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Analysis methods of the bearing capacity of concrete arch bridges

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Abstract: *In order to effectively check the bearing capacity of the concrete arch bridge, taking a bridge with a main span of 180m as the engineering background, aiming at the characteristics that there is a stress difference in the cross-section of the main arch ring but the deformation is coordinated, combined with the finite element software, based on the existing "Design Specification for Masonry Bridge and Culvert of Highway" for checking the ultimate bearing capacity of the main arch ring, the formula for reducing the cross-section bearing capacity of the cable-hoisted concrete arch bridge considering the stress gradient was derived, and the above-derived formula for reducing the cross-section bearing capacity was compared with the current specification. The results show that: the calculation results according to the derived formula for reducing the cross-section bearing capacity are consistent with the values of the current specification in terms of the pattern, and all of them are on the small side, and a reduction coefficient of 0.945 is obtained. It is recommended that when calculating the concrete arch bridge using the cable-hoisted staged arch-forming method according to the current specification, the reduction coefficient of 0.945 should be taken into account.*

Keywords: *Concrete arch bridge; Reduction factor; Cable hoisting; Arch in stages*

1. Introduction

Reinforced concrete arch bridges are favored by bridge engineers at home and abroad due to their advantages such as high stiffness, small temperature deformation, and good economic performance, and they are especially suitable for the mountainous and canyon areas in China. According to the literature research [1-2], in terms of the number of constructions, spans, and construction techniques, the concrete arch bridges in China are all at the world's leading level. Representative concrete arch bridges include the Wanzhou Yangtze River Bridge with a main span of 420m and the Beipanjiang Grand Bridge with a main span of 445m [3], which are the highway bridge and railway bridge with the largest spans in the world respectively.

The bearing capacity problem of concrete arch bridges is a key issue in the theory of arch bridges, and numerous scholars [4-9] have conducted research on the bearing capacity problem. China has formulated the "Design Specification for Masonry Bridge and Culvert of Highway" (JTG D61-2005) [10], which adopts the equivalent beam-column method. The concrete arch ring is equivalent to an eccentrically compressed column of the corresponding length to calculate the bearing capacity of the arch. However, the method for calculating the bearing capacity adopted in the above specification, that is, the equivalent beam-column method, is only applicable to the calculation of the bearing capacity of concrete arch bridges that are formed into an arch at one time. The internal forces of a completed concrete arch bridge are related to the construction method. For a concrete arch bridge whose main arch ring is constructed by the cable-hoisting construction method, the formation of the main arch ring has gone through three construction stages: hoisting the prefabricated arch rib to form an arch, pouring the longitudinal joint concrete, and pouring the roof concrete. Therefore, there is a stress difference in the cross-section of the arch ring after the arch is formed, that is, the cross-section of the arch ring is a composite cross-section with a stress gradient distribution.

In view of this, relying on the actual project, based on the characteristic that the cross-section of the main arch ring shows a stress gradient distribution, combined with the finite element software, taking the deformation coordination of each part of the arch ring cross-section during the subsequent construction of the superstructure on the arch as the premise, this paper deduces the formula for reducing the cross-section bearing capacity of the arch bridge considering the stress gradient, and compares the above-derived formula for reducing the cross-section bearing capacity with the current

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specification, obtains the resistance reduction coefficient of the main arch ring, and modifies the existing specification.

2. Project Overview and Finite Element Model

2.1 General Situation of the Project

The cross-section of the main arch ring is a single-box triple-cell box section, with a width of 7.60m and a height of 3.1m. The thickness of the middle web is 45cm, the thickness of the side webs is 26cm, and the thickness of both the top and bottom slabs is 26cm. The main arch ring is composed of prefabricated arch ribs, cast-in-place longitudinal joints between the ribs, and a cast-in-place top slab. The height of the prefabricated arch rib is 3.2m, the width of the longitudinal joint between the ribs is 0.3m, and the thickness of the cast-in-place top slab is 0.15m. Both the main arch ring and the bridge deck are made of C50 concrete, and the columns above the arch are made of C40 concrete.

