, the stress state of the soil is disturbed, resulting in the soil pressure field being directly applied to the retaining wall. Under the action of the force, the retaining wall structure deforms and displaces. Simultaneously, the soil around the pit also tends to have relative motion, which will bring about internal forces, namely, shear stress. When the displacement occurs, the internal force does work, and the conservation of energy occurs in the foundation pit system. In the construction, the whole project always complies with the law of conservation of energy, and the work done by the external force (gravity) is always equal to the work done by the internal force. On the one hand, the work done by the internal force is manifested as shear stress. On the other hand, the work done by internal force during compressive deformation of internal support and bending deformation of parapet structure are also included. The former is stored in internal support as compressed elastic potential energy, while the latter is stored as bending deformation energy in parapet structure.

Improvement to the MSD theory mainly lies in the fact that new parameters were introduced to the original MSD method: the bending strain energy \(U\) of the wall itself and the compressive elastic potential energy \(V\) of the inner support. A new law of conservation of energy is constructed:
\[ \Delta P = \Delta W + \Delta U + \Delta V, \] fully considering the different physical and mechanical properties of soil in the construction environment, the interaction between the supporting structure and soil, and the dynamic development of undrained shear strength of soil.

### 3.2.1. Gravity Work of Soil

During foundation pit excavation, the variation of the total gravitational potential energy at the end of the foundation pit excavation is \( W \), and the variation at the stage of excavation \( m \) is \( W_m \). The working principle diagram is shown in Figure 5.

The formula for calculating the work done by the soil gravity is as follows:

\[
W_m = \sum_{i=1}^{I} \left[ \int_{\Omega} \gamma_i(m, i) v(m, i) d\Omega \right],
\]

\[
W = \sum_{M=1}^{M} W_m,
\]

where \( \Omega \) is the influence range of the deformation region, \( i \) is the soil layer on the construction site, and the total number of layers is \( I \); \( m \) refers to the number of layers designed for foundation pit construction, and the total excavation has \( M \) layers. \( \gamma_i(m, i) \) is the average weight of layer \( i \) at excavation stage \( m \); \( v(m, i) \) is the change of vertical displacement of the rock and soil mass in layer \( i \) during the first excavation. By dividing the excavated soil into multiple layers, the work done by external forces accumulated in each stage can be more accurately calculated, and the variation of the total gravitational potential energy of the soil within the influence range can be obtained.

### 3.2.2. Work Done by Shear Stress

During foundation pit excavation, when excavation goes down to a certain depth, the deformation of the foundation pit at this stage is as follows: the entire foundation pit system, including the soil around the pit, takes the bottom of the retaining wall as the center and makes the triangular rotation, among which the retaining wall makes rigid rotation without deformation, as shown in Figure 6:

\[
\gamma_{mob} = 2\Delta \theta.
\]

\[
\tan \theta = \frac{\Delta w_{max}}{l},
\]

\[
\Delta w_{max} = l\theta = \frac{l \gamma_{mob}}{2}.
\]

During foundation pit construction, when the rock and soil body have plastic deformation without relative slip, the shear strength in the soil body does not reach the due shear strength \( c_u \) and is defined as the apparent value of the undrained shear strength \( c_{mob} \). The apparent shear strength coefficient \( \beta(m, i) \) is defined as the ratio between \( c_{mob} \) and \( c_u \) as shown in the following equation:

\[
\beta = \frac{c_{mob}}{c_u}.
\]

The construction is carried out under the condition that the soil around the pit is not drained and the shear stress work (the internal force work of the foundation pit system) of the soil is done. Its schematic diagram is shown in Figure 7.

The calculation formula of the shear stress work is as follows:
where $c_{mob}(m,i)$ is the apparent shear strength; $\Delta y$ is the shear strain of soil; $\beta(m,i)$ is the apparent shear strength coefficient of soil; $c_u(m,i)$ is the value of undrained shear strength; and $\Delta y(m,i)$ refers to the increment of shear strain under the condition of consolidated and undrained soil in layer $i$ at excavation stage $m$.

### 3.2.3. Bending Strain Energy of Retaining Wall

Wang Haoran et al. [13] have introduced deformation energy $P$ stored in bending deformation of retaining wall to the study of the original MSD method and published Wang Haoran’s improved MSD theory considering the anisotropy of soil. $P$ can be obtained by integrating the flexural stiffness of the retaining wall $EI$ and the incremental value of horizontal displacement and deformation of the retaining wall $\Delta w$:

$$P = E I \int_0^w \left[ \frac{d^2 w^2}{dy^2} \right]^2 dy. \quad (11)$$

Substituting equation (1) into equation (11), the following formula is obtained:

$$P = \frac{\pi^5 EI (\Delta w_{max})^2}{l^4} + \frac{\pi^4 EI (\Delta w_{max})^2}{4} \sin (4\pi s/l), \quad (12)$$

where $EI$ is the flexural rigidity of the engineering retaining wall and $s$ is the distance between the inner support at the lowest applied position in the pit and the bottom of the parapet wall.

