Study on Stability of Super-high Straight-web Cellular Steel Sheet Pile Cofferdam

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Abstract. In recent years, steel sheet pile cofferdam has been adopted in some hydropower reconstruction and extension projects because of its many advantages. However, due to insufficient understanding of the bearing mechanism of super-high cellular cofferdam and lack of Engineering experience, there is no exact method to accurately predict the deformation and failure forms of the cofferdam, and there is no perfect safety and economic design theory, which restricts its popularization and application. Taking the Tarbela 4th Extension Project as background, the mechanical behavior and stability evaluation of the cofferdam in 30m deep water during construction and operation are studied by FEM, and influence of structural form and parameters on deformation characteristics of key parts is analysed. Results show that the maximum interlock tension occurs near mudline or top of berm during high lateral load. Development distribution of shear deformation are more in line with the Kitajima Shoichi’ method. The overturning deformation of cell can be effectively restrained by embedment, the rotation point changes regularly with the increase of berm and embedment. The sliding surface in cell conforms to Hansen's calculation theory, the position of rotation and change of base pressure are closer to the Japanese standard. With the stiffness loss, the change of interlock tension is similar to the damage of low carbon steel, the shear sliding surface is related to berm which above the top overturning deformation is the largest. Based on the stability of cofferdam, there are various combinations of berm height and embedded depth, which can reduce the quantity of embankment works and the cost of later maintenance. The research results are of reference significance for the design and construction of similar super-high steel sheet pile cofferdams.

1. Introduction

The cellular steel sheet pile cofferdam is low in engineering cost, short in construction period, and small in space, making it advantageous in a general sense. It has been widely applied in the construction of water conservancy hubs, wharfs, and piers [1]. Based on traditional design theory,
Experimental research in Japan, Europe and the United States and engineering practice experience, construction technical specifications of the cellular cofferdam is designed [2-3]. Due to immature design theory and the bearing mechanism and complicated experimental conditions, no method has been put forward to accurately predict the modes of cofferdam deformation and failure. Foreign scholars represented by American professor Clough et al. [4-10] applied first the finite element method to the cellular steel sheet pile cofferdam. As for the first to third stage of cofferdam project of Melvin Price Lock and Dam 26 (R), the effects of interlock slack and yield conditions, cofferdam size and embedded depth on the interlock tension, deformation of cell, failure surface, and the location of the rotation point during the load-bearing period of the cellular cofferdam are considered successively, and the two-dimensional and three-dimensional stability characteristics and failure mechanism of the project are studied in relative depth. Under the condition of initial geostress, filling method, and construction sequence, the three-dimensional response of the cofferdam during the load-bearing stage is reflected more realistically by analyzing the interlock tension and the characteristics of deformation during construction and operation of cellular cofferdam. Further to demonstrate that the empirical design method is conservative and that it is feasible to make it simpler, laying a solid foundation for the application of the cellular-type steel sheet pile structure in hydraulic engineering. With the advancement of computer technology, the improvement of finite element calculation methods and the limitation of experimental conditions, numerical analysis methods have more favoured by scholars faced with complex problems. Domestic scholars represented by Professor Wang Yuanzhan et al [11-14] have carried out in-depth research on the mechanical characteristics of cellular steel sheet pile breakwater under wave load by analyzing the failure mode, instability characteristics and stress distribution law of cellular structure with the joint rotation between piles and nonlinear contact effect between pile and soil taken into consideration using shell element simulation and three-dimensional model of cellular steel sheet pile structure established by ABAQUS, thereby accumulating relevant calculating experience.

Different from port projects, in hydropower projects, the cellular-type steel sheet pile structure is a temporary cofferdam with one side to retain water, mainly considering hydrostatic pressure. Scholars at home and abroad have carried out related research on the stability and failure mechanism of the cellular steel sheet pile cofferdam with low and medium height. They believe that the interlock rupture during the filling period is the most possible failure mode of the cofferdam [15]. The shear deformation in cell basically conforms to the Terzaghi theory with overturning and rotation point inside the cell. However, due to scale limit, there is little research on the failure mechanism of the super-high steel sheet pile cofferdam and the prevention of interlock rupture and filling leakage. There is also lack of understanding about the cofferdam stability and economic impact brought by the construction of a berm and its height change. Although the traditional method is simple and widely used, it fails to consider the pile-soil interaction, making it limited in application and conservative in results. The numerical analysis method based on finite element method can simulate the nonlinear mechanical behavior between the pile and soil and the spatial effect of the force deformation, making it an important, efficient, and reliable means to analyse the stability of steel sheet piles.

