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Optimum design of large span concrete filled steel tubular arch bridge based on static, stability and modal analysis

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Abstract: A three-dimensional finite element model was established for a large span concrete filled steel tubular (CFST) arch bridge which is currently under construction. The arch rib, the spandrel columns, the prestressed concrete box-beam, the cast-in-situ concrete plate of bridge deck, the steel box-beam and the crossbeams connecting the two pieces of arch ribs, were modeled by three-dimensional Timoshenko beam elements (3DTBE). The suspenders were modeled by three-dimensional cable elements (3DCE). Both geometric nonlinearity and prestress effect could be included in each kind of element. At the same time a second finite element model with the same geometric and material properties excepted for the sectional dimension of arch rib was set up. Static dynamic analyses were performed to determine the corresponding characteristics of the structure. The results showed that the arch rib's axial rigidity could be determined by static analysis. The stability and vibration of this system could be separated into in-plane modes, out-of-plane modes and coupled modes. The in-plane stability and dynamic characteristics are determined by the arch rib's vertical stiffness and that of out-of-plane is determined by the crossbeams' stiffness and arch rib's lateral stiffness mainly. The in-plane stiffness is much greater than that of out-of-plane for this kind of bridge . The effect of geometric nonlinearity and prestress effect on bridge behavior is insignificant.

Key words: Concrete filled steel tubular arch bridge, Finite element model, Stability, Modal analysis, Geometric nonlinearity, Prestress effect
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INTRODUCTION

In the beginning of the 1990s, the material of steel tube filled by concrete was introduced to the construction of arch bridges. On one hand, since the compressive strength of this kind of material is much greater than that of concrete and stone material, the dead load of the bridge can be decreased and the span could be increased to a great degree. On the other hand, the steel tubes can be made in the factory and transported to the construction site. The steel tubes are usually assembled consisting of a few segments and erected with cableway to form the arch rib for the first construction stage, for the following construction stages, the arch rib serves as falsework. In this way, the bridge can be built quickly and conveniently. The construction time consumed is much lower compared to that in traditional construction methods. The number of this kind of bridge has increased rapidly in recent years and the span has approached to 400 meters. With lengthening of the span, the stability and dynamic properties become more and more important. The characteristics of the arch bridge have been studied by many researchers with many methods (Chen, 1996; Fan, 1993). However much less attention has been paid to combined static, stability and dynamic analyses of the built-up arch bridge; and the relations between the

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arch rib's in-plane stiffness and the rib's out-of-plane stiffness during the design stage. This paper focused on the development of two finite element models to clarify the issues discussed above. The factors which could affect the behavior of the bridge, such as arch rib's stiffness, constraint condition, prestress effect, geometric nonlinearity, etc were all taken into account in the analysis done by using ANSYS(5.5.1) software.

BRIDGE DESCRIPTION

The Sanmenjiantiao bridge is under construction now and will be completed in August 2001. It was designed by the Zhejiang Communication Design Office. The bridge is located in Sanmen County of Zhejiang Province and spans the Jiantiao Port. It spans 245 meters between two springing points. The traffic lanes have total width of 15 meters and two 1.5-meters wide walkways. It is a typical half through CFST arch bridge with assembled arch rib and concrete bridge deck. The bridge deck under the arch rib is suspended to the bottom chord members by suspenders and the other part supported by the spandrel columns.

Fig. 1 shows the details of one piece of arch rib(ø 800 × 14 denotes that the outer diameter of the steel tube is 800 mm and the thickness of the wall is 14 mm). The axial line is a parabola. The sag-to-span ratio is 1/5. Two pieces of ribs are connected by seven cross-beams, two cross-beams are located under the bridge deck and the other five above the bridge deck. Each piece of arch rib is a spatial truss system composed of chord members, web members and connections between chord members at the same level. The chord member is ø 800 × 14 steel tube filled with concrete. The web members is ø 300 × 10 steel tube. Two chord members at the same level are connected by two pieces of steel plates and the cavum between them is filled with grade 50 concrete. The dimension of the cross section of each piece of arch rib is uniform along the axis. The crossbeam is also spatial truss structure and made up of ø 300 × 10 steel tubes. On the level of the bridge deck the two arch ribs are connected by two steel box beams.