After the prefabricated arch ribs of the bridge are hoisted into an arch shape by the cable-hoisting method, the longitudinal wet joints and the top slab are then poured. After the construction of the main arch ring is completed, the superstructure above the arch is constructed. The main arch ring is transversely divided into three arch ribs. After the middle rib is closed, the two side ribs are closed successively. After removing the stay cables, the top slab concrete with a thickness of 15cm is poured to complete the construction of the entire main arch ring.

2.2 Finite Element Model

According to the construction technology, the arch ring is simulated by using a composite cross-section. The arch feet of the main arch ring are fixed, and the fixation in the general support is used for simulation. The arch ring and the superstructure above the arch are rigidly connected. The concrete material is taken as 26KN/m³, the concrete of the longitudinal joint between the ribs is taken as 29 kN/m³, the concrete of the top slab is taken as 19KN/m³, and the secondary dead load such as the bridge deck pavement is taken as 39KN/m³. The finite element model and the simulated construction sequence are shown in Figure 1.

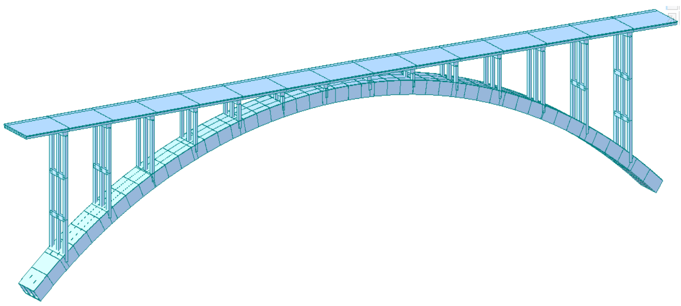

Figure 1 Finite element model of Haima Bridge

The construction process of the bridge is as follows. step.1: The prefabricated arch ribs form an arch. step.2: Pour the longitudinal wet joint. step.3: The longitudinal wet joint forms stiffness. step.4: Pour the top slab concrete. step.5: The roof slab has been constructed and completed. step.6: Construct the columns above the arch during the construction. step.7: Construct the main girder and the second-stage permanent load.

3. The revision of the formula for the bearing capacity of the main arch

3.1 Premises for the revision of the bearing capacity formula

During the process from the formation of the main arch ring to the completion of the bridge, the stress increments at the junctions of the upper edge cross-section and the lower edge cross-section of the cross-section at the arch foot position during the construction process are shown in Figure 2 and Figure 3 respectively. Among them, Construction Process 1 is from the formation of the main arch ring to the construction of the arch upper bent frame, Construction Process 2 is from the construction of the arch upper bent frame to the construction of the bridge deck main girder, and Construction Process 3 is

ISSN 2663-1563 Vol. 7, Issue 1: 122-128, DOI: 10.25236/AJAGE.2025.070116

from the construction of the bridge deck main girder to the completion of the second stage of the bridge deck.

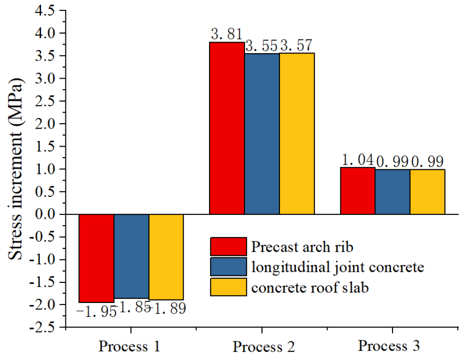

Figure.2 Stress increment of upper edge of arch foot section

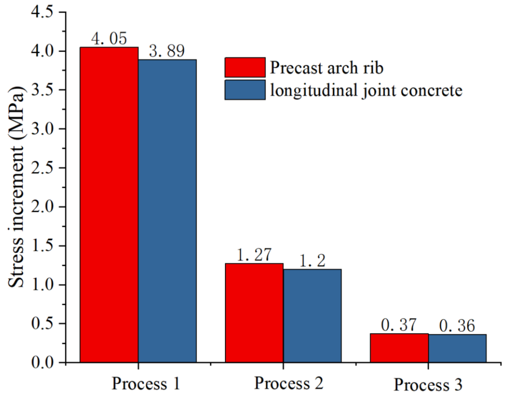

Figure 3 stress increment of lower edge of arch foot section

During the process from the formation of the main arch ring to the completion of the bridge, the stress increments at the junctions of the upper edge cross-section and the lower edge cross-section of the 1/4 cross-section of the arch ring during the construction process are shown in Figure 4 and Figure 5 respectively.