The maximum depth of water in the cellular cofferdam of the fourth phase extension project of Tarbela Hydropower Station is 30 m, larger than the water retaining height specified in the specification [16], making it the highest temporary water retaining cofferdam in hydropower projects worldwide. The cellular structure is relatively less used in domestic hydropower construction, let alone the design and construction of 30 m deep cellular cofferdam [17], leaving prominent problems concerning stability and cost. Therefore, it is of great theoretical and practical significance to study the law of earth pressure distribution, interlock tension, shear and overturning deformation under the different embedded depth, sheet pile stiffness and berm height of the super-high steel sheet pile cofferdam and explore stability characteristics and failure modes during construction and operation.
2. Overview

The Tarbela Hydropower Project is located 113 km northwest of Islamabad and is mainly used for irrigation, power generation, and flood control. It includes main and auxiliary dams, main and auxiliary flood discharge tunnels, irrigation tunnels, diversion tunnels, and power stations, with a total installed capacity of 3478 MW and the height of 143 m. In the fourth phase of the extension project, it is required to construct a cofferdam in the downstream of the power house, connected to the retaining wall of the third machine room in the east and the west side of the bank. Due to site limitation, the straight-web steel sheet pile cofferdam is adopted. The fourth machine room is located in the downstream of the reservoir area with a maximum water depth of over 30 m (Figure 1).

![Figure 1. Cofferdam axis layout diagram](image)

Note: The thick red line in the drawing is the axis of cellular steel sheet pile cofferdam.

The overburden on the left side of the cofferdam exceeds 20 m, the water depth is 23 m to 30 m, and the underwater ground elevation is 310 m to 316 m. The overburden is divided into three layers from top to bottom: gravel layer (CG), silty sand (SM), gravel and pebble silt soil (SMg). The water depth of the right part of the cofferdam foundation is 3 m to 30 m, the underwater ground elevation is 310 m to 338 m with little coverage on the bedrock. However, here is a strong weathering layer in the carbon schist. The bedrock is mainly composed of carbonaceous schist and limestone while diabase is near the right bank.

The cellular cofferdam faces the west with a length of 206.87 m with its left bank connecting with the retaining wall of the previous plant and its right bank connected with the mountain. The cofferdam consists of 5 main cells and 4 arc cells, which are connected by wye piles. The diameter of the main cell is 23.76 m, while the maximum radius of the arc cell is 5.73 m. For the overall layout of the cofferdam, see Figure 2(a). For cellular connection and monitoring point arrangement, see Figure 2(b). The maximum water level at the downstream is 341.5 m, the dome elevation is 343.0 m, and its maximum height is 32 m. AS 500-12.7 straight-web steel sheet pile (Table 1) is used. For its interlock connection, see Figure 3. The geological section at the central axis of the 2# main cell is a typical section. Its surface is mainly drifting gravel layer. The berm height is 20 m with 2.5 m deep sheet piles (Figure 4).
3. Model generalization and model of finite element analysis

3.1. Mesh model
Calculation range: Take 2# main cell and 1# arc cell as the research object and the cellular center as the coordinate origin. The outboard face is set as the X-axis positive direction and the vertical direction the Z-axis positive direction, while the Y-axis is set based on the right hand rule. In order to improve calculation efficiency, it is necessary to take half of main and arc cells with a total length of 15.74 m (Figure 5) along the Y-axis direction to calculate using symmetry of the load and structure. In order to reduce the boundary effect, it is necessary to take nearly 6 times the cellular diameter from the inboard side to the outboard side along the X-axis, totaling 140.0 m and 68.0 m below the surface along the Z-axis with the top and bottom elevations of the model being 343.0 m and 245.0 m respectively.

| Section   | Density/kg·m⁻³ | Elastic modulus/GPa | Thickness/t/mm | Interlock resistance/kN·m⁻¹ |
|-----------|----------------|---------------------|----------------|-----------------------------|
| AS 500-12.7 | 7850          | 210                 | 12.7           | 5500                        |

Figure 2. Plane layout of cellular cofferdam
Figure 3. Straight-web pile section
Figure 4. 2# main cofferdam section
3.2. Mechanical model