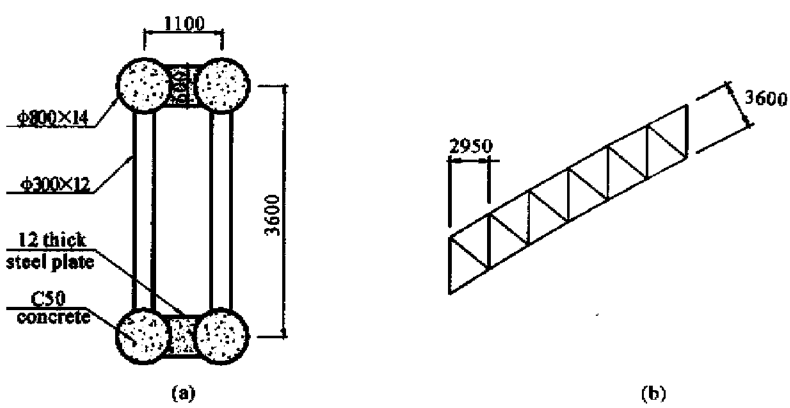

Fig.1 Arch rib (mm)
(a)cross-section; (b)a segment in elevation

Fig.2 shows the details of one segment of bridge deck comprised of two pieces of prestressed concrete box beams and concrete plate cast in-situ with grade 40 concrete between the flanges of two neighboring prestressed beams. Each beam weighs 72.5 tons, the total number is 68. The beam is suspended by two suspenders at each end. Each suspender is comprised of 55 (5 mm diameter) galvanized iron wires. The details of the suspender's anchorage are not shown in Fig.2.

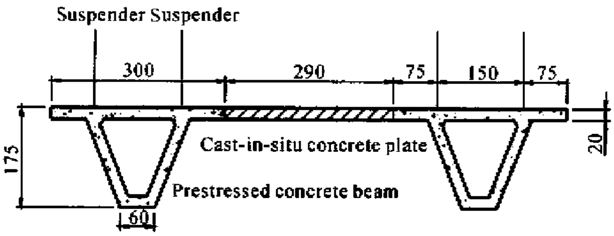

Fig.2 A segment of longitudinal section of bridge deck (cm)

FINITE ELEMENT MODEL(FEM)

The soil-structure interaction was not considered since the two springings of the bridge were built on massive reinforced concrete blocks resting on solid rock. The constraints of two springings were assumed to be fixed in all analysis. The geometric and material properties were calculated from the design drawings. The structure was modeled as a composition of substructures:

FEM of arch rib

Since the minimum distance between two level chord members is only 0.3 meters, the connections between them has enough stiffness to make them behave together as one chord member. Considering this member's sectional size and its longitudinal length, 3DTBE with six degrees of freedom at each end were therefore used to model it (Ding et al., 1985; Yin, 1987; Wang et al., 1997). This determined the model type of the whole arch rib. The vertical and declining web members were represented by the same kind of elements. Thus the spatial truss arch rib was modeled as a plane truss structure in elevation.

We know that the chord member is made up of steel and concrete; how to determine the modulus of elasticity of this assembled material is crucial. Much research has been done on this subject and the corresponding codes have been made in different construction departments(Jiang et al., 1998). But all these codes have limitation of applications. Since the material is so complex that it is almost impossible to make a code to deal with all kinds of loading conditions. Treating the material as reinforced concrete is a very simple method in practice and it is safe.

For 3DTBE, both geometric nonlinearity and shear deflection were taken into account. The 12×12 tangential stiffness matrix could include linear elastic and shear stiffness, nonlinear large displacement stiffness, initial stress stiffness. The number of elements used to model the arch rib was 336.

Cross-beam was represented by spatial truss system of steel tubes with each steel tube modeled by 3DTBE.