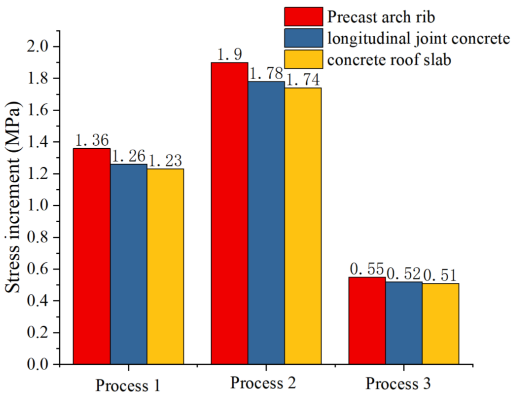

Figure.4 Stress increment of top edge of 1 / 4 arch ring section

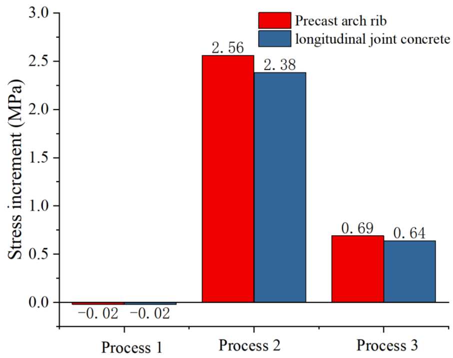

Figure.5 Stress increment of lower edge of 1 / 4 arch ring section

During the process from the formation of the main arch ring to the completion of the bridge, the stress increments at the junctions of the upper edge cross-section and the lower edge cross-section of the arch crown cross-section during the construction process are shown in Figure 6 and Figure 7 respectively.

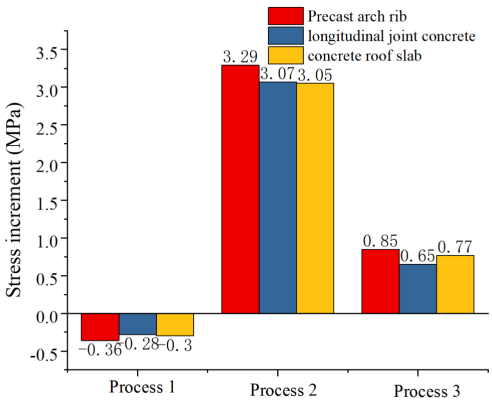

Figure.6 Stress increment of top edge of vault section

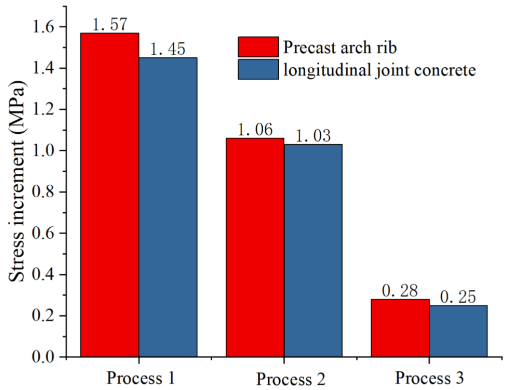

Figure.7 Stress increment of lower edge of vault section

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As can be seen from Figures 2 to 7, after the main arch ring is formed through phased construction, during the subsequent construction processes of the arch upper bent frame, the bridge deck main girder and the second-stage pavement, the stress increments at the junction positions of the upper and lower edges of the prefabricated arch ribs of the main arch ring, the longitudinal joint concrete and the concrete roof slab are basically the same. The percentage of the maximum stress difference is within 5%, which meets the engineering accuracy. According to the above calculation results, it can be considered that there is no relative slip in the collaborative force-bearing during the subsequent construction of the superstructure of the arch for the main arch ring formed in stages by cable hoisting.