The Mohr-Coulomb constitutive model (MC model) can well describe the disruptive behavior of soil and other discrete materials. Its physical and mechanical parameters can be measured by the routine test method, which is easily and widely used in geotechnical engineering [19]. The classical M-C criterion is an equilateral equiangular hexagon on the plane. If the tension is positive, the shear yield function in the three-dimensional principal stress space can be expressed as:

\[ F = (\sigma_1 - \sigma_3) - (\sigma_1 + \sigma_3) \sin \varphi - 2 \cos \varphi = 0 \]  

(1)

Note: \( \sigma_1 \) and \( \sigma_3 \) are the first and third principal stress; \( \varphi \) is the internal friction angle of the material.

To avoid the cumbersome calculation and slow convergence caused by the sharp corner of yield surface, the extended classical M-C constitutive model is used to simulate force deformation of soil under hydrostatic load. This model selects a continuous smooth flow potential function to ensure that the plastic flow runs in the same direction. The yield characteristics of the extended model are expressed by \( p \), the equivalent stress and \( q \), the equivalent stress of mises. The M-C yield function can be obtained as follows:
\[ F = - p \sin \varphi + \frac{q}{\sqrt{3}} K(\theta) - c \cos \varphi = 0 \] 

(2)

**Note:** \( p \) is the equivalent compressive stress; \( q \) is the equivalent stress of \( \text{mises} \); \( \theta \) is the Lode angle.

According to the safety coefficient of steel sheet pile interlock stability specified in the US specification [2], the straight-web steel sheet pile is still in elastic state during the loading period. Therefore, it is acceptable to adopt the linear elastic constitutive model to simulate the deformation of the steel sheet pile.

Hard contact theory and the penalty friction model are adopted for the contact surface between the pile and soil. Since the steel sheet pile is much more rigid than the soil body, the contact surface of the steel sheet pile is set as the main control surface while that of the soil body is set as the subordinate surface. Since the contact analysis is highly nonlinear, it is necessary to adopt surface-to-surface contact method, the finite slip formula and reasonable position error limit and small interference to solve the problem of calculation convergence [20].

**Table 2. Recommended values for the main indicators**

| Parameter | Saturated unit weight/(kN·m\(^{-3}\)) | Moist unit weight/(kN·m\(^{-3}\)) | Compressive modulus/M Pa | Friction angle/° | Cohesive n/kPa |
|-----------|-------------------------------------|---------------------------------|-------------------------|-----------------|---------------|
| Coarse sand | 19.0 | 17.0 | 33.0 | 32 | 0 |
| Sand gravel | 20.0 | 18.0 | 25.0 | 21 | 0 |
| CG | 21.0 | 19.0 | 28.0 | 35 | 0 |
| SM | 19.0 | 17.0 | 12.0 | 25 | 7.0 |
| SMG | 20.0 | 18.0 | 16.0 | 28 | 5.0 |
| CS | 28.0 | 25.0 | 22000.0 | 34 | 350.0 |

**3.4. Construction simulation**

Firstly, apply gravity to the foundation soil and generate the initial geostress field without displacement using the automatic stress balance method (Figure (a)). Then, activate the steel sheet pile and the filling, the berm and the bank respectively and establish stable contact between them and the steel sheet pile. Later, apply gravity and lateral water pressure in turn to simulate the state of stress and deformation during construction and water retaining (Figure 8(b) to Figure 8(d)).

![Figure 8 Key construction stage of cofferdam](image)
4. Verification of finite element model

4.1. Empirical design
Terzaghi (1945), TVA (1957), and Schroeder and Maitland (1979) are the methods commonly used to calculate the lock tension. They differ in the maximum value of soil pressure and its distribution [5]. Since there is symmetry force on the cofferdam, it is necessary to take the four key positions on the top of main and arc cells on the inboard side (Figure 9) to analyse the interlock tension.

In the empirical design method, the distribution and calculation of earth pressure are mainly based on the US specification [2]. According to TVA (1957), the interlock tension $T_m$ is calculated at the junction of main and arc cell.

$$ p = 1.2 \sim 1.6 k_u \sigma'_v $$
$$ T_m = pL \sec \alpha $$

Note: $k_u$ is the Rankine active earth pressure coefficient; $\sigma'_v$ is the vertical effective stress of the soil inside the cell; $P$ is the effective lateral earth pressure; $L$ is the distance between the center of the main cell and that of the arc cell; $\alpha$ is the angle between the axis of the cell and the line connecting the main cell and the pile.