FEM of bridge deck

The arch rib is the major structure of an arch bridge. The function of the bridge deck is to transfer the dead load of itself and live load to the arch rib through suspenders, it bears local live load only. If the bridge deck was modeled by shell elements, the element number is so large that the analysis will become very time consuming. On the other hand, since the contribution of stiffness of bridge deck to the whole structure is relatively insignificant, the bridge deck was represented by grid-beams. That is to say the prestressed beam was modeled by 3DTBE. The cast-in-situ concrete plate between two neighboring beams was divided into 13 parts longitudinally. Each part was modeled by 3DTBE. High calculating efficiency could be achieved and errors brought about by this assumption could be neglected.

FEM of suspender and spandrel column

Two-node cable element with six degrees of freedom, i.e., three translational movements in horizontal, vertical, and transverse direction for each node was used to represent the suspender. On each end of the prestressed beam, the two suspenders were modeled by only one cable element, so the element number of arch rib and bridge deck could be reduced markedly. Sixty elements were used to model the suspenders. The spandrel columns were modeled by 3DTBE and eighty elements were used.

Two FEM

In order to investigate the bridge's stability and dynamic characteristics due to the change of the arch rib's stiffness, two FEMs were made. The first one was made according to the real bridge, with the second one being the same as the first one except for the arch rib's sectional dimension (the height of the cross section was reduced from 3.6 meters to 2.0 meters). Fig.3 shows the first FEM incorporating all the above elements.

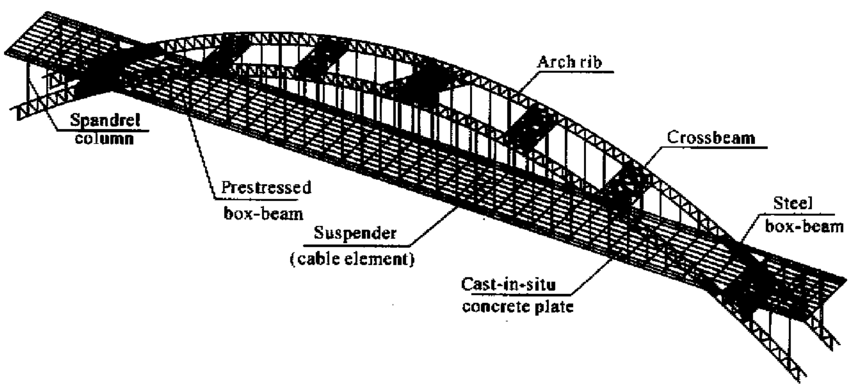

Fig.3 Finite element model

STABILITY ANALYSIS OF THE FIRST MODEL

The stability analysis was aimed to determine the buckling load and buckling modes when the structure begins to lose carrying capacity. In general there are two types of buckling problems. The first one is to determine the load factor of an idealize structure which has no geometric and material errors, small displacement due to buckling load and the relation between strain and stress of material is linear. The buckling load is the upper limit of the structure's carrying capacity. The second one is to determine the structure's carrying capacity when the nonlinear properties of geometry and material errors are considered(Li, 1985; Xiang et al., 1991).

Until now no corresponding bridge code has been established to cope with the CFST arch bridge. The stability problem is usually solved by two methods. The first one is to treat the arch rib as a compressive bar, then determine the load factor of the bar according to the other construction departments' CFSTS codes, such as 《Specification for design and construction of CFSTS》 which is the standard of China engineering technology. The second method is to solve the stability eigenvalue problems by general finite element analytical software(Xu et al., 1993).

The first method corresponds to the traditional Eouler problem. This method is obviously very conservative and it cannot reflect the carrying capacity of a complex system. The second method also has its limitations, because we do not know how the geometric and material nonlinearity affect the stability, so a big load factor is adopted.

