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SEISMIC-RESISTANT ANALYSIS AND DESIGN OF LARGE-SPAN PRE-TENSIONED STRUCTURES

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SUMMARY

In recent ten years a lot of Large-Span Pre-tensioned Structures have been widely applied in China. In the cable Structure design theory, seismic-resistant analysis and design is one of the most important constituents of the limit state design. Because of the complicacy of its dynamic response, there is not a sound cable structure design code for this aspect. In this paper Seismic-Resistant Analysis and Design method is presented, and its reasonability is proven through two numerical examples.
Key words: Large-Span Pre-tensioned Structures; Seismic-Resistant Analysis and Design

INTRODUCTION

In recent ten years a lot of cable structures have been widely applied in China (see Fig.1). These structures can be divided into different categories depending upon the criterion used for classification. In accordance with their stiffness characteristics, they can be classified as: stiff、semi-stiff and flexible structure. In the cable structure design theory, seismic-resistant analysis and design is one of the most important constituents of the limit state design. Because of the complicacy of its dynamic response, there is not a sound cable structure design code for this aspect.

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Fig. 1 Example of cable structure: Zhouzhou International Conference & exhibition Center

Recently much attention has been paid on Seismic-Resistant Analysis and Design of Large-Span Pre-tensioned Structures. Most analyses only aim at individual projects, but there has been not practical achievement in this field. In this paper Seismic-Resistant Analysis and Design method is presented, and its reasonability is proven through two numerical examples.

SEISMIC-RESISTANT ANALYSIS AND DESIGN OF LARGE-SPAN PRE-TENSIONED STRUCTURES

Seismic-Resistant Analysis

The equation of motion of cable structures can be written down as

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=-[M]\{\ddot{U}_g\}+\{R\} \quad (1)$$

$$[C]=r[M]+s[K] \quad (2)$$

$$[K]=[K_E]+[K_G] \quad (3)$$

where $[M],[C],[K]$ are mass, damping, and stiffness matrices of large-span pre-tentioned structures. $\{\ddot{U}\},\{\dot{U}\},\{U\},\{\ddot{U}_g\}$ are nodal acceleration vector, nodal velocity vector, nodal displacement vector, and ground acceleration vector. $[K_E]$ is the elastic stiffness matrix, $[K_G]$ is the geometric stiffness matrix. $\{R\}$ is the total non-equilibrium force.

There are two kinds of methods to solve the equation of motion: the mode superposition method and time-history analysis. The former is a linearized dynamic analysis method, and the latter is a nonlinear method.

One factor to remember in the dynamic analysis of cable systems is that superposition of loads and displacements is not strictly valid for nonlinear problems. Thus, the mode superposition method of dynamic analysis in which results from decoupled analyses of the separated modes are superposed is not a true representation of the nonlinear behavior.

However, if a prestressed static configuration is established which has significant stiffness and if small in-service dynamic loads are then added, it is reasonable to assume linearity of the superposed dynamic response. In this case, the mode superposition method can be used.

When considering flexible systems wherein the prestressed stiffness is small, geometric nonlinearities predominate in the dynamic response. In this case, the direct time integration technique can be used in the time-history analysis. This technique can also be used for stiff systems in place of using the mode superposition method presented previously.

In either mode superposition or time-history analysis, the initial state of cable structures should be firstly analyzed and obtained, on which the seismic-resistant analysis can be carried out.

Initial state analysis

Initial state is defined as the self-equilibrium state of cable structures with self-weight and pre-tensions. According to the characteristic of structures, there are two kinds of methods for initial state analysis: nodal equilibrium method and deflection method. The former is suitable for the cable-member pre-tensioned structures, and the latter is for the cable-beam structures. For cable-member structures, the initial state can be realized and its final form should fulfill the nodal equilibrium conditions. But for cable-beam structures, the deflection method should be satisfied in the initial state, and of course the equilibrium condition should be fulfilled at initial state.

For cable-member structures, the nodal equilibrium conditions can be written as:

$$[A]_{n\times m}\{t\}_m=\{F\}_n \quad (4)$$

where, m = number of cable or member elements;

n = non-restrained DOF;

[A] = coefficient matrix relating to nodal geometries;

{t} = pre-tensions of members;

{F}= nodal force vector.

Assuming the expectation of $\{t\}$ is $\{t_0\}$, which is not necessary to fulfill the equilibrium, and the pre-tension vector $\{t\}$, which satisfies the equilibrium's condition.

The difference between {t} and $\{t_0\}$ is {dt}, which can be written as :

$$\{dt\} = \{t\}_M - \{t_0\} \tag{5}$$

The optimization solution is to which fulfills the equilibrium as (4) and $\{dt\}$ is the smallest.

