Analysis of tower foundation of long span single tower cable stayed bridge

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Abstract. The pylon is an important stressed structure in the single pylon cable-stayed bridge. The rationality of bearing platform and pile foundation of pylon foundation is one of the key research contents in the design of single pylon cable-stayed bridge. The main bridge of Tarim bridge in Xinjiang is a single tower double span cable-stayed bridge with span arrangement of (41 + 168 + 168 + 41) m and total height of cable tower of 125m. Arc shaped concrete tower columns are adopted. The stress of cable tower foundation is complex. The stress analysis of foundation scheme is one of the key problems of this project. In this paper, the geological conditions, structural stress, economic rationality, construction convenience and other factors are comprehensively considered, the different foundation types are compared and analyzed, the reasonable foundation type and layout type are determined, and the stress of the pile and cap structure of the main tower foundation is calculated and analyzed. Through the solid finite element model, this paper analyzes the influence of the stiffness deformation of pile cap on the stress of pile group foundation. The calculation results show that the stress of dumbbell shaped pile cap is more reasonable than that of rectangular foundation pile cap, and the number of pile foundation and the calculated pile length are reduced in varying degrees. By adjusting the pile foundation arrangement under dumbbell shaped foundation, the stress of pile foundation can be more uniform, which can be used as the foundation for similar projects. The foundation design provides reference and reference.

1. Introduction

Pile foundation is a kind of foundation type with high bearing capacity and wide application range, and it is also a common foundation type for long-span bridges. Many scholars have carried out the research on the mechanical characteristics of pile group foundation, and formed the calculation and analysis method of pile group foundation. The provisions on the bearing capacity and effect of single pile and pile group also have the recommended calculation method in the code.

Pylon is the most important stressed component of cable-stayed bridge. The commonly used pylon types are H-shaped, inverted Y-shaped, A-shaped, diamond shaped, etc[1]. These pylon types are straight or broken line. In recent years, with the development of construction capacity and social economy, variable cross-section arc concrete pylon has become a tower type scheme that can be considered because of its beautiful shape. In addition, the shape and stiffness of the foundation cap and the arrangement of the piles under the cap have a certain impact on the superstructure, so it is necessary to carry out a detailed calculation and analysis of the foundation stress to determine a reasonable foundation scheme[2].

This paper makes a detailed calculation and Analysis on the stress of the tower foundation of the single tower cable-stayed bridge of the main bridge of Tarim bridge in Xinjiang, analyzes the influence...
of the stiffness deformation of the bearing platform on the stress of the pile group foundation, and compares it with the conventional analysis method recommended by the code, so as to finally determine the reasonable foundation scheme and pile foundation layout form.

2. Project introduction

2.1. Overall bridge scheme
Tarim bridge is located in the southeast of Alar city, Xinjiang, across the Tarim River. The bridge connects the East Ring Road in the planning road network of the main urban area of Alar city in the north and the Alar upstream reservoir highway in the south. It is an important part and key project of the first division highway.

Tarim bridge is divided into main bridge and approach bridge. The main bridge is a single tower double span cable-stayed bridge (Figure 1). The main beam is an integral steel box girder, and the cables are fan-shaped spatial double cable planes. In order to improve the structural stiffness of the cable-stayed bridge and effectively reduce the deflection of the main beam and the horizontal displacement of the tower top under live load, an auxiliary pier is set at 41m away from the transition pier for the two main spans, that is, the span arrangement of the main bridge is 41 + 168 + 168 + 41 = 418m. The bridge adopts integral cross-section form, with a total width of 36.5m (including cable anchorage zone), two-way six lanes, and the design load is highway class I.

Figure 1. Elevation layout of bridge

2.2. Construction conditions

2.2.1. Topography
The bridge is located in the alluvial plain of the Tarim River Basin, with open terrain and little terrain change, high in the West and low in the East, high in the South and North and low in the middle, with an altitude of 1008.00 ~ 1013.00 M. Tarim River is characterized by wide and shallow riverbed, developed inner and edge beaches, and developed floodplain on the right side of the main channel, belonging to wandering Valley landform.

2.2.2. Hydrological conditions
The Tarim River is mainly supplied by snowmelt in Tianshan Mountains, and it varies greatly under the influence of climate. The dry season is from April to May every year, and the wet season is from
July to September. The design maximum water level is 1010.859m (once in 100 years), the general scouring depth is 4.42m, the local scouring depth is 2.46M, and the scouring elevation is 1003.979m.