It can be seen from Figure 10 that the distribution law of the interlock tension obtained by using the empirical design method and the finite element method is basically the same. It is because the sheet pile is relatively shallow, the interlock tension at the joint of the main and arc cells is almost linear, which is similar to that of the layer without any cover. Its maximum value is about 1745.6 kN·m$^{-1}$, smaller than the designed value of 2331.9 kN·m$^{-1}$. Since the empirical design method does not take into consideration the interaction between the key elements such as the cell, filling and the basic system, its design value tend to be conservative [5].

![Figure 10](Image)

4.2. Practical measurement
During the monitoring period, with the influence of construction, water level fluctuation, and drainage rate, the value calculated using finite element method is slightly different from the measured deformation value (Figure 2b). After the drainage is completed, the deformation error range is within 13.8%, and the deformation values are all smaller than 250 mm, thus meeting the engineering requirements (Table 3).
Table 3. Engineering and FEM calculated values

| Monitoring point | Engineering measurement (mm) | Model calculation (mm) | Range (B-A)/B (%) | Allowable deviation/mm |
|-----------------|-------------------------------|------------------------|-------------------|-----------------------|
| 7#              | 129.2                         | 126.9                  | 2.0               | 250.0                 |
| 8#              | 112.7                         | 130.8                  | 13.8              |                       |

The result of empirical analysis and engineering measurement shows that the finite element model has high accuracy when it comes to the construction and water retaining period.

5. Stability analysis
Stability characteristics of the cellular steel sheet pile cofferdam are different at different stages. Generally speaking, the problem of the steel sheet pile burst is particularly critical in the cell filling period; the interlock tension, cellular deformation, and the failure form are more complicated in the super-high water load period, thus the stability problem of the cofferdam is more prominent.

5.1. Construction period of cell and berm
Interlock rupture or the web pile is pulled away of the cell close to the ground usually occurs after the completion of cell filling. The calculation result with the influence of different embedded depths taken into consideration is shown in Figure 11. The actual embedded depth of the cell is 2.5 m, the maximum interlock tension is about 1745.6 kN·m⁻¹, which meets the stability requirements of interlock rupture. As the embedded depth increases, the interlock tension first decreases and then increases and tends to be stable afterwards. When it transits from the bottom of the cell to the mudline, the embedded depth has little effect on the interlock tension. After the berm is built, take a key point of the pile PILE-A, which is slightly higher than the mudline, and its change of the interlock tension is shown in Figure 12. The pressures between the soil inside the cell and the sand gravel inside cancel each other out. As the height of the berm increases, the interlock tension first decreases and gradually tends to be stable. When the berm lower than 5 m, the slope of the tension curve is the largest with maximum reduction of 56.4%. Under this circumstance, the height of the berm has a greater influence on the reduction of the interlock tension.

5.2. Operation period of high water level
5.2.1. Interlock stability
During the operation period of high water level, the cellular cofferdam overturns toward the inboard, the bottom restrains the cell from moving backward, and the outboard sheet pile bulges and deforms...
outward at the mudline to form a distinct transition zone with the maximum interlock tension mainly concentrated near the mudline of the sheet pile in the outboard side (Figure 13).

![Figure 13 Bulge of the outboard sheet pile](image)

It can be seen from Figure 14 that the height of the berm and the depth of the sheet pile can reduce the interlock tension to different extent. The slope of the curve decreases with the increase of the height of the berm until it is stable. With constant height, the interlock tension decreases with the increase of the embedded depth, and the decrease range is quite different. Under different conditions, the two have different degree of influencing interlock tension. When the berm is ≤10 m, the slope of the interlock tension curve is the largest with different embedded depths. The bulging deformation of the bottom sheet pile in the outboard decreases, which means that the berm plays a major role in restraining the backward deformation of the cell and reducing the interlock tension. As embedded depth increases, the sheet pile gets harder, thus larger influence on the interlock tension and the slope of the curve. When the embedded depth exceeds or equals 10 m, the difference between the slopes of the curve is small, and the curve is almost parallel to the horizontal axis, indicating that at this stage, the berm height and the embedded depth have insufficient influence on the interlock tension; that is to say, its change is controlled by other factors in the design. Take the design value of the interlock stability under the operation condition of high water level as the standard, the berm height with different embedded depth that meet the engineering requirements can be obtained.