The internal force of the completed bridge is related to the construction method. After the prefabricated arch ring forms the arch, the stress generated by its own dead load is σ_{C1}. After the construction of the longitudinal joint concrete between the ribs is completed, it reaches σ_{C2}. After the roof slab concrete is poured, the stress of the prefabricated main arch part is σ_{C3}. At this time, the stress of the concrete between the ribs increases from zero to σ'_{C1}, and the stress of the roof slab concrete is zero. That is to say, the construction method of the main arch ring formed in stages by cable hoisting causes a stress difference in the arch ring section. During the construction of the superstructure of the arch, the stress difference in the main arch ring section always exists. When the main arch ring is loaded in the later stage and reaches the ultimate bearing capacity state, the prefabricated arch rib part will inevitably reach the ultimate bearing capacity state first due to the stress accumulation in the construction process.

3.2 Derive the bearing capacity checking formula according to the cross-sectional reduction method

During the arch-forming process of the main arch ring in stages, under the combined action of its own dead load and the dead load of the concrete in the longitudinal joint and roof section, the total axial force borne by the precast arch rib is defined as N_C, the total bending moment is defined as M_C, and the stress is σ_C. If the cross-sectional area of the precast arch rib is A_1, the moment of inertia is I_1, and the material design strength is f_{c1d}. Then the resistance value of the precast arch rib is:

$$N_{f1} = \varphi A_1 f_{c1d} \tag{1}$$

In the formula: φ is the influence coefficient of the eccentric distance of the axial force in the arch rib on the bearing capacity of the member. f_{c1d} is the design value of the strength of the concrete material of the arch rib section. A_1 is the cross-sectional area of the precast arch rib.

For the sake of safety, it is assumed that all the concrete of the roof is borne by the precast arch rib, and the stress of the concrete in the longitudinal joint starts to increase from zero after the formation of the main arch ring. Then the stress difference between the precast arch rib and the concrete of the longitudinal joint and the roof is σ_C. When the main arch ring reaches the ultimate bearing capacity under the action of external loads, if the ultimate compressive stress of the concrete in the arch rib section is f_{cd}, then the stress difference between the section of the precast arch rib and the concrete of the longitudinal joint and the roof is:

$$\sigma = f_{cd} - \sigma_C \tag{2}$$

If the cross-sectional area of the longitudinal joint of the arch rib is A_2, the cross-sectional area of the roof is A_3, and the design strength of each component section in the combined section is f_{cd}. According to the plane section assumption, in accordance with the principle of equal axial force effect.

The effective resistance value provided by the longitudinal joint to the combined cross-section structure is:

$$N_f = \varphi A_2 (f_{cd} - \sigma_C) \tag{3}$$

The effective resistance value provided by the roof to the combined cross-section structure is:

$$N_f = \varphi A_3 (f_{cd} - \sigma_C) \tag{4}$$

ISSN 2663-1563 Vol. 7, Issue 1: 122-128, DOI: 10.25236/AJAGE.2025.070116

If the total axial force effect that makes the main arch ring reach its ultimate bearing capacity is N, the checking formula for the bearing capacity of the main arch ring obtained from Equations (1) to (4) should be:

$$\gamma_0 N \le \varphi f_{cd}\left[A_1 + \frac{f_{cd}-\sigma_C}{f_{cd}}\left(A_2 + A_3\right)\right] \tag{5}$$

If we let $\lambda = \frac{f_{cd}-\sigma_C}{f_{cd}}$, then Equation (5) can be reduced to:

$$\gamma_0 N \le \varphi f_{cd}\left[A_1 + \lambda\left(A_2 + A_3\right)\right] \tag{6}$$

In the equation: γ_0 is the structural importance coefficient. For the first, second, and third levels of structural design safety grades, it takes values of 1.1, 1.0, and 0.9 respectively.

