Study on Calculation Method of Internal Force of Integral Abutment-Pile-Soil Interaction

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Abstract. Integral abutment jointless bridge (iajb) has the advantages of long service life, convenient construction and low construction and maintenance cost. At present, it has been widely used at home and abroad. Based on an actual iajb, an experimental model of integral abutment pile structure is designed and made. The quasi-static test is carried out under low cyclic displacement load, and the interaction between integral abutment, H-steel pile and soil is studied, with emphasis on the strain and bending moment of abutment and pile foundation. The test results show that the strain distribution of pile body is "Cup" shape when the abutment moves forward and "olive" shape when the abutment moves negatively. The maximum compressive stress and tensile stress under positive displacement load are greater than those under negative displacement load. Therefore, when the temperature increases, the internal force of pile foundation is greater than that when the temperature decreases, which means that H-shaped steel foundation pile is more unfavorable when the temperature increases in summer. In order to reduce the adverse effect of temperature on foundation piles, it is suggested that the overall closure temperature of the bridge should be slightly higher than the annual average temperature. In addition, the calculation also shows that when negative load is adopted, the pile bending moment calculated by these methods is not different from the test results, and the distribution law is similar to that of traditional foundation piles. However, under normal load, the pile bending moment calculated by classical theory or bridge code is quite different from the test results, and the distribution law is also different. In this paper, the moment of integral abutment pile-soil interaction is calculated accurately by polynomial fitting method and huanglin method, which can be used in practical engineering and provide reference for the design and application of iajbs.

Keywords. Bridge engineering; Integral abutment jointless bridge; H-shaped steel pile; Abutment-pile-soil interaction; Pile strain; Pile bending moment

1. Introduction

Integral Abutment Jointless bridge (IAJB) connects the girder, abutment and pile foundation as a whole. The expansion and contraction deformations of girder under earthquake action or seasonal temperature variation will induce longitudinal horizontal cyclic deformations of the abutment and pile foundation. In order to adapt this cyclic deformations, flexible pile is recommended to adopt for foundation pile [1]. H-shaped steel piles have favorable flexibility, strong deformation capacity and excellent seismic performance [2, 3], which have been widely used as the pile foundation in IAJBs. Some researchers have studied the performance of H-shaped steel pile under horizontal load. Guo et al. [4] conducted experimental research on four H-shaped steel piles and found that the response of pile-soil system with small displacement was...
mainly controlled by the nonlinear characteristics of soil mass. Zhao et al. [5] believed that bending moment of H-shaped steel pile around weak axis can reduce the bending moment of pile body and increase the deformation ability of structure. Arsoy et al. [6] found that the H-shaped steel pile could well adapt to the deformation induced by girder under temperature load, which showed that the H-shaped steel pile was the best pile type to support the integral abutment. Burdette et al. [7] conducted a test of H-shaped steel pile with abutment, and found that the maximum displacement of H-shaped steel pile at the ground surface can exceed 25.4mm. Zhao et al. [8] conducted a quasi-static test of integral abutment- H-shaped steel pile structure. The test results showed that the energy dissipation capacity and bearing capacity of H-shaped steel pile bent in strong axis were greater than that bent in weak axis, but the ductility was smaller. However, these studies did not consider the impact of abutments and backfill, and further research is needed.

Huang et al. [9, 10] conducted the pseudo-static test research on the abutment- H-shaped pile-soil interaction firstly. The research showed that the earth pressure behind the abutment is mainly distributed in either 'triangular' or 'trapezoidal' shapes along the depth direction, and the earth pressure behind the abutment has an obvious cumulative effect. The methods given in JTG D60-2015 [11] and several existing research theories [12-18] are not suitable for the calculation of earth pressure behind the abutment of IAJBs. Meanwhile, the abutment and the backfill behind the abutment will have a significant impact on the deformation and internal force of the H-shaped steel pile foundation, causing the cumulative deformation effect. On this basis, references [9, 10] put forward a calculation method considering the cumulative deformation effect of earth pressure behind the abutment, which is called Huang-Lin method in this paper.