The {t} can be obtained by using least two-multiple method:

$$\mathrm{M}_s = \|\{dt\}\| = \{dt\}^T\{dt\} \tag{6}$$

$$\delta \mathrm{M}_s = \sum_{i=1}^{k} \frac{\partial \mathrm{M}_s}{\partial t_{ki}} \delta t_{ki} = 0 \tag{7}$$

Substituting (5) into (6), and then into (7), we get (8):

$$\begin{gathered}[BB_s]\{t_k\} = \{B_s\} \\ \{t_k\} = [BB_s]^{-1}\{B_s\}\end{gathered} \tag{8}$$

Where: $[BB_s]$ is sym. matrix and $\{B_s\}$ is right item.

For cable-beam structures, the forming process from the zero-stress state to the initial state should obey the deflection coordinate conditions and the final pre-tensions of structures should fulfill the equilibrium conditions. From very stiff cable structures, the deflection from the zero state to the initial state can be ignored, therefore the equilibrium pretensions can be simply and easily obtained through so-called "add force and remove member" method, shown as in fig. 2(a).

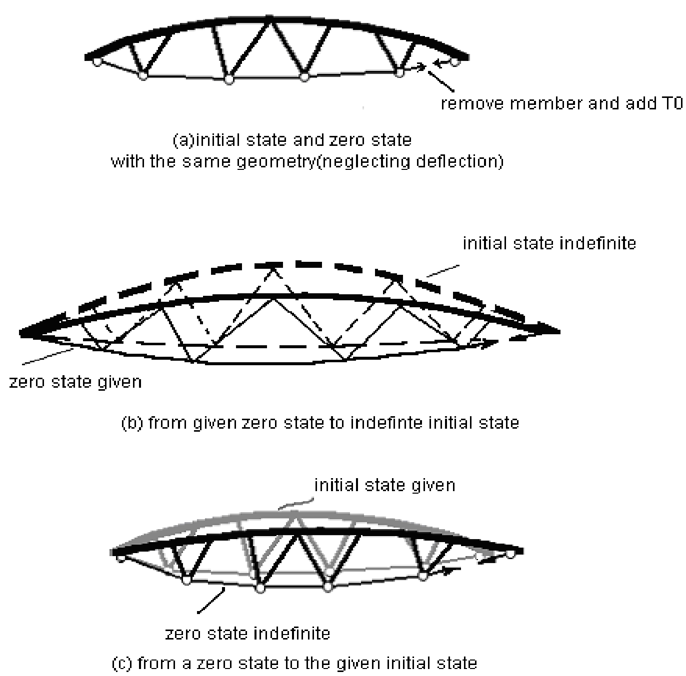


Fig.2 zero state and initial state

But for semi-stiff or flexible cable structures, the initial state analysis becomes sophisticated. If the geometry at initial state can be indefinite, the simple nonlinear solution can be adopted to find the equilibrium initial state from the given zero state under the prescribed pretension T_0. If the geometry at initial state should be the same as the given beforehand by architects, the corresponding initial pretensions at the given geometry should be iteratively solved by nonlinear methods (Fig. 2(b)). One of the solution methods is so-called "inverse iterative method". The zero state is firstly assumed and the prescribed pretension T_0 is acted on the structures' zero state. The deflection and the final initial state can be obtained by simple nonlinear solution. If the difference between such obtained geometry and given initial state is greater than the given error allowance, the zero state should be assumed again, and the nonlinear solution should be carried and repeated till the difference falls in the error allowance (Fig 2 (c)).

Earthquake-resistant Design

There are some special characteristic of large–span pre-tensioned structures compared the normal structures should be take account in earthquake-resistant design. These are the follows: not only the horizontal earthquake but also the vertical earthquake action should be considered.

- Slacking of cables may happen during earthquake actions.
- Generally there are very dense modes for pre-tensioned large –span structures.

Considering the above characteristics, the design code of Shanghai for cable strctures specifies the following points for earthquake-resistant design of cable structures:

When adopting mode superposition method for cable structures, the action of mode j on the i'th DOF is:

$$F_{ji} = \delta_n \alpha_j \gamma_j X_{ji} G_i \quad (i = 1, 2, \cdots n,\ j = 1, 2, \cdots m) \tag{9}$$

Where, δ_n = adjustment coefficient, can be taken as 1.25-1.30;

α_j = earthquake influence coefficient of mode j;

γ_j = participant coefficient of mode j;

X_{ji} = relative displacement at node i of mode j.

For complex and important cable structures, time-history analysis should be carried out in earthquake-resistant design. The horizontal and vertical earthquake actions should be considered respectively. More than two actual strong motions and one artificial simulation acceleration time history curve must be selected. Analysis time is typically 5-10 times of basic cycle if structures and should be 10 seconds longer.