φ is the influence coefficient of the eccentricity of the axial force in the combined section of the arch ring on the bearing capacity of the compression member. f_{cd} is the design value of the combined section strength. A_1, A_2, and A_3 are the cross-sectional areas of the precast arch rib, longitudinal joint, and roof in the corresponding combined section respectively. λ is the reduction coefficient of the calculated area of the longitudinal joint and roof sections.

The above Equation (6) is the checking formula for the bearing capacity with stress differences existing in the main arch section, which is derived by using the cross-section reduction method for the arch-forming in stages through cable hoisting.

4. Comparison of cross-sectional reduced bearing capacity formula and code

Compare the cross-sectional reduced bearing capacity formula derived above with the current code to obtain the resistance reduction coefficient of the main arch ring constructed in stages by cable hoisting, and amend the existing code.

The material of the main arch ring is C50, with a designed strength of $f_{cd} = 22.4\text{MPa}$ and an elastic modulus of 3.45×107 kN/m^2. A_1 = 6.39 m^2, I_1 = 8.73 m^4. When the cross-section is eccentric upward: y_1=1.74 m. When it is eccentric downward: y_1 = 1.36 m. The cross-sectional area of the longitudinal joint A_2= 1.17 m^2, I_2 = 0.84 m^4. The cross-sectional area of the top plate A_3= 0.78 m^2, I_3=6.5×10^{-4} m^4. After the arch is formed, $A = A_1 + A_2 + A_3 =$ 8.34 m^2, $I =$ 11.80 m^4. According to the calculated length $L_a = 0.36 \times l_0 = 71.17$ m, the cross-sectional height b in the bending plane is 3.2 m. It can be known from the code that φ is 0.59.

According to Code [10], the bearing resistance of the main arch ring is:

$$R = \varphi f_{cd} A = 0.59\times2.24\times104\times8.34 = 1.1\times105 \text{ kN}.$$

The cross-sectional reduction formula derived in this paper for the resistance of the vault cross-section is as follows:

If the cumulative stress of the prefabricated arch rib cross-section before the construction of the spandrel structure is σ_C=4.43 MPa and the concrete stress of the longitudinal joint between the ribs is $\sigma_C^{'}$=0.31MPa, then: $\lambda = \frac{22.4-4.43}{22.4} = 0.80$. Meanwhile, from $A_2\sigma_C^{'} / A_1\sigma_C$=0.017, it is further demonstrated that the method of neglecting the arch-formation stress accumulated in the longitudinal joint cross-section during the derivation process of the above formula is advisable.

It can be known from Equation (7) that the calculated value of the cross-sectional resistance at the vault position is:

ISSN 2663-1563 Vol. 7, Issue 1: 122-128, DOI: 10.25236/AJAGE.2025.070116

$$R=\varphi f_{cd}\left[A_1+\lambda\left(A_2+A_3\right)\right]=0.59\times 2.24\times 10^4\times\left[6.39+0.80\times 1.95\right]$$
$$=1.05\times 10^5\text{kN}$$

The calculation results of the vault cross-section show that: the calculation result according to the revised bearing capacity calculation formula is smaller than that calculated by the current code, and the percentage difference is 4.5%.

For the cross-section at the L/4 position: If the cumulative stress of the prefabricated arch rib cross-section before the construction of the spandrel structure is σ_C=4.38 MPa, then $\lambda=\frac{22.4-4.38}{22.4}=0.81$. According to Equation (7), the calculated value of the cross-sectional resistance at the L/4 position is:

$$R=\varphi f_{cd}\left[A_1+\lambda\left(A_2+A_3\right)\right]=0.59\times 2.24\times 10^4\times\left[6.39+0.81\times 1.95\right]$$
$$=1.05\times 10^5\text{kN}$$

The calculation results at the 1/4 position of the main arch ring show that: the calculation result according to the revised bearing capacity calculation formula is smaller than that calculated by the current code, and the percentage difference is 4.5%.