However, references [9, 10] did not analyze the distribution patterns of pile strain in the test model of integral abutment -H-shaped steel pile -soil interaction, and also did not study the calculation method of internal force of integral abutment-pile foundation. Therefore, on the basis of references [9, 10], the strain distribution pattern of the H-shaped pile and the internal force calculation method of integral abutment-pile foundation-soil interaction are studied. This paper can provide a reference for the formulation of relevant specifications.

2. Experimental program

2.1 Specimen design

The detail dimensions of specimen in reference [9] is shown in Table 1. The total length of the H-shaped steel pile is 3.21 m, of which 0.31 m (2B) embeds into the abutment to meet the consolidation requirements of the abutment and the H-shaped steel pile [19]. Thus the actual buried depth of the H-shaped steel pile (calculated pile length) is 2.9 m. The vertical longitudinal reinforcement of abutment adopts Ф12 and Ф 8 HRB335 ribbed reinforcement bars, and the stirrup adopts Ф 6 HRB335 plain reinforcement bars. The yield strength of reinforcement bar is 337 MPa, and its ultimate strength is 454 MPa. H-shaped steel pile was made of Q235 steel with yield strength of 238 MPa and ultimate strength of 424 MPa.

The test soil box is 3.0 m long, 2.0 m wide and 4.0 m high. The soil behind the abutment and the soil around the pile were filled with medium-coarse sand, and there was no filling in front of the abutment. Therefore it was asymmetric soil boundary condition in the abutment. Test sand was dense sand with a relative compactness of 53%, water content of 1.3%, density of 1.50 g.cm\(^{-3}\), porosity ratio of 0.8, cohesion of 0 kPa, internal friction angle of 35°, Poisson's ratio of 0.3[20], and \(m\) value(Proportionality coefficient for the resistance coefficient of the foundation) of 20,000 kN/m\(^4\) [18]. The specific information of this model can be referred to [9].

| Table 1. The Detail Dimensions of Specimen (Unit: mm) |
|-----------------------------------------------------|
| Abutment | H-shaped steel pile |
| Longitudinal length | Transverse length | Height |
| Strong axial | Weak axial | Web thickness | Wing thickness | Total length |

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2.2. Test setup and instrumentation

Sensors such as strain gauges, linear variable differential transducers and earth pressure cells was arranged in the model, and the layout details are shown in Figure 1. Figure 1(a) is the layout diagram of strain gauges and Figure 1(b) is the layout of earth pressure cells. As shown in Figure 1(a), 18 concrete strain gauges were arranged on the abutment (S1–S18), 60 steel strain gauges were used on the H-shaped steel piles (S19–S78). As shown in Figure 1(b), 18 earth pressure cells were symmetrically arranged on the web of H-shaped steel pile (T1–T18), with odd numbers arranged on the back of the pile and even numbers arranged on the front of the pile. 6 earth pressure cells (T19–T24) were arranged at the back of abutment and pile top. 12 earth pressure cells (T25~T36) was set in the backfill behind abutment.

| Width (W) | Width (B) | T1 | T2 | T3 |
|-----------|-----------|----|----|----|
| 560       | 660       | 1000 | 217 | 155 | 6   | 10  | 3210 |

![Layout of strain gauges](image1.png) ![Layout of earth pressure cells](image2.png)

**Figure 1.** Detailed layout of test points (Unit: mm)

2.3 Loading protocol

In the test process, the displacement was taken as the control parameter, and the load was carried out with the displacement increment of 2 mm per stage. The displacement was calculated by the temperature effect, and the specific formula is \( \Delta l = \alpha l (T - T_0) \). The loading frequency is 1 Hz, 3 load cycles per stage, and the test process stopped when the maximum displacement is 16 mm. The loading process is shown in Figure 2(a). The displacement values mentioned below are result of the second displacement loading among the three displacement loading cycles in the test.