### 3.2.4. Compressive Elastic Potential Energy of Internal Support

Liu et al. [14] put forward compressional elastic potential energy $V$ in the foundation pit support, and the calculation formula of $V$ is as follows:

$$V = \sum_{i=1}^l \frac{E_p A_p}{2l_p} (\Delta \gamma_p)^2 \sin \omega, \quad (13)$$

where $E_p A_p$ is the compressive stiffness of the $p$ channel support; $l_p$ is the length of the internal support; and $\omega$ is the angle between the internal support and the side wall of the foundation pit. For the support perpendicular to the inner wall of the foundation pit, $\sin \omega = 1$; $\Delta \gamma_p$ is the deformation of the road support at this position, which is equal to the deformation of the retaining wall structure of the foundation pit in the same position.

During foundation pit construction and excavation, the energy conservation relationship is as follows: the work done by the external force (gravity) of soil is equal to the sum of the work done by the internal force (shear stress) of soil, the bending deformation energy of retaining wall, and the compressive elastic potential energy of internal support. That is,

$$W = U + P + V. \quad (14)$$

### 3.3. Calculation of Improved MSD Deformation

The retaining wall structure of the working shaft foundation pit on the North Bank of a tunnel in Jinan is mainly embedded in the soft silty clay layer, and the over consolidation ratio OCR $=1$; Li [2] has carried out more than 100 groups of soil tests with 19 kinds of silty clay and analyzed the statistical relationship between $\gamma_{mob}$ and $\beta$, which satisfies the following equation:

$$\beta = 0.75346e^{-100\gamma_{mob}/0.035} + 0.821. \quad (15)$$

The influence coefficient of the soft soil layer deformation area where the foundation pit is located $a = 1.5$, that is, the length of excavation deformation influence area $l = 1.5s$. After the first support (concrete support) is applied at the position of $2.1m$, the excavation reaches the depth of $7.7m$. At this time, the soil is silty clay with $\gamma_i = 19.5 kN/m^3$, which is heavy. The length of the support from the bottom of the retaining wall $s = 37 m$, that is, the influence length of excavation deformation zone $l = 1.5s = 55.5 m$. The work done by gravity and by the shear stress of the soil in each layer can be calculated using equations (7), (8), and (10), respectively. The sum of work done by gravity and shear stress is as follows:

$$W = 2280\Delta w_{max},$$
$$U = 4357\beta\Delta w_{max}. \quad (16)$$

The flexural stiffness of the shield shaft retaining wall structure $EI = 1037.7 kN/m^2$, the compressive stiffness of the first concrete support $E_p A_p = 1716 MN/m$, and the effective length of the support is $l_p = 35 m$. According to equations (12) and (13), the flexural strain energy of the retaining wall and the compressive elastic potential energy of the internal support at this time are calculated as follows:

$$P = 6.3\Delta w_{max}^2, \quad (17)$$
$$V = 4.7\Delta w_{max}^2.$$
According to equations (14), (15), and (18), the maximum horizontal displacement of the wall during the excavation of the second layer \( \Delta w_{\text{max}} = 4.3 \text{ mm} \).

Similarly, the maximum horizontal displacement of the wall \( \Delta w_{\text{max}} = 6.3 \text{ mm} \) when the third layer is excavated; \( \Delta w_{\text{max}} = 10.02 \text{ mm} \) when the fourth layer is excavated; \( \Delta w_{\text{max}} = 7.1 \text{ mm} \) when the fifth layer is excavated; and \( \Delta w_{\text{max}} = 11.2 \text{ mm} \) when the sixth layer is excavated.

The maximum displacement of each stage is calculated. The incremental horizontal position of the wall at each excavation step is obtained according to equation (2), and the displacement curve of the surrounding parapet is drawn in Figure 8.

Cosine function is used to draw the bending section of the retaining wall in each stage, and then the sum of the maximum horizontal displacement increment of the underground continuous wall in each stage is obtained. The above displacement images are superimposed to obtain the horizontal displacement of the retaining wall calculated by the improved MSD method, as shown in Figure 9. It is found that, in the depth direction, the horizontal displacement curve of the retaining wall first increases and then decreases. The maximum displacement occurs in the depth of 28.5 m of the diaphragm wall; it is 5 m below the fifth support and 2.7 m above the foundation pit base. The maximum displacement value is about 28 mm, approximately 0.06% H (H being the continuous wall depth).