Table 1 lists only the first five bucking load factors and modes of the first type buckling analysis and the load factors of the second type buckling analysis of the bridge. In the analysis, material nonlinearity was not considered. The first bucking mode was out-of-plane with the two arch ribs behaving in phase. The first in-plane buckling mode appeared very late, it was the fifth mode and the two arch ribs and bridge deck behaved in-phase. The first in-plane buckling mode contained non-symmetric single waves while the first out-of-plane buckling mode contained symmetric half waves. Fig.4 shows the first in-plane, out-of-plane and torsion modes. The first in-plane buckling load factor was almost twice the first out-of-plane buckling load factor. The difference of the load factor between two load cases was 8.36% for the first out-of-plane mode and 9.85% for the first in-plane mode. When the geometric nonlinearity was taken into account, the difference of load factor between linear and geometric nonlinear analysis was 4.6% for the first out-of-plane mode and 6.6% for the first in-plane mode for the first load case. The corresponding values were 3.4% and 5.3% for the second load case.

Table 1 Buckling load factors of the first model

Mode number	Dead load only			Dead load plus the maximal live load		
	Linear elasticity	Geometric nonlinearity	Mode property	Linear elasticity	Geometric nonlinearity	Mode property
1	6.5578	6.1235	LS1	6.0095	5.4917	LS1
2	7.7069		C	7.0212		C
3	9.4935		C	8.7268		C
4	9.7508		C	8.8005		C
5	10.267	10.0692	VA2	9.2556	8.7643	VA2

Note: V-vertical, L-lateral, C-coupled, A-not symmetric, S-symmetric, 1,2...-number of half wave

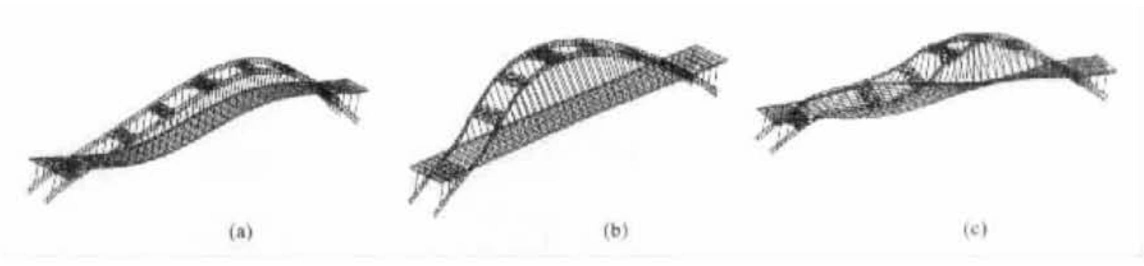

Fig.4 The first buckling mode
(a) in-plane mode; (b)out-of-plane mode; (c)torsion mode

We could conclude that the effect of geometric nonliearity was insignificant; and that the ratio of live-to-dead load was relatively small. The arch rib's in-plane stiffness was much greater than that of out-of-plane. So in the following analysis the geometric nonleari-ty was neglected and the dead load case was considered only so that the calculation time can be reduced to a great extent. Since the first out-of-plane load factor was big enough to keep the bridge safe, we only had to decrease the first in-plane load factor to optimize the design of the arch rib. It is well known that the arch rib is subjected to axial force and is an assembled structure. The axial stiffness is mainly determined by the sectional area of the chord member while the bending stiffness is mainly dependent on the distance between two chord members in addition to the sectional dimension of chord member. In the second model, the height of the arch rib's section decreased from 3.6 meters to 2.0 meters; all other geometric and material properties were unchanged. Lower in-plane load factor, little changes in out-of-plane load factor and in material stress were expected under the above conditions.

STATIC AND STABILITY ANALYSIS OF THE TWO MODELS

Static and stability analyses were done for the two models. Table 2 lists the static analysis results including the maximal stress of chord members, web members and the archcrown deflection. No tensile stress appeared in chord members. Both compressive and tensile stress appeared in the web members, so the maximal absolute value is listed.