For the cross-section at the arch foot position: If the cumulative stress of the prefabricated arch rib cross-section before the construction of the spandrel structure is σ_C=5.47 MPa, then $\lambda=\frac{22.4-5.47}{22.4}=0.76$. According to Equation (7), the calculated value of the cross-sectional resistance at the arch foot is:

$$R=\varphi f_{cd}\left[A_1+\lambda\left(A_2+A_3\right)\right]=0.59\times 2.24\times 10^4\times\left[6.39+0.76\times 1.95\right]$$
$$=1.04\times 10^5\text{kN}$$

The calculation results show that: at the arch foot position, the calculation result according to the revised bearing capacity calculation formula is smaller than that calculated by the current code, and the percentage difference is 5.5%.

In summary, for the concrete arch bridges that are formed in stages by cable hoisting, the bearing capacities calculated according to the above-derived cross-sectional reduction formula are all smaller than the values specified in the code. The reduction ratios at the arch foot, the 1/4 position of the arch ring and the vault cross-section are 5.5% and 4.5% respectively. Considering safety, the bearing capacity specified in the current code is revised based on the 5.5% reduction ratio of the bearing capacity at the arch foot position, that is, a reduction coefficient of 0.945 is considered on the basis of the bearing capacity specified in the current code.

5. Conclusion

After the main arch ring is formed in stages, there is a stress difference in the cross-section of the main arch. The cross-section can still work together to bear the load during the subsequent construction process, and there is no relative slippage of the construction cross-section. However, when checking the bearing capacity according to the current code, the reduction of the bearing capacity caused by the cross-sectional stress difference needs to be considered. The calculation examples show that the pattern of the calculation results according to the derived cross-sectional reduced bearing capacity formula is consistent with the values specified in the current code. It is recommended that when calculating the concrete arch bridges formed in stages by cable hoisting using the current code, a reduction coefficient of 0.945 should be considered. This paper can provide a reference for the design and construction control of the concrete arch bridges formed by cable hoisting.

References

[1] Chen Baochun, Ye Lin. Current situation investigation and development direction analysis of

ISSN 2663-1563 Vol. 7, Issue 1: 122-128, DOI: 10.25236/AJAGE.2025.070116

concrete arch bridges in China [J]. Chinese and foreign highways, 2008, 28 (2): 89-96.

[2] Wei Jiangang, Chen Baochun. Application and research progress of long-span concrete arch bridge abroad [J]. World bridge, 2009 (02): 6-10.

[3] Zhang S, Zhao R D. Model Experiment of Construction Process of Long-Span Concrete Arch Bridge Rigid Skeleton[C]// World Symposium on Smart Materials and Applications: WSSMA 2019 : Harbin, China, 5-7 July 2019.2020.

[4] Zheng Zhenfei, Zhang Shangjie, Peng Dawen, et al. Experimental study on ultimate bearing capacity of reinforced concrete arch [J]. Journal of Fuzhou University, 1982 (02): 79-90.

[5] Du Renyuan, Chen Baochun. Experimental study on ultimate bearing capacity of reactive powder concrete arch [J]. Engineering mechanics, 2013 (05): 50-56.

[6] Lin shangshun, Chen Baochun. Calculation of bearing capacity of reinforced concrete arch by equivalent beam column method [J]. Journal of Fuzhou University (Natural Science Edition), 2016, 44 (1): 110-114.

[7] Sun Chao, Chen Baochun, Huang Wenjin. Comparison of calculation methods for bearing capacity of masonry arch bridges [J]. Chinese and foreign highways, 2014, 34 (6).

[8] Wei Jianfeng. Construction process and static analysis calculation of completed state of super long span reinforced concrete arch bridge [D]. Southwest Jiaotong University, 2011.

[9] He Fei, Guo Guangqi, Du bin, et al. Study on parameter value and calculation method of truss composite arch bridge [J]. Sino foreign highway, 2010, 30 (3): 139-143.

[10] Ministry of transport of the people's Republic of China. JTG d61-2005 code for design of highway sewage bridge and culvert [S]. Beijing: People's Communications Press, 2005.