2.2.3. Engineering geology
The stratum at the bridge site is quaternary Holocene alluvial and chemical accumulation layer, mainly composed of yellow (upper) ~ gray (lower) silt, silty clay, silty sand, fine sand, medium sand, etc., with a thickness of more than 150m, mainly distributed in the Tarim River Basin. Among there are 5 layers of fine sand, dense, good engineering performance, high bearing capacity and low compressibility. The soil layer and its underlying soil layer can be used as foundation bearing stratum.

2.2.4. Earthquake
The bridge site is located in the strong earthquake area, the peak acceleration of ground motion is 0.1g, the basic intensity of earthquake is 7 degrees, and the level of seismic fortification measures is 8 degrees, which belongs to the seismic unfavorable section. There is liquefaction phenomenon in the stratum in this area, the liquefaction level is medium to serious, and the stratum is prone to liquefaction during earthquake, which has a certain degree of impact on the proposed buildings.

3. Foundation stress analysis

3.1. Foundation form selection
For long-span bridges such as cable-stayed bridges, the cable tower foundation should have larger bearing capacity, and the suitable deep foundation forms mainly include caisson foundation and pile group foundation. Open caisson foundation has the characteristics of large buried depth, strong integrity, good stability, and can bear large vertical and horizontal loads. However, the disadvantage of open caisson foundation is that the construction period is long, and it is easy to cause quicksand phenomenon when pumping water in the well for fine sand and silt soil. The bridge site is located in the alluvial proluvial plain of Tarim River. The stratum lithology is mainly silty sand and fine sand. Under the action of groundwater for a long time, it is easy to produce adverse geological phenomena such as "flowing mud" and "flowing sand". During the sinking process of open caisson, it is easy to tilt and the construction quality is not easy to control. Therefore, it is not recommended to use open caisson foundation.

Pile group foundation is a kind of deep foundation which connects the top of several piles into a whole through the cap to bear the dynamic and static load together. Among them, large diameter bored pile is a kind of foundation form widely used in long-span bridge structure at home and abroad. It is characterized by high bearing capacity, strong adaptability, and can be used in various complex adverse geological conditions. The design and construction technology are relatively mature.

Combined with the geological soil layer characteristics and construction conditions in Alar area, the existing bridge structure around the bridge site is investigated, and the cable tower foundation form is adopted. Finally, the large diameter bored pile group foundation is adopted.

3.2. Determination of pile diameter
The size of pile foundation should meet the requirements of bearing capacity and deformation, and at the same time, it should be economical, reasonable and convenient for construction. In the design process, the pile foundation layout types of 1.8m, 2.0m and 2.5m in diameter are compared. Due to the large horizontal force and bending moment at the bottom of one side tower column, the foundation is arranged as an integral rectangular cap to reduce the cross bridge reaction of pile group foundation under external force.
According to the requirements of the code, the length of the pile foundation is checked, and the short-term effect combination is adopted for the effect transmitted to the bottom of the pile cap. The comparison of foundation layout schemes with different pile diameters is shown in the table 1 below:

| Pile foundation layout | Pile diameter (m) | Number of roots | Pile length (m) | Concrete (m$^3$) | Concrete volume ratio |Concrete (m$^3$) | Concrete volume ratio |
|------------------------|------------------|----------------|----------------|-----------------|----------------------|-----------------|----------------------|
| Type I                 | 1.8              | 75             | 68             | 12978           | 1.00                 | 8744            | 1.00                 |
| Type II                | 2.0              | 68             | 65             | 13886           | 0.98                 | 8802            | 1.01                 |
| Type III               | 2.5              | 53             | 62             | 16130           | 1.14                 | 10668           | 1.22                 |

According to the above comparison results, the amount of foundation concrete of type 1 and type 2 is less than that of type 3. The number of pile foundation of type 2 is less than that of type 1, so it has more advantages in construction organization. Based on comprehensive consideration, bored pile with diameter of 2.0m is adopted for the foundation under the cable tower.

3.3. Consideration of stiffness of pile cap

Code for design of subgrade and foundation of highway bridges and culverts (JTGD63-2007) provides the calculation and design method of pile foundation [6]. In the calculation method of pile top reaction of single pile with multi row piles, the absolute rigidity of pile cap is assumed, the deformation of soil is considered, and the solution is carried out according to the displacement method of structural mechanics. The cable tower foundation of the bridge bears large external force, the plane size of the...
bearing platform is large and the force transmission is complex, so the influence of its own deformation on the pile top reaction should be considered according to the actual stiffness of the bearing platform.