As shown in Figure 15, under the influence of the height of the berm and the depth of the sheet pile, the positions of the maximum interlock tension changes accordingly, which are mainly concentrated on the top of the inboard berm and below the outboard mudline.

![Figure 15. Position of maximum interlock tension](image)

The berm height has a significant impact on the interlock tension. Take 2 m and 8 m berms as the boundary of the position change. when the berm is about 2 m to 8 m high, the maximum interlock tension is close to the top of the inboard berm, and the rest is below the outboard mudline. In addition, the depth decreases with the increase of berm height, and the position gradually approaches the mudline. Since 2.5 m is relatively shallow, the position of maximum interlock tension is always below the outboard mudline and it shows similar change with the increase of the berm height. In particular, when the height is 20 m, the maximum interlock tension is about 2155.2 kN·m⁻¹, and is located 0.5 m below the mudline. Under the same condition, the designed value of the maximum interlock tension is
about 2331.9 kN·m$^{-1}$ located at the mudline. The calculation results are relatively close. Under the condition of high water level operation, in order to ensure the interlock stability, it is advisable to check the maximum interlock tension near the outboard mudline or at the top of the inboard berm.

5.2.2. Internal shear

With the influence of the surface soil on the filling in cell taken into consideration, shear deformation inside cell under different berm heights and embedded depths is shown in Figures 16-17. When there is no berm, the cell is overturned because of high lateral load, then the soil pressure of the backwall filling decreases to the ultimate active state. Insufficient shear capacity of the filling leads to a bended local plastic zone near the bottom of the cell (Figure 16(a)). As the embedded depth decreases, plastic deformation extends to the upper of the cell and forms an angle close to 90° with the bottom surface, and gradually approaches the middle. Meantime, plastic deformation also occurs in the soil of the arc cell (Figure 16(b)). When the embedded depth is reduced to 2.5 m, the deformation surges from middle to the outside of the cell, and then develops along the central axis, finally forming a semi-conical failure surface, which is similar to the shearing theory proposed by Kitajima Shoichi (Figure 16(c)). During shear deformation, plastic failure occurs first near the outboard sheet pile inside the cell. With the continuous development of deformation, vertical shear deformation occurs on the axis of the cell and extends to about the height of the intermediate cell, which is different from Terzaghi’s shear failure assumption. This is because the traditional shear theory makes the cell independent and ignores the influence of the underlying soil layer on the upper part. Obviously, this approach is conservative.

As shown in Figure 17, different from the above phenomenon, when the berm is 5 m, the lateral confining stress of the soil and the shearing capacity increases under lateral load. Only a local strip plastic zone is formed at the top of the berm, and the change of the embedded depth has little effect on the shear deformation. When the embedded depth is 2.5 m, the plastic zone is still at the top with the increase of the height, but shear deformation is obviously improved, indicating that the shear deformation is greatly affected by the height of the berm during water retaining period.
5.2.3 Overturning
The steel sheet pile cofferdam is overturned to the inboard as a whole, and its deformation decreases with the decrease of the cell height (Figure 18). When there is no berm and the embedded depth is 2.5 m, the cofferdam stability moment is insufficient and overturns to the inboard. The rear soil is squeezed and uplifted with overturning deformation about 3292 mm (Figure 18(a)). When the berm is 20 m and the embedded depth is 12.5 m, the cell and the berm resist the lateral load together with no obvious overturning and the deformation is about 114.9 mm. (Figure 18(b)). The cofferdam stability in the two extreme situations is significantly different.

![Figure 18 Overturning deformation of cofferdam](image)

It can be seen from Figure 19 that the overturning deformation of the cell is affected by the berm height and the embedded depth. When the berm is ≤ 5 m, the deformation reduction and the slope of the curve are the biggest under the same embedded depth. At this stage, the berm height has a significant influence on the cofferdam overturning and deformation. When the berm height is 5.0 m or more, the deformation curve goes downward in a linear manner and the slope decreasing gradually with the increase of the embedded depth. The overturning deformation of adjacent curves are slightly different with the change of the berm. The embedded depth has little effect on the overturning and deformation while the berm height plays a dominate role in this respect. Therefore, the berm height which meets the engineering requirements can be obtained under different embedded depths by taking the allowable deviation of engineering overturning deformation as the standard.