4. Finite Element Analysis Method for Deformation Prediction

This paper also adopted the finite element analysis method to prove the reliability of the improved MSD method.

In the meantime, a modified molar Kulun model (modified Mohr–Coulomb) was adopted to cover the shear dilatancy, shear hardening (the Moore–Kulun model is suitable for loose cemented granular materials, including soil, soft rock, concrete, and other formation conditions. The excavation deformation of the foundation pit can be roughly calculated), and unloading/reloading modulus of the soil. The friction hardening characteristics were used to simulate the plastic shear strain under deviatoric stress. And cap-type hardening is used to describe the volume deformation of the main stress compression. When the material initially yields, there will be multiple secondary yield surfaces on the original yield surface, which can effectively simulate the excavation process of the foundation pit.

4.1. Basic Assumptions

(1) All the rock and soil in the calculation are considered to be uniformly distributed and isotropic elastoplastic materials

(2) Before excavation of foundation pit, it is determined that the soil is consolidated under self-weight stress to exclude the change of initial stress and character of soil caused by relevant construction work before formal excavation

(3) Before the excavation of the foundation pit, drainage operation is carried out and a waterproof curtain is laid to simplify the calculation of the effect of removing drainage and groundwater seepage

(4) In the process of model calculation, the changes in the mechanical properties of soil caused by the construction are excluded

4.2. The Model Size. Midas GTS NX three-dimensional finite element numerical calculation software was used for simulation calculation. The calculation model was built according to the actual engineering conditions with a size of 350 m × 180 m × 100 m, and the excavation foundation pit model, located at the positive center of the soil model, is 151 m long, 19 to 32.2 m wide, 20 to 28 m deep, and 40 m deep, as shown in Figure 10.

4.3. The Boundary Conditions. Three-direction boundary constraints of nodes X, Y, and Z were established on the bottom of the model, corresponding lateral boundary constraints were set on both sides of the model, and rotation constraints were set on the column pile (Figure 11).

4.4. Parameter Selection. The soil parameters referring to geological prospecting data are shown in Table 2. The envelope is made of linear elastic material, and the calculation parameters are shown in Table 3. Slab unit model was adopted for the ground wall, side wall, and floor slab of each floor; the beam unit model was adopted for the waist beam, antipulling pile, and vertical column of each floor. All structures were simulated in accordance with the design and construction drawings.

4.5. Deformation Calculation of Finite Element Method

4.5.1. Analysis of Initial In Situ Stress Balance. Before the official start of excavation, the displacement of each item should be guaranteed to be zero, and the self-displacement of the model under the action of gravity and the consolidation state of rock and soil should be simulated. This steady state is taken as the initial state before officially starting the excavation (Figure 12).

4.5.2. Analysis of Horizontal Displacement of Retaining Wall. For the midpoint position of the long edge of the foundation pit, that is, the site monitoring point ZQT05, the horizontal displacement of the retaining wall after the completion of the main structure of the site is compared (Figure 13). At the same time, the horizontal displacement data of site monitoring point ZQT05 in the construction link of the main structure is selected, and the results are shown (Figure 14). During the construction of the main structure of the north bank working well, lateral displacement occurred on all sides of the foundation pit wall, and the displacement moved towards the interior of the foundation pit (the accumulated change was positive). In the depth direction, the
5. Field Monitoring Data Analysis

5.1. Monitoring Sites. The deep horizontal displacement of the retaining wall can quantitatively reflect the deformation of the underground diaphragm wall in the depth direction and is the most direct and effective monitoring project. Deep horizontal displacement monitoring points should be set at the characteristic positions of the retaining wall, such as the middle part of the edge of the foundation pit and the positive angles. The distance between adjacent monitoring points should be controlled between 20 m and 50 m. During the construction of the underground diaphragm wall, the incline-measuring pipe and the reinforcement cage are tied in advance and then poured together into a wall, as shown in Figure 15.

5.2. Monitoring Data Analysis. Figures 16 and 17 reflect the deep horizontal displacement of the surrounding parapet at the monitoring points ZQT05 and ZQT15 (AA monitoring section in Figure 1) after excavation of each layer of soil.

At monitoring point ZQT05, after the excavation of the second layer of soil and the installation of the second concrete support, the displacement peak occurred at a depth of about 13 m. After the third layer was excavated and the lower steel support was applied, the displacement peak value increased to 11 mm, appearing at about 15 m. After the foundation pit excavation was completed, the displacement peak increased to about 28 mm, and the position was located at 26 m. At monitoring point ZQT15, after the excavation of the second layer of soil, the displacement peak of 5.5 mm appeared at about 11 m. After excavating the
**Figure 9:** Deformation superposition of continuous wall.