This test was load by MTS electro-hydraulic servo loading system, and the displacement loading point was located on the abutment, 0.35 m below the abutment top. According to the axial compression ratio, the vertical force exerted on the top of the abutment is 25.02 kN, in the form of counterweight, as shown in Figure 2(b).
3. Test results and discussions

References [9] and [10] analyzed the deformation, rotation angle and earth pressure of the test model. This paper mainly analyzes the strain and bending moment the test model.

3.1. Strains of pile body

The test showed that the abutment produced rigid body displacement and rotation, without flexural deformation, and the abutment strain value is small. Therefore, only strain results of pile body were analyzed in this paper. In addition, the strain value of pile body in this paper is the average value of two strain gauges located at the ends of left and right flanges on one side.

3.1.1 Strain distribution pattern along the depth

It can be seen from Figure 3(a) that under positive loading, the front of the pile was stretched while the back was compressed. The tensile and compressive strains on the front and back sides were in generally symmetrical distribution. The strain distribution pattern of pile body along the pile depth was a 'Cup'-shaped distribution, which is different from the traditional 'Olive'-shaped strain distribution pattern. Due to the influence of unbalanced earth pressure behind the abutment, the bending moment on pile top caused by positive loading was much larger than negative loading, and then induced a larger strain value at pile top in positive direction. The pile generated a large cumulative deformation at the shallow buried depth, made the strain value of the pile in this area increase sharply, showing a 'Cup'-shaped distribution in a radial state.
As shown in Figure 3(b), with the abutment moved toward negative direction, the front of the pile was compressed and the back was stretched. Except for individual sections, the tensile and compressive strains on the front and back of the pile were in generally symmetrical distribution. Both tensile strains and compressive strains were increased at shallow buried depth, then decreased with the increase of buried depth, showing an 'Olive'-shaped distribution. That was basically consistent with the strain distribution of traditional horizontally loaded piles. The maximum tensile strain occurred at the buried depth of 0.8 m (5.16B), while the maximum compressive strain occurred at the buried depth of 1.0 m (6.45B).

Comparing with Figure 3(a) and Figure 3(b), tensile strains and compressive strains are different under different loading direction. The former is a 'Cup'-shaped distribution, and the latter is a typical 'Olive'-shaped distribution. The maximum tensile strain and the maximum compressive strain under positive loading were obviously greater than that under negative loading. Therefore, the internal force of the pile under temperature raising is larger than that under temperature dropping. It indicates that the H-shaped steel foundation pile is the most adverse when high temperature in summer. In order to reduce the adverse effect of temperature raising, the overall bridge closure temperature should be slightly higher than the annual average temperature.
3.1.2 Strain distribution pattern of the cross-section

Figure 4 shows the strain distribution of the cross-section of the pile foundation under the positive displacement. Due to space limitation, only the sections with a buried depth of 0.6m (measured points S41, S42, S43, S44) are given. It can be seen from Figure 4 that the tensile and compressive strains of the section are basically symmetrical, which conforms to the plane cross-section assumption. And the neutral axis is also generally consistent with the geometric central axis of the section. Therefore, the H-shaped steel pile foundation of the IAJB is still in an elastic state under the action of temperature, which is consistent with the linear change of the load-displacement skeleton curve described in reference [9].

![Figure 4. Distribution of weak axial cross-section strain of pile foundation at depth of 0.6m](image)

3.2 Strain change course of pile body

3.2.1 Strain change course of pile body along depth

Figure 5 shows the tensile and compressive strain change course of the pile body when the abutment moved toward positive direction. Only strain change course at the buried depths of 1.0m, 1.8m, 2.0m and 2.4m was given. The strain change course of other buried depths was basically similar. In addition, for the strain change course under negative displacement is not given, for it was basically similar to the course in the positive direction. It can be seen from Figure 5 that the tensile and compressive strains basically increase linearly with the increase of the loading displacement, so the test model is in the linear elastic stage. However, the maximum tensile strain under the displacement load of 16 mm is close to 1000 µε, which is closer to the elastic ultimate strain of steel.