NUMERICAL EXAMPLES

Zhengzhou International Conference & exhibition Center

Large-span pre-tentioned system is used in Zhengzhou International Conference & exhibition Center. The structural system is composed of spacial trusses that is supported by masts and cables. The main truss is the string frame system. Because the structure is complex, the time-history analysis is carried out based on the Chinese seismic design code. In this analysis, Strong motion record (EL Centro NS 1940) is selected. The partial solution is presented as follows.

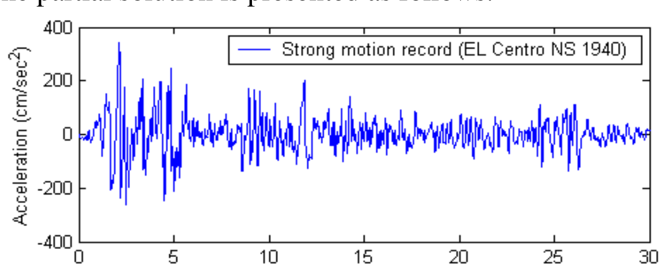


Fig. 3 Strong motion record (EL Centro NS 1940)

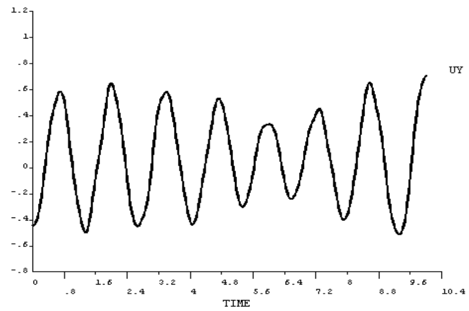


Fig. 4 Vertical displacement-time curve of a node in top chord

The maximum value of vertical displacement of a node in top chord is 0.7 meter in 10 sec.

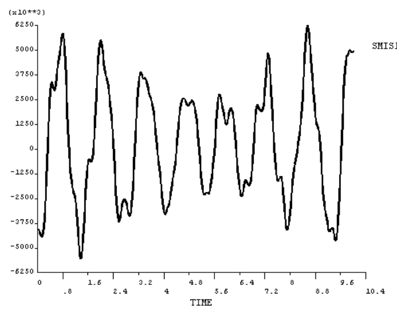


Fig. 5 Axial-force-time curve of a top chord

The maximum value of Axial-force of a top chord is 5524 KN in 1.36sec.

FuDan gymnasium

Pre-tensioned membrane roof of FuDan gymnasium is made up of steel structure and membrane structure. The steel structure is large-span pre-tensioned structure including two aspects: main arch and spacial truss. The spacial struss is hanged in main arch through a series of long cables and supported by a series of the bottom cables.

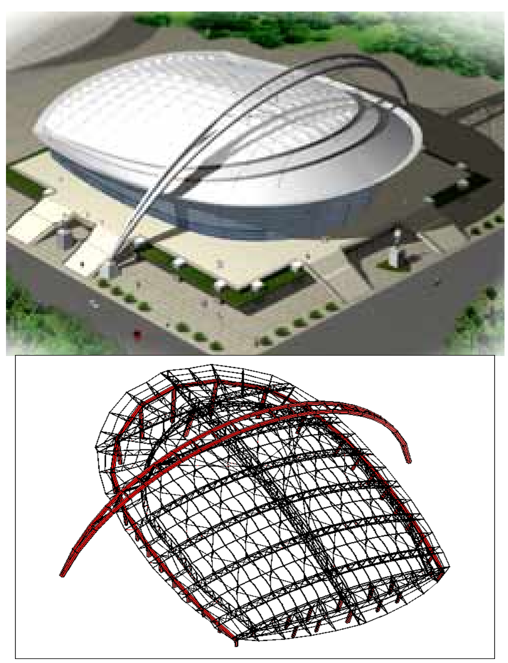


Fig. 6 FEM model of FuDan gymnasium

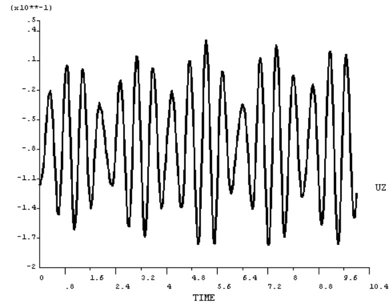


Fig. 7 Vertical displacement-time curve of a node in main arch

The maximum value of vertical displacement of a node in main arch is 0.176 meter in 7.22 sec.