![Figure 19 Overturning deformation curve of cofferdam](image)

5.3 Effect of stiffness loss on stability
The stiffness of straight-web piles has an important influence on the stress and deformation of the cellular cofferdam. During the operation of high water level, AS 500-12.7 straight-web steel sheet pile is still used to simulate the influence of the stiffness loss on the stability of the cellular cofferdam by reducing the elastic modulus. Due to space limit, this paper takes the embedded depth of 12.5 m as an example.
\[ E_i = \frac{1}{1 + \alpha_i} E_0 \]  

\textbf{Note:} \( E_i \) is the elastic modulus after reduction; \( \alpha_i \) is the reduction factor; \( E_0 \) is the initial elastic modulus.

5.3.1. \textit{Interlock failure}

As shown in Figure 20, the interlock hoop stress shows a similar trend with the change of the reduction factor, and the rupturing location is located at the top of the berm or overburden. Since the elastic modulus of the material is reduced because of plastic deformation [21], the steel sheet pile with the decrease of stiffness is similar to the tensile failure of low carbon steel. When there is no berm, the change of stress growth rate can be divided into three stages: stage 1 in which the stress increases slowly while the restraining reaction force increases; stage 2 in which the initial stress growth rate is the largest, and then gradually slows down to reach the tensile limit; stage 3 in which the stress decreases sharply with a significant shrinkage, resulting in concentrated deformation. The severe bulge and interlock rupture of steel sheet pile occurs at the mudline at the critical moment of b and c (Figure 21). When the berm is set, the tensile limit of the steel sheet pile is delayed with the stress change curve moving downward. When the berm reaches 15 m, the stress curve drops almost linearly as the berm height increases.

5.3.2. \textit{Shear failure}

It can be seen from Figure 22 that the weakening of the steel sheet pile leads to cell bulge as the cell receives the pressure from the soil side. As deformation increases and the pile is pushed away from the soil, the soil inside the cell reaches the active limit state. The plastic deformation firstly starts from the top of the berm or overburden, and then extends toward the top of the cell. When the berm is lower than a certain value, the plastic surface develops to the middle of the cell, and the steel sheet pile has ruptured (Figure 22(a)-(b)). As the berm height increases, the pressure on the soil side decreases. Before the pile ruptures, the soil inside the cell becomes unstable and forms an active rupture surface, which is similar to the phenomenon of soil landslide with the unstable sliding surface similar to the plane (Figure 22(c)~(e)), which is similar to the phenomenon of Kitajima Shoichi’ test.

5.3.3. \textit{Overturning failure}

Weaker stiffness of the sheet pile leads to less tightening impact on the cell filling. Part of the materials in the cell contributes to anti-overturning. The overturning deformation increases with the increase of the reduction coefficient (Figure 23). When the berm is set, the change of initial overturning curve is almost linear with slope decreasing as the berm increases. The embedding depth
has some, but not crucial influence on the initial deformation. As the berm height increases, the curve is almost parallel to the horizontal axis. When the reduction coefficient is greater than 0.9, the slope of the curve increases. As the sheet pile softens, the filling at the top of the berm slides, which leads to the inboard surge of overturning.

6. Position of cell rotation point

6.1. Finite element method

In the anti-overturning stability analysis, the position of the rotation point is crucial to the stability of the cofferdam. To describe the change of the position of the rotation point, this paper sets the center of the bottom of the cell as the coordinate origin, the direction of the outboard as the positive X-axis, and the vertical direction as the positive Z-axis (Figure 24).

Figure 25 shows the positional change of the rotation point during high water level operation when no berm is provided. Cellular deformation is composed of a number of strip arcs. Take the bottom as the reference and determine the coordinates of the center of the circle as the rotation point by geometric calculation. As embedded depth increases, it moves gradually from the midpoint below the cellular bottom to the bottom and outboard of the cell. When the embedded depth is increased to 10 m, the rotation point moves closer to the outboard sheet pile and slides down the vertical axis. That is because the as embedded depths increases, the hardening effect grows stronger, which is similar to the rigid body structure. In this way, the rotation point moves closer to the front toe of the cell.