3.2.2 Strain change course of pile body along cross section

Figure 6 shows the strain change course at the buried depth of 0.4m (measured points S45, S47) and 0.6m (measured points S41, S43). Figure 6(a) is the cross section at the buried depth of 0.4m, and Figure 6(b) is the cross section at the buried depth of 0.6m. The tension and compression strains on the front and back (at one cross-section) of the pile are basically symmetrical, as shown by the blue dashed lines in Figure 6(a) & (b). In detail, the strain change course of a single measured point (such measured point S41) has a certain degree of asymmetry, which is caused by the cumulative deformation of the pile body [9].
Figure 5. Tensile strain and compressive strain versus displacement load.
3.3. Bending moment of pile body

The pile body was regarded as an elastic foundation beam. And based on the strain of the pile body, the bending moment $M(z)$ can be expressed as:

$$M(z) = EI \cdot \phi(z) = \frac{EI}{B} (e_t(z) - e_c(z)) \quad (1)$$

Where: $E$ — Elastic modulus of pile body (kN/m²); $I$ — Cross sectional moment of inertia of pile foundation (kN/m⁴); $\phi(z)$ — Curvature at buried depth of $z$; $e_t(z)$ — Measured tensile strain at buried depth of $z$; $e_c(z)$ — Measured compressive strain at buried depth of $z$; $B$ — Weak axial width of H-shaped steel pile (m).

Figure 7 shows distribution of bending moment along the buried depth of the pile. Figure 7 (a) is the bending moment of the pile body under the load in the positive direction, and Figure 7 (b) is that under negative direction. As shown in Figure 7(a), the bending moment of the pile body gradually decreased along the depth of the pile, and the maximum bending moment occurred at the top of the pile. Under displacement load of +16mm, the maximum bending moment was 12.87 kN•m. As shown in Figure 7(b), the bending moment of the pile body first increased and then decreased along the depth of the pile. The maximum bending moment occurred at the buried depth of 1.0m (6.45B). Under displacement load of -16 mm, the maximum bending moment was 6.28 kN•m.

Comparing Figure 7(a) with Figure 7(b), the bending moment distribution under positive loading of the abutment was differently than that under positive loading, and the maximum bending moment was also quite different. The former was about 2.05 times larger than the latter, so pile foundation under the action of temperature raising is more unfavorable than under the action of temperature dropping.
4. Internal force calculation

4.1 Internal force of abutment

Reference [10] used a series of classic theories to calculate the earth pressure behind abutment, such as, JTG D60-2015 [11], NCHRP curve [12], Rankine [13], Coulomb [13], Burk-Chen [14, 15], Massachusetts [16], England [17], Dicleli [18], Barker [12]. Then it proposed an improved calculation method for the earth pressure behind abutment (Huang-Lin method) based on the experimental values. The calculation method of bending moment and shear force of abutment and pile top was given, as shown in Figure 8. The calculation formula of bending moment and shear balance system is as follows:

\[ F_g + F_a + F_p = 0 \]  \hspace{1cm} (2)
\[ M_g + M_a + M_p = 0 \]  \hspace{1cm} (3)

Where:  
- \( F_g \) — MTS loading force, which specifies that the propulsive force is positive (toward the bank slope, same as section II) and the pulling force is negative (toward the river span, same as section II);  
- \( F_p \) — Shear at bottom of abutment, the direction shown in Figure 8 is positive; other symbols are the same as before;  
- \( F_a \) — Resultant of the earth pressure behind the abutment;  
- \( h_i \) — Action position of the resultant;  
- \( M_g \) — Bending moment induced by MTS;  
- \( M_0 \) — Bending moment at the bottom of abutment induced by the earth pressure resultant at the back of abutment.