**Figure 10:** Numerical simulation model. (a) Calculation model. (b) Continuous wall structure.

**Figure 11:** Boundary condition diagram of the model.
third layer of soil, the maximum displacement increased to 10.5 mm, and the position moved down to about 13 m. After the third excavation, the maximum displacement increased to 18 mm, and the position moves down to about 20 m. After foundation pit excavation is completed, the maximum displacement increased to 21 mm, and the position moved down to 22 m. To sum up, the monitored deformation peak value and peak position of the diaphragm wall at each stage are consistent with the ones calculated with the improved MSD method.

It is concluded that the deep horizontal displacement of the retaining wall is mainly affected by the depth of the soil layer.
excavation, the form of the enclosing structure, stiffness, the depth of the embedded soil layer, and the prestress of internal support. With the increase of excavation depth, the maximum horizontal displacement of the retaining wall structure increases, and the peak position moves downwards.

6. Comparison of Improved MSD, Finite Element Analysis, and Monitoring Data

6.1. Trend of Data Curve. In order to verify the effectiveness of the improved MSD method and the finite element analysis method in practical engineering, the deformation prediction curves of the enclosing wall obtained with the two methods were compared with the monitoring data, as shown in Figure 18.

As shown in the figure, when the deformation prediction of the center point of the long side of the foundation pit is carried out, the deformation trend and peak position calculated by the improved MSD method and the finite element analysis method of Midas software are basically the same as those calculated with the actual monitoring data. The deformation of the data points shows a “belly-like” distribution, which first increases with depth and then decreases. When the depth is between 0 and 18 m, the field monitoring data of ZQT15 is greater than the results calculated with the improved MSD method and the finite element method, and the field monitoring data of ZQT05 is smaller than the results calculated with the improved MSD method and the finite element method. However, when the depth is greater than 18 m, the field monitoring data and the results calculated by the improved MSD and the finite element method show an opposite trend. That is, the overall trend of the ZQT15 field monitoring data curve is less than the curve trend calculated with the improved MSD method and the finite element method, and the overall trend of the ZQT05
field monitoring data change curve is larger than the results calculated by the improved MSD method and the finite element method. The overall trend of changes in the curve is the same.

6.2. Peak and Position of Horizontal Displacement. At the ZQT05 measuring point, the horizontal displacement and its peak position are shown in Table 4.

The peak position obtained by numerical simulation is located at a depth of 28 m, and the peak value is about 20 mm. The peak position calculated by the improved MSD method is located at the depth of 27 m, and the peak value is 29 mm. However, the peak position obtained by actual monitoring is located at a depth of 28 m, and the peak value is 30 mm.

It is clear that the horizontal displacement of the retaining wall obtained with the improved MSD method is
Figure 17: Change of the deep horizontal displacement of the enclosure wall at monitoring point ZQT15.

Figure 18: Comparison between the predicted value and the monitored value.
closer to the in situ monitoring data. Especially, at depths below 22 m, the improved MSD method performs better than the measured data than the numerical simulation.

7. Conclusions

This paper is based on the foundation pit engineering of the working well on the north bank of a tunnel in Jinan. Taking into consideration of the typical stratigraphic characteristics of the tunnel, a new law of conservation of energy is constructed for the traditional MSD (mobile strength design) theory to put forward an optimized MSD method that is used to calculate the deformation of the foundation pit of the shield working well on the north bank of the tunnel. Finally, the calculation results of foundation pit deformation by the optimized MSD method, finite element method, and on-site monitoring data analysis method are compared to demonstrate the reliability of the prediction system of the optimized MSD method. The conclusions are as follows:

1. This study introduces the bending strain energy of the wall and the compressive elastic potential energy of the internal support to improve the energy conservation system of foundation pit deformation. The horizontal displacement curves of the continuous wall in each step of the excavation are obtained with the improved MSD method, which are superimposed to obtain the final horizontal displacement curve of the retaining wall of the foundation pit.

2. Midas GTS NX numerical calculation tool is used to analyze the displacement deformation of each step of the excavation. And it is found that the horizontal displacement of the parapet obtained by numerical simulation showed a trend of first increasing and then decreasing in the direction of depth. The peak value is concentrated at a depth between 27 to 28 m. The peak deformation value after the foundation pit excavation is 16.9 mm, with a small overall change.

3. The predicted deformation values are close to the monitoring data and show the same trend, which proves the reliability of the improved MSD method in continuous wall deformation prediction with finite element analysis and shows high reference value for similar projects.

Data Availability

The data adopted to support the findings are included in the article.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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