As shown in Figure 26, take the top of the pile of inboard cell as the origin with the vertical axis Z and the horizontal axis X pointing to the inner and outer layers of the cell respectively. With the embedded depth of 2.5 m as an example, the rotation point moves upward from a point below the
bottom of the cell to the tip of the pile, and then move along the horizontal line to the outer cell with the increase of the berm height. As shown in Figure 27, rotation points with different embedded depths are mostly located in the outside soil layer and move farther away from the pile tip as the height of the berm increases with unilateral displacement increase almost linearly. That is because the berm plays a key role in restraining the overturning deformation of the cofferdam, making the rotation point move away from the pile tip and towards the bottom of outside cell. The cell and internal filling take the initiative, and the anti-overturning stability is improved.

6.2. Comparative analysis of finite element method and empirical method

6.2.1. Comparison with the Hansen method
During the high water level operation without the berm, the overturning deformation of the cofferdam is the largest with the whole cell similar to the rigid structure rotating around a certain point in the foundation. Plastic zone appears between the front and rear pile tips (Figure 28(a)). Since the cell filling and the foundation soil are similar to some extent, the upper structure and the foundation are coordinated in deformation with the rotation point locating at the bottom of the cell. The failure surface is approximately circular and located inside the cell, which is basically consistent with the result of Hansen’s theory (Figure 28(b)).

That method regards the cellular structure as a rigid body and the failure surface as a logarithmic spiral. Under non-basement condition, the cofferdam failure surface is in the cell or the foundation with the rotation point locating between the sheet piles, which can be determined using tests or the geometric method. Influenced by the embedded depth, the plastic zone in the cell begins to dissipate from near the bottom of the inboard sheet pile of cell (Figure 29(a)) as the embedded depth increases. Then it moves to the outboard pile tip until it disappears, which is the opposite to the development of the failure surface (Figure 29(b)). That indicates that with appropriate increase of embedded depth, the plastic zone still inside the cell, which is in line with the above-mentioned calculation theory. When the cellular structure has a tendency to overturn, the plastic zone begins to develop at the outboard pile tip, which means that the outboard pile tip soil has been broken at the beginning of the overturning and there may be a “leakage effect” during rotation. Further increase of the embedded depth or the establishment of berm can prevent plastic deformation of the soil (Figure 29(c)-(d)).
6.2.2. Comparison with Japanese specification method

As mentioned above, the rotation point moves upward from a point below the cellular bottom to the tip of the pile, and then moves along the horizontal line to the outside soil layer. The base stress distribution transitions from a triangle to a trapezoid (Figure 30), which is similar to the cellular anti-overturning model in the Technical standards and commentaries for port and harbour facilities in Japan[3]. This model assumes the cell as a rigid whole and divides the cellular structure into four kinds of force models according to the position of the rotation point, then obtains positional parameters using iterative method, and finally determine whether the cellular structure is unstable based on the sand body model test and by using the ratio between the horizontal displacement of the cell top and the height above the mud line, which is less than 1.5%, as a control value. Although this method takes into consideration the relationship between the rotation point and the pressure on the cell and there is no need to assume the position of the rotation point, it ignores the case where the local soil of the foundation is in tension when the rotation point is inside the cell, and does not consider the possible "leakage effect".

It can be seen from the above analysis that the assumption concerning the rigid structure and rotation mode obtained from the empirical method are consistent with each other. Taking the cell as the rigid structure rotating around the pole can reflect the cellular overturning mode authentically. However, with different emphasis, there exist different criteria for evaluating anti-overturning stability and uncertain factors such as theoretical assumptions and computational simplification, which is the limitation of the empirical method. In comparison, the finite element method takes into consideration...
the soil-structure interactions, the coordinated deformation of filling and foundation, and can simulate the real stress state of the cell. It can produce more accurate position of the sliding surface and rotation point in different cases, making the result more credible. Result shows a correlation between the position of the rotation point and overturning deformation and the stress distribution of the base under the impact of the berm and the embedded depth, that is, as the rotation point moves closer to the outboard side near the cellular bottom, the deformation gradually decreases, and the stress distribution of the basement transitions from a triangle to trapezoid, making the cofferdam more stable in its anti-overturning performance.