**Figure 7.** Bending moment distributed of the pile body under positive and negative directions
The bending moment test results at the bottom of the abutment, calculated value of classical theories mentioned above and Huang-Lin method are shown in Figure 9. Each theoretical calculation value increases linearly with the increase of displacement load, but there are great differences in the calculation results of various theoretical methods. And the variation of the test value is also different from the theoretical value.
However, it can be seen from Figure 9 that the bending moment at the bottom of abutment can be obtained more accurately by using the Huang-Lin method. In addition, when the loading displacement was not more than 8 mm, only the theoretical calculation value and variation pattern of NCHRP theory are in good agreement with the experimental results. When the loading displacement exceeded 8 mm, the calculated values and variation pattern of each classical theory, including NCHRP method, are quite different from the experimental results. The reason is that when the loading displacement exceeded 8 mm, cumulative deformation of the pile body began to appear [9].

4.2 Bending moment calculation of pile body

For space limitation, the shear force calculation of pile body is not discussed.

4.2.1 Polynomial fitting calculation method of internal force

The bending moment $M_z$ of the pile can be expressed by the function $M(z)$:

$$M_z = M(z)$$

The function $M(z)$ can be expressed as formula (5) after expansion with Taylor series:

$$M(z) = \sum_{i=0}^{N} A_i(x)z^i$$

Where, $z$ — Buried depth of pile body; \(x\) — loading displacement, not exceeding 20 mm, $N$ — Degree of maximum powder; $i$ — Degree of polynomial; $A_i(x)$ — parameter related to horizontal displacement, which can be obtained by fitting the test results of pile bending moment based on strain in Section II B.

Usually, under this circumstance of $N=6$, the calculation accuracy of physical quantities such as earth pressure, deformation and bending moment in pile-soil interaction can be met. In this paper, $N$ is taken as 4. The parameters after fitting are calculated as shown in Formula (6):

$$A_i(x) = a_i x + b_i$$

Where, $A_i x$ and $B_i$ are coefficients. When loading toward positive direction, the values can be ascertained from Table 2. When loading toward negative direction, the values can be ascertained from Table 3.

Table 2. Relationship between Coefficients $a_i$, $b_i$ and the Number of Polynomials under positive loading

| $i$ | 0   | 1   | 2   | 3   | 4   |
|-----|-----|-----|-----|-----|-----|
| $a$ | -0.86 | -0.73 | -1.64 | -1.8 | -0.72 |
| $b$ | 1.32 | 0.88 | 2.07 | 2.00 | 0.88 |
Table 3. Relationship between Coefficients \( a, b \) and the Number of Polynomials under negative loading

\[
\begin{array}{cccccc}
\hline
i & 0 & 1 & 2 & 3 & 4 \\
\hline
a & 0.13 & 0.5 & 0.39 & 0.07 & 0.005 \\
b & 0.58 & 0.13 & -0.12 & -0.04 & -0.01 \\
\hline
\end{array}
\]

Comparison of bending moment distribution calculated by fitting formula and test is shown in Figure 10. As shown, the pile bending moment calculated by the fitting formula is close to the test result, and its correlation coefficient is 98.0%, the average value is 0.999, and the variance is 2%. The method has higher fitting precision, so it can be used to calculate the bending moment of pile body. In addition, it can be seen from Table 3 that under negative loading, the accuracy requirement can be met if \( N = 3 \).

![Figure 10. Comparison of formula (1) and formula (5) under positive loading and negative loading](image)

4.2.2 Theoretical calculation method of pile internal force

The bending moment and shear force at the abutment bottom are the same as bending moment and shear force at the pile top, and also the external force boundary condition of pile top. The bending moment and shear force were calculated by different theoretical methods of earth pressure behind the abutment, so the external force boundary condition of pile top was different under different theoretical methods. Therefore, it will greatly affect the calculation of the internal force of the pile body. In this section, based on the above-mentioned classical theories, Huang-Lin method and the external force obtained from the test, the
internal force of pile body is calculated by the 'm' method, and compared with the polynomial fitting calculation method in Section III.B.1.