Based on the above point of view, the finite element analysis software Midas-FEA is used to establish the bearing platform entity finite element model (figure 6) for the foundation scheme of 2.0m pile diameter (figure 4), and the comparison with the code algorithm is carried out. In the finite element model, the node elements are established at the tower base and pier bottom, which are rigidly connected with the bearing platform. The load is applied to the bearing platform through these node elements. The axial force transmitted by the tower column is transmitted to the tower base according to the surface load, and the shear force and bending moment are transmitted to the top centroid of the tower base according to the concentrated force (figure 7). The horizontal resistance of the side soil is considered in the pile foundation of the pile cap. Each pile foundation is simulated by establishing a spring element at its corresponding position. The spring stiffness is calculated according to the "m" method combined with geological exploration data by using empirical formula.

![Figure 6. FEA solid finite element model of pile cap foundation](image)

![Figure 7. loading mode in finite element model](image)

The short-term effect combination is adopted for the action effect transmitted to the bottom of pile cap. The results of the two calculation methods are shown in the table 2 below:

| Computing method          | Single pile top reaction $F_z$ (kN) | Vertical displacement of pile cap $D_z$ (mm) |
|---------------------------|------------------------------------|--------------------------------------------|
|                           | Max      | Min      | Max      | Min      |
| Finite element method     | 15621    | 5356     | -7.942   | -2.129   |
| Canonical algorithm       | 12661    | 9397     | -5.443   | -3.827   |

Note: negative value of $D_z$ indicates downward displacement

The maximum value of $F_z$ and $D_z$ ($D_z$ is the absolute value) obtained by the canonical algorithm is less than the maximum value of finite element analysis, while the minimum value of $F_z$ and $D_z$ ($D_z$ is the absolute value) is greater than the minimum value of finite element analysis. The reasons are briefly analyzed as follows: ① in the code algorithm, it is assumed that the bearing platform is absolutely rigid, and the deformation coordination of the bearing platform itself is not considered, so that the pile foundation far away from the tower base shares more effect transmitted to the bottom of the bearing platform; ② the cable tower of the bridge adopts arc shaped tower column, and the transverse bending moment transmitted to the top of the bearing platform by the two limb tower
column under static condition is large, and the action direction is opposite. As the abutment is regarded as a rigid body, the transverse bending moment effect can not be transferred to the bottom of the pile cap, resulting in the calculated pile top reaction force is small. The cable-stayed bridge of this project has large distance from the root of arc-shaped double leg tower, large bearing platform size, large number of pile groups, and large transverse effect under single leg tower. The calculation results show that the maximum pile top reaction value obtained by standard algorithm is small, which is unsafe in checking the length of pile foundation.

3.4. comparison of pile cap shapes

The cable tower foundation needs to bear the load transmitted by the tower column and the pier column. Different pile foundation layout forms adopt different bearing platform dimensions, which will also affect the distribution of pile top reaction. The rectangular pile cap (figure 4) and Dumbbell Pile Cap (figure 8) are compared by using pile foundation with diameter of 2.0m. The solid finite element model of dumbbell shaped pile cap is established by Midas FEA (figure 9).

| Table 3 summary of comparison results of rectangular cap and dumbbell cap |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Pile foundation layout          | Pile diameter  | Number of roots | $N_{\text{max}}$ (kN) | pile foundation | Pile cap | Pile foundation | Pile cap | Pile foundation |
| rectangle                       | 2.0           | 68              | 15621              | 69              | 14740    | 1.00            | 9362    | 1.00           |
| Dumbbell shape                  | 2.0           | 62              | 14836              | 67              | 13050    | 0.89            | 8146    | 0.87           |

Note: $N_{\text{max}}$ is the maximum single pile top reaction
According to the above comparison and selection results, under the condition of meeting the requirements of the bearing capacity of the superstructure, the amount of foundation concrete (pile foundation + pile cap) of the dumbbell shaped pile cap is 13% less than that of the rectangular pile cap, the size of the dumbbell shaped pile cap is smaller, the number of pile foundations and the calculated pile length are reduced in varying degrees, which is more advantageous in the construction organization.

3.5. influence of pile foundation arrangement on pile foundation stress

The dumbbell shaped pile cap is adopted, and the arrangement forms of pile foundation are compared and selected by combining with the finite element analysis.

By looking at the vertical displacement results of dumbbell shaped cap before optimization (figure 10), it can be seen that the $D_Z$ value of cap changes greatly along the bridge direction, while the $D_Z$ value of the middle connecting section of cap is smaller. According to the relationship between force and deformation, it can be judged that the reaction force of pile top along the bridge direction is quite different, and the reaction force of pile top at the middle connecting section of pile cap is relatively small, which can not give full play to the bearing capacity. According to the results of pile top reaction (figure 11, only the pile top reaction at the representative pile position) which is consistent with the above judgment, the maximum single pile top reaction ($N_{\text{max}}$) is 14836 kN, which appears at the edge of the pile cap along the bridge, and the minimum single pile top reaction ($N_{\text{min}}$) is 7708 kN, which appears at the middle connecting section of the pile cap. The reason is that the bending moment along the bridge direction from the pylon and pier column to the bearing platform is larger, which is the main stress direction of the foundation; the pile foundation at the middle connecting section bears less effect on the transmission of the pylon column.