7. Design analysis
In this project, the designed embedded depth of the 2# arc cell and 2# main cell is 2.5m with a 20 m berm added based on the experience. Calculation result shows that on the one hand, the stability of the cofferdam is satisfactory and the design is reliable; on the other hand, if the embedded depth increases, the height of the berm can be appropriately reduced while ensuring the stability of the cofferdam, meaning that there are various combinations of berm height and embedded depth, which can reduce the work amount of berm and its maintenance cost. Relevant literatures and specifications suggest that the berm height should be determined based on engineering experience without providing any reasonable calculation method. The conservativeness of the empirical method may lead to an increase in economic cost. It can be seen from Table 4 that when the embedded depth is maintained at 2.5 m on the basis of ensuring stability, the berm height can be reduced to 15 m; if the depth is increased to 5 m, the berm can be reduced by half. With the increase of the embedded depth, the “hardening effect” of pile can make the cofferdam more stable. However, when driving piles, too thick or too dense overburden may cause the interlock to fall off. In this regard, the US specification [2] suggests setting the embedded depth less than 9.1 m. Therefore, taking into consideration the original design and technical and economic factors, when the embedded depth is 2.5 m, the berm height should be 15 m to 20 m. If the embedded depth increases, the berm height can be further reduced, but it should not be lower than 10 m.

Table 4. Combined scheme of berm and embedment

| Berm height/m | 5.0 | 10.0 | 15.0 | 20.0 |
|---------------|-----|------|------|------|
| Stability requirement | A | B | C | A | B | C | A | B | C |
| Embedded depth/m | 2.5 | − | − | − | + | − | + | + | + | + | + |
| | 5.0 | − | − | − | + | + | + | + | + | + |
| | 7.5 | − | − | − | + | + | + | + | + | + |
| | 10.0 | − | + | + | + | + | + | + | + | + |
| | 12.0 | − | + | + | + | + | + | + | + | + |

Note: A-Interlock stability, Tension≤2331.9kN m⁻¹; B-Overturning deformation, Displacement≤250mm; C-Shear failure, Non-plastic deformation: + (−) - Satisfy (Dissatisfy) requirement

8. Conclusion
This paper studies mechanical behavior and stability evaluation of the cellular cofferdam with 30 m water depth during its construction and operation using the finite element method based on the cellular steel sheet pile cofferdam of Tarbela Phase IV Extension Project. This paper analyses the influence of structural form and parameters on the deformation characteristics and mechanical behavior of key parts, thus providing reference for the design and construction of super-high cellular steel sheet pile cofferdam for hydropower reconstruction and extension project.

(1) The result of the interlock tension distribution of the super-high cellular steel sheet pile cofferdam gained from numerical calculation and empirical calculation are basically the same with each other. The deformation values gained from numerical calculation and from practical engineering are basically consistent with each other, indicating the high accuracy of the model.
During cell filling and the berm placement, the stability of the cofferdam in this project is satisfactory. The key measure to improve the interlock tension is not to increase the embedded depth of the sheet pile, but berm placement to decrease the pulling effect on the interlock of the inboard side. Different from general situations, the problem of interlock stability is particularly prominent during high water level operation. The berm placement and increase of the embedded depth both play a key role in reducing the interlock tension. It is appropriate to check the result by taking near the outboard mudline or the top of the inboard berm when it comes to the maximum interlock tension. The shear deformation inside the cell begins at the bottom of the inboard cell, and finally forms a semi-conical shear failure surface, which is similar to the method proposed by Kitajima Shoichi, but different with Terzaghi theory. The berm placement can increase the lateral confining stress of the soil to increase the shear strength of the cell. When the berm height is increased to a certain value, overturning deformation is linearly reduced under different embedded depths with the berm playing a dominate role. The change of the interlock tension with the change of stiffness of the steel pile and the tensile failure of the low carbon steel are similar. The sliding surface of the cell filling due to shearing instability is approximately flat with the overturning deformation of the berm top being the largest.

The rotation point moves outward from inside of cell with the change of the berm height and the embedded depth. The stress distribution of the base transits from a triangle to trapezoid, and the overturning deformation is reduced. When calculating the anti-overturning stability, the Hansen method fails to take into consideration the rotation point outside the cell, which is a great limitation. The calculation of the cellular stress mode based on Japanese specification can basically avoid the above problems and should be more reasonable. For major projects, it should be verified by model test or finite element method.

When various stability requirements are met, this paper further compares the possible schemes of this project. The result shows that when the embedded depth is 2.5 m, the berm height should be 15 m to 20 m. As the embedded depth increases, the berm height can be further reduced, but not less than 10 m.

Acknowledgments
The fund: The General Program of National Natural Science Foundation of China (51879207)

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