JTG D60-2015 gives the method of calculating the bending moment \( M(z) \) and shear force \( Q(z) \) of pile body according to the 'm' method, as shown in the following formulas (7) and (8):

\[
M(z) = A_3 y_0 \alpha^2 EI + B_3 \phi_0 \alpha EI \left[ C_d M + D_3 \frac{F}{\alpha} \right] \tag{7}
\]

\[
Q(z) = y_0 A_4 \alpha^2 EI + \phi_0 B_4 \alpha^2 EI + M \phi_4 + F' D_4 \tag{8}
\]

Where: \( A_3, B_3, C_3 \) and \( D_3 \) are dimensionless coefficients for calculating the bending moment of pile body; \( A_4, B_4, C_4 \) and \( D_4 \) are dimensionless coefficients for calculating the shear force of pile body, which can be obtained by the 'm' method table in JTG D60-2015; \( y_0 \) and \( \phi_0 \) can be calculated by formula (9):

\[
y_0 = F_p \delta_{Q0} + M_p \delta_{QM}
\]

\[
\phi_0 = -F_p \delta_{MM} - M_p \delta_{QM}
\tag{9}
\]

Where: \( F_p, M_p \) — Shear force (kN) and bending moment (kN•m) on pile top, which can be calculated by formulas (2) and (3); \( \delta_{Q0}, \delta_{QM} \) — The horizontal displacement and rotation angle of the pile foundation on the ground when the horizontal force \( F_p = 1 \) acts on the pile top; \( \delta_{MQ}, \delta_{MM} \) — The horizontal displacement and rotation angle of the pile foundation on the ground when the unit moment \( M_p = 1 \) acts on the pile top. For the case where the pile foundation is placed in non-rock foundation, the calculation formula is shown in Formula (10):

\[
\delta_{Q0} = \frac{1}{\alpha^2 EI} \left[ \frac{B, D_1 - B, D_1}{A, B_1 - A, B_1} \right] + k_1 \left( A, B_1 - A, B_2 \right)
\]

\[
\delta_{MQ} = \frac{1}{\alpha^2 EI} \left[ \frac{A, D_1 - A, D_1}{A, B_1 - A, B_2} \right] + k_1 \left( A, B_1 - A, B_2 \right)
\]

\[
\delta_{QM} = \frac{1}{\alpha^2 EI} \left[ \frac{B, C_1 - B, C_1}{A, B_1 - A, B_2} \right] + k_1 \left( A, B_1 - A, B_2 \right)
\]

\[
\delta_{MM} = \frac{1}{\alpha^2 EI} \left[ \frac{A, C_1 - A, C_1}{A, B_1 - A, B_2} \right] + k_1 \left( A, B_1 - A, B_2 \right)
\tag{10}
\]

Where, \( A_2, B_2, C_2, D_2, A_3, B_1, C_3, D_3, A_4, B_4, C_4 \) and \( D_4 \) are coefficients, which can be ascertained from JTG D60-2015.

4.2.3 Bending moment of pile body

Figure 11 and Figure 12 show the bending moment distribution patterns of pile body along the buried depth direction. Due to the limitation of space, only the boundary conditions of external forces on the pile top based on Rankine theory and Huang-Lin method are given in the figure. Among them, Figure 11 (a) & (b) shows the bending moment distribution pattern of pile body under the external force boundary condition of pile top based on Rankine theory under positive and negative loading respectively. Similarly, Figure 12(a) & (b) is based on Huang-Lin method.
As shown in Figure 11(a) that under positive displacement load, the bending moment distribution pattern of pile body along the buried depth direction increased at first and then decreased, and the maximum value appeared at the buried depth of 3.87 times the pile diameter. Compared with the test results of Section III.B.1, it can be seen that the distribution pattern of the two is obviously different. The maximum bending moment obtained from the test was at the top of the pile, while the maximum bending moment calculated based on Rankine theory and 'm' method in this section was at the depth of 3.87 times the pile diameter. At the same time, the maximum bending moment was also quite different, the former was only 12.87 kN•m, while the latter reached 40.48 kN•m. In addition, the maximum bending moment calculated by the method based on JTG D60-2015 is 49.26 kN•m, and its direction was opposite to the test value except for the large value. It can be seen from Figure 11(b) that under negative loading, the bending moment distribution pattern increases at first and then decreases, which is similar to the test results, but the maximum bending moment value is slightly smaller than the test value, the former is -5.5 kN•m and the latter is -6.0 kN•m.