The original dumbbell shaped cap foundation scheme is optimized and adjusted, so that the maximum single pile top reaction is reduced, the pile group pile top reaction distribution is more uniform, and the bearing capacity of each pile can play a role as much as possible. By increasing the number of pile rows along the bridge and the size of pile cap along the bridge, the resistance of the foundation to the bending moment in the main stress direction can be improved; by reducing the...
number of pile rows across the bridge and the size of pile cap across the bridge, the pile foundation in the middle connecting section can bear more effects. After adjustment, the number of pile foundations is still 62, and the bearing platform volume is basically the same as the original scheme (figure 12).

Figure 12. optimization scheme of dumbbell shaped cap foundation

Midas-FEA was used to establish a finite element model to analyze the optimization scheme (figure 13). Check the displacement nephogram of the optimization scheme (figure 14) and compare it with the original scheme. The vertical deformation of the pile cap of the optimization scheme is more uniform, and the $D_z$ extreme value changes from $-6.696mm \sim -3.229mm$ to $-5.091mm \sim -2.093mm$ (negative value means displacement downward). Looking at the pile top reaction results of the optimization scheme (figure 15, only the pile top reaction at the representative pile position is shown), $N_{\text{max}}$ is 11435 kN, which appears at the edge of the inner corner of the pile cap along the bridge, $N_{\text{min}}$ is 5594 kN, which appears at the edge of the outer corner of the transverse bridge of the pile cap, while the single pile top reaction at the middle connecting section of the pile cap is increased, which is 10453 kN. Compared with the original scheme, the maximum single pile top reaction of the optimized scheme is smaller, the difference of pile group top reaction is smaller, the effect transmitted to the cap is more evenly distributed among the piles, and the pile foundation in the middle connecting section of the cap bears more effect.
Figure 14. vertical displacement nephogram $D_Z$ (mm) of dumbbell shaped cap after optimization

Figure 15. vertical reaction $F_Z$ (kN) of dumbbell shaped pile top after optimization

The following table 4 lists the comparison of rectangular, dumbbell shaped and optimized dumbbell shaped pile cap foundation layout schemes under short-term effect combination.

| Pile foundation layout | Number of roots | $N_{\text{max}}$ (kN) | $N_{\text{min}}$ (kN) | Pile length (m) | Concrete volume ($\text{m}^3$) | Concrete volume ratio | Concrete volume ($\text{m}^3$) | Concrete volume ratio |
|------------------------|-----------------|------------------------|------------------------|-----------------|-----------------------------|------------------------|-----------------------------|------------------------|
| rectangle              | 68              | 15622                  | 5356                   | 68              | 14740                       | 1.00                   | 9362                        | 1.00                   |
| Dumbbell shape         | 62              | 14836                  | 7708                   | 65              | 13050                       | 0.89                   | 8146                        | 0.87                   |
| After optimization     | 62              | 11435                  | 5594                   | 62              | 12271                       | 0.83                   | 8501                        | 0.91                   |

Note: $N_{\text{max}}/N_{\text{min}}$ is the maximum / minimum single pile top reaction

It can be seen from the above table that the amount of foundation concrete (pile foundation + pile cap) of the two schemes of dumbbell cap is less than that of rectangular cap, and the optimization scheme of dumbbell cap has the best economy. At the same time, the length of pile foundation of the optimized scheme is smaller, and the advantage in construction organization is more obvious. The final design scheme of cable tower foundation adopts dumbbell shaped cap optimization scheme (Fig. 12).

3.6. Stress analysis of pile foundation under earthquake action

According to the requirements of relevant codes\cite{7,8}, the seismic checking calculation of cable tower
foundation under the combination of earthquake accidental action (E1, E2) is carried out. The permanent action extracted from the analysis results of the overall calculation model of the cable-stayed bridge and the seismic action extracted from the seismic analysis model are combined (considering the two directions along the bridge x and across the bridge y respectively). The E1 and E2 seismic action combinations are loaded into the finite element model of the cable tower foundation (figure 13) respectively, and the spring stiffness of the simulated pile foundation is adjusted (m dynamic is taken as 2 times of m static). The pile top reaction of the single pile is calculated, and the vertical bearing capacity and bending bearing capacity of the single pile are checked. The seismic checking results of the vertical bearing capacity under the combination of E1 and E2 earthquakes are shown in the table 5 and 6 below.