Table 2 Static analysis results

Items	Model-1		Model-2	
	Linear elasticity	Geometric nonlinearity	Linear elasticity	Geometric nonlinearity
Chord member(Mpa)	19.968	19.946	20.541	20.479
Web member(Mpa)	94.302	94.315	126.11	126.37
Deflection(m)	0.10469	0.10530	0.10758	0.10848

In each model the difference between all linear and nonlinear analysis results was very small. Results of the two models showed that the stress of chord members increased limitedly; and that although the stress of web members increased rapidly, the absolute value was relatively small compared to the material's allowable stress. The archcrown's deflection was almost unchanged. The static analysis showed that the structure's static behavior mainly depended on the axial stiffness of the arch rib; and that the effect of geometric nonlinearity could be neglected.

Table 3 lists the buckling load factors and the first eight modes of the two models. The system's stability could be separated into in-plane modes, out-of-plane modes and the coupled modes. The first in-plane mode appeared first in mode-2, where the load factor decreased from 10.267 of model-1 to 4.3035. The first out-of-plane mode appeared second, and compared to the load factor of mode-1, the reduction was only 6.59%. Comparison of the load factors of mode-1 with those of mode-2 showed that the corresponding load factors of in-plane decreased rapidly and those of out-of-plane changed little.

Table 3 Buckling load factors and buckling modes

Mode number	Model-1		Model-2	
	Linear elasticity	Mode property	Linear elasticity	Mode property
1	6.5578	LS1	4.3035	VA2
2	7.7069	C	6.1254	LS1
3	9.4935	LS3	7.0979	C
4	9.7508	C	8.9775	LS3
5	10.267	VA2	9.3993	C
6	11.748	LS5	9.9847	VA4
7	19.971	VS3	10.536	VS3
8	21.720	C	11.208	C

Note: V-vertical, L-lateral, C-coupled, A-not symmetric, S-symmetric, 1,2...-number of half wave

MODAL ANALYSIS OF TWO MODELS

The natural frequencies of a structure will, if the frequency spectrum of the dynamic loading is known, indicate whether or not the structure is likely to respond dynamically. The shapes of the modes will indicate in which way the structure is likely to respond and the best positions for placing artificial dampers if required. On the other hand the frequencies and the shapes of modes indicate the stiffness of the structure in different directions(Buchholdt, 1985; Clough et al., 1975).

Since the axial compressive force can decrease the stiffness of a structure, the arch bridge frequencies vary with not only the span but also loading conditions. For this bridge, the maximal live load is only about ten percent of the total dead load. Thus when calculating the prestress effects on the bridge, the dead load was considered only.

The calculated first ten natural frequencies and shapes of modes of vibration are listed in Table 4 for both models with or without considering prestress effects. The vibration of this system could be separated into in-phase modes and out-of-phase modes. In the inplase mode, the arch rib and bridge deck vibrated in-phase. In the out-of-phase mode the arch rib and bridge deck vibrated to opposite direction; or the arch rib vibrated only while bridge deck had no or only small modal motion. The in-phase and out-of-phase modes could be further separated into in-plane modes of vibration, out-of-plane modes of vibration and the pattern of coupled mode between these two directions. The analytical results showed that there were a small number of out-of-phase modes. There were only two out-of-phase modes in the first twenty modes (Fig. 5). The patterns of the first in-plane mode, out-of-plane mode and coupled mode were the same as those of the respective modes in the stability analysis results (Fig. 4). In each model the difference of system vibration frequencies resulting from inclusion or non-inclusion of the prestress-effect was small; and the greater the frequency, the smaller the difference. Comparing the output of model-1 to model-2, the first mode of vibration in model-1 was out-of-plane while that in model-2 was in-plane; The reduction of the first frequency of in-plane was 33.40% and that of out-of-plane was only 5.96%. frequencies of in-plane decreased much but those of out-of-plane decreased a little. The conclusion drawn from the stability analysis was confirmed here.