**Figure 11.** Distribution pattern of pile bending moment based on Rankine theory
Figure 12. Distribution pattern of pile bending moment based on Huang-Lin method

It can be seen from Figure 12(a) & (b) that the bending moment distribution pattern of pile body calculated by Huang-Lin method is basically consistent with the test results, and the maximum bending moment value is also close, indicating that Huang-Lin method can calculate the bending moment of pile body more accurately, while other methods have larger errors and even opposite patterns.

Figure 13 shows the distribution of the maximum bending moment of pile body with the load displacement under various theoretical calculation methods.
Figure 13. Maximum bending moment of the pile under various theoretical calculations

As shown in Figure 13, the maximum bending moment of pile body calculated by each theory gradually increases with the increase of loading displacement, but the trend is different. The maximum bending moment calculated by Rankine theory increases positively with the increase of loading displacement; while maximum bending moment calculated by Coulomb theory first decreases positively and then increases negatively with the increase of displacement. The maximum bending moment calculated by other method all increase negatively. It can also be seen from Figure 13 that polynomial fitting method and Huang-Lin method are among them, and have good agreement with the experimental results.

To sum up, there is a big difference between the existing classical theoretical methods and experimental value. The polynomial fitting method and Huang-Lin method proposed in this paper can accurately calculate the internal force of pile body, which can be used to calculate the internal force of H-shaped steel pile foundation of IAJB.

5. Conclusions

In this paper, the pseudo-static test of abutment-pile foundation-soil interaction of IAJB is carried out, the distribution pattern of strain and bending moment of pile body is studied, and the calculation method of internal force of abutment-pile foundation-soil interaction is discussed. The conclusions are as follows:

(1) Under positive displacement load, both the maximum compressive strain and the maximum tensile strain are obviously greater than that under negative load. Therefore, the internal force of pile foundation under temperature raising is greater than that under temperature dropping. The stress of H-shaped steel pile foundation is the most unfavorable when the temperature rises in summer. In order to reduce the adverse effect of temperature rise on pile foundation, it is suggested that the overall bridge closure temperature should be slightly higher than the annual average temperature.

(2) Under negative loading, the distribution pattern of strain and bending moment of IAJB are basically similar to that of the traditional pile foundation, showing an 'Olive'-shaped distribution. However, due to the influence of unbalanced earth pressure behind the abutment, the distribution patterns of pile strain and bending moment are different from traditional pile foundation under positive loading, and the pile strain presents a 'Cup'-shaped distribution.
The calculation shows that the bending moment at the bottom of the abutment calculated by the existing theories and JTG D60-2015 are quite different from the test results, while the Huang-Lin method can obtain more accurately.

The calculation also shows that under negative loading, the bending moment of pile calculated by the existing earth pressure theories or JTG D60-2015 have little deviation from the test results, and the distribution pattern is similar to that of traditional pile foundation. However, under positive loading, the bending moment of pile calculated by the existing earth pressure theories or JTG D60-2015 are quite different from the test results, and their distribution patterns are obviously different. Huang-Lin method proposed in this paper can accurately calculate the bending moment of integral abutment-pile foundation-soil interaction, which can provide reference for the design and application of IAJB in China.

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