Table 5  Seismic action of E1 and E2 along the bridge and vertical bearing capacity of single pile

| working condition | Accidental action combination of earthquake along bridge | Design pile length (m) | Maximum pile top reaction (kN) | Allowable bearing capacity of single pile (kN) |
|-------------------|---------------------------------------------------------|-------------------------|--------------------------------|-----------------------------------------------|
| E1                | vertical force (kN) 592254 bending moment (kN·m) 919119 Horizontal force (kN) 68279 | 74                      | 13796                          | 16343                                         |
| E2                | vertical force (kN) 633840 bending moment (kN·m) 205691 Horizontal force (kN) 129057 | 74                      | 19433                          | 19612                                         |

Table 6  Lateral seismic action of E1 and E2 bridges and vertical bearing capacity of single pile

| working condition | Accidental action combination of transverse bridge earthquake | Design pile length (m) | Maximum pile top reaction (kN) | Allowable bearing capacity of single pile (kN) |
|-------------------|---------------------------------------------------------------|-------------------------|--------------------------------|-----------------------------------------------|
| E1                | vertical force (kN) 597703 bending moment (kN·m) 3434663 Horizontal force (kN) 116512 | 74                      | 13098                          | 16343                                         |
| E2                | vertical force (kN) 612786 bending moment (kN·m) 7787100 Horizontal force (kN) 147856 | 74                      | 17331                          | 19612                                         |

Note: ① the design pile length in Table considers the influence of river erosion and seismic liquefaction soil layer; the allowable bearing capacity of single pile; ② under E2 condition in Table considers the adjustment coefficient of foundation seismic allowable bearing capacity in detailed rules for seismic design of highway bridges (JTG/T B02-01-2008).

The results of seismic checking calculation of pile bending bearing capacity under E1 and E2 earthquake conditions are shown in the table 7 and 8 below.

Table 7 checking calculation of bending bearing capacity of single pile under earthquake action of E1 and E2 along bridge direction

| working condition | Checking calculation of bending moment (kN·m) | Calculation of reinforcement (mm²) | Actual reinforcement (mm²) | Actual reinforcement ratio (%) |
|-------------------|----------------------------------------------|-----------------------------------|----------------------------|-------------------------------|
| E1                | 3177                                         | 15708                             | 32160                      | 1.02                          |
| E2                | 5893                                         | 20358                             | 32160                      | 1.02                          |

Table 8 checking calculation of bending bearing capacity of single pile under seismic action of E1 and E2 transverse bridge

| working condition | Checking calculation of bending moment (kN·m) | Calculation of reinforcement (mm²) | Actual reinforcement (mm²) | Actual reinforcement ratio (%) |
|-------------------|----------------------------------------------|-----------------------------------|----------------------------|-------------------------------|
| E1                | 6755                                         | 15708                             | 32160                      | 1.02                          |
| E2                | 8553                                         | 28264                             | 32160                      | 1.02                          |
From the above results, it can be seen that the mechanical performance of the cable tower pile foundation under the combination of E1 and E2 earthquake action does not exceed the allowable bearing capacity and bending bearing capacity of the pile.

4. Conclusion
The cable tower of the main bridge of Tarim bridge in Xinjiang adopts the arc-shaped concrete tower column with variable cross-section, which is rare at present. The external effect transmitted to the cable tower foundation is large and the force transmission is complex. In the process of calculation and analysis of cable tower foundation, a three-dimensional solid finite element model is established and compared with the conventional analysis method recommended by the code.

(1) Using large diameter bored pile group foundation, through reasonable pile diameter selection and pile foundation layout, can well meet the stress demand of foundation structure, and achieve better economy and construction convenience.

(2) In large-scale pile group foundation, the stiffness deformation of pile cap has a significant impact on the distribution of pile top reaction of each pile foundation. The pile top reaction calculated by the standard algorithm is smaller, and the result obtained by the finite element method is closer to the real situation, which is safer for the bridge structure.

(3) For the single pylon cable-stayed bridge, the size and engineering quantity of dumbbell shaped cap are smaller than that of rectangular cap, the number of pile foundation and the calculated pile length are reduced in varying degrees, and the section form is more reasonable.

(4) Using the finite element technology can more accurately analyze the mechanical characteristics of large cap pile group foundation, and provide more reliable analysis results for the bridge foundation. The calculation and analysis of the cable tower foundation of Tarim bridge has accumulated valuable experience and exploration for similar projects, and can provide reference for other similar projects.

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