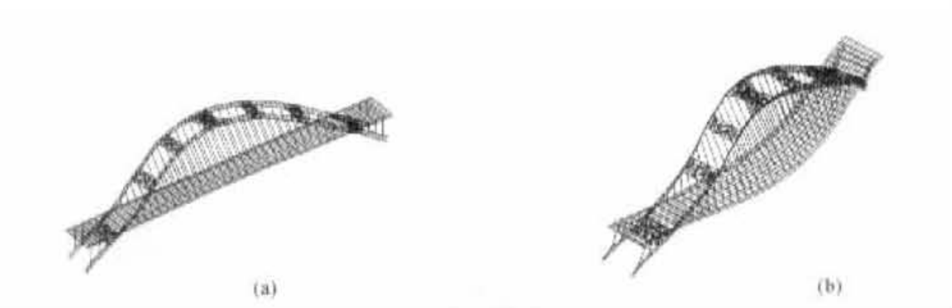

Fig. 5 Out-of-phase modes
(a) the first mode; (b) the second mode

Table 4 Natural frequencies

MN	Model-1				Model-2			
	NP(Hz)	P(Hz)	D(%)	MP	NP(Hz)	P(Hz)	D(%)	MP
1	0.35571	0.33529	5.74	OLS1	0.32312	0.28458	11.93	VA2
2	0.48516	0.46205	4.76	VA2	0.33449	0.31389	6.16	OLS1
3	0.84831	0.79462	6.33	C	0.80314	0.74780	6.89	C
4	0.94088	0.93998	0.09	OLS1	0.87785	0.83710	4.64	VA4
5	1.1764	1.1553	1.79	VS1	0.91401	0.87314	4.47	VS1
6	1.2081	1.1825	2.12	VA4	0.93031	0.92892	0.15	OLS1
7	1.3250	1.2426	6.22	C	1.2366	1.1785	4.70	C
8	1.3930	1.3701	1.64	C	1.2632	1.2088	4.31	C
9	1.6330	1.6242	0.54	C	1.5366	1.5279	0.57	C
10	1.7229	1.7142	0.50	VS3	1.5954	1.5717	1.49	VS3

Note: MN-mode number; NP-not include prestress-effect; P-include prestress-effect; D-difference; MP-mode property; V-vertical; L-lateral; C-coupled; A-not symmetric; S-symmetric; O-arch rib and bridge deck vibrate out-of-phase; 1, 2...-number of half wave

CONCLUSIONS

1. Static analysis can only determine the axial stiffness of the arch rib but not the bending and torsional stiffness.

2. The geometric nonlinearity's effect on the structure's static characteristics is negligible. The effect on the stability and dynamic behavior is also small and can be neglected in practice, but it is much greater than that on the static behavior.

3. The system's stability can be separated into in-plane modes, out-of-plane modes and coupled modes between in-plane and out-of-plane modes. The in-plane buckling load factor mainly depends on the arch rib's vertical bending stiffness while the out-of-plane buckling load factor is determined by the lateral bending stiffness of the arch rib and the stiffness of cross-beams. The contribution of arch rib's torsional stiffness to out-of-plane buckling load factor is secondary.

4. The system's vibration can be separated into in-phase and out-of-phase vibration modes. The in-phase modes and out-of-phase modes can be further separated in the same way as done in the stability analysis. The conclusion drawn from stability analysis is confirmed again.

5. The in-plane stiffness is much greater than out-of-plane stiffness for this kind of bridge in general. The prestress effect is insignificant in stability and modal analysis. The optimum design of arch rib can be achieved by applying the results obtained in this study. After the sectional dimension of the chord member is determined by static analysis, the dimension of arch rib, i.e. the distance between two chord members at the same level and in elevation can be determined respectively by stability and dynamic analysis. The calculating principle is that the in-plane first buckling load factor and frequency should be close to the out-of-plane first buckling load factor and frequency.

6. Reducing arch rib size not only reduces material costs, but also facilities arch rib manufacture and erection. And the whole structure will look more elegant.

7. The dead load of this bridge is huge, almost 11 000 tons. The bridge deck is as much as 75 percent of the total dead load, which necessarily requires the large dimension of the arch rib. In order to increase the span of this kind of bridge, the self-weight of the bridge deck should be reduced in addition to optimizing the arch rib. High strength materials must be adopted and reasonable structure type of bridge deck must be considered.

8. The appropriate FEM of a structure is the key to successful analysis. The structure should be modeled with as much detail as possible to represent the geometric and structural form.

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