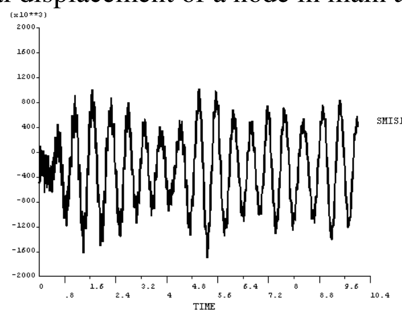
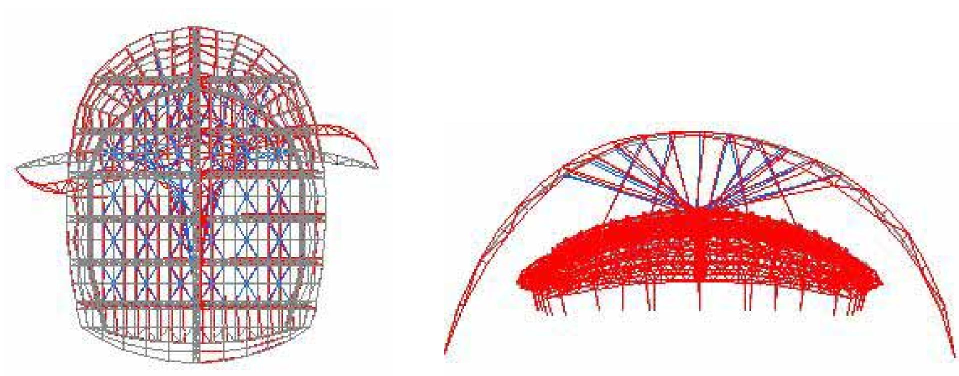
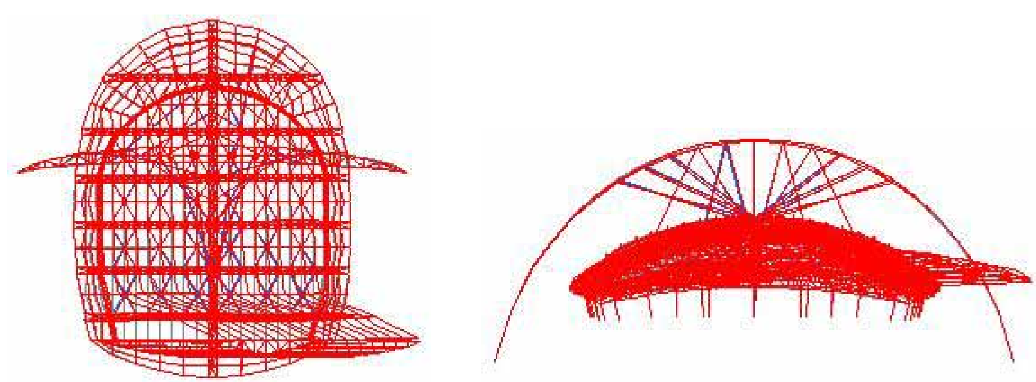


Fig.8 Axial-force-time curve of a element in main arch

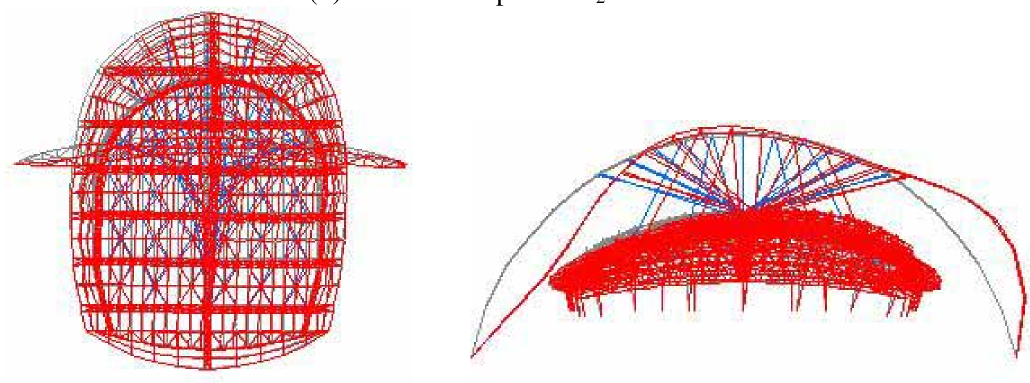
The maximum value of Axial-force of a top chord is 1702 KN in 5.28sec.



Basic period $T_1 = 0.798s$



(b) The Second period $T_2 = 0.709s$



（c）The Third period $T_3 = 0.605s$

Fig. 9 mode superposition of FuDan gymnasium

CONCLUDING REMARKS

In this paper Seismic-Resistant Analysis and Design method is presented, and its reasonability is proven through two numerical examples. For stiff and semi-stiff pre-tentioned structures, the mode superposition method can be used. For the key and complex systems, nonlinear elastic time-history analysis should be used to analyze the structures in the frequent-occurrence earthquake additionally. For flexible pre-tentioned structures, the time-history analysis must be carried out.

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