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Design and research of gymnasium structure in Zhejiang University*

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Abstract: This paper deals with the issues involved during the design of a complex gymnasium located at the new campus of Zhejiang University. The complexity comes from the gymnasium's being of three parts: long-span membrane structure, prestressed concrete structure and extraordinarily long tubular steel structure without seams. The paper first presents considerations of the prestress design, followed by analyses of the stress states due to temperature changes and concrete shrinkage. Buckling and postbuckling analyses were performed to determine the load-carrying capacity of the perfect and imperfect tubular steel structure of the inclined arch system, while dynamic relaxation method and general nonlinear finite element analysis were used to carry out shape-finding and stress analyses of the membrane structure respectively. Finally, collated monitoring date was applied to control the construction quality and verify the design parameters. Some useful conclusions are available at the end of the paper.

Key words: Prestress structure, Membrane structure, Tubular structure, Extraordinarily long seamless structure, Temperature shrinkage, Stability

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INTRODUCTION

The gymnasium project located in the new campus of Zhejiang University is a symbol building (Fig.1). The requirements on the functions and the esthetics make it distinguished from a traditional gymnasium. It is a two-story building with the first floor used for field competition and the second floor for students' basketball training. The heavy live loads acting on the second floor and the requirement of 35 m unobstructed space challenge the creativity of the structural engineers. Furthermore, the architects are concerned with the sculptural qualities and visual possibilities, as well as with the search for expression and meaning. No seams are permitted for the 120 m long structure. The beauty of tension roof structures connecting and unifying the inner and outer worlds, in working together with nature instead of subduing it, inspired the architectures adopting tension membrane structures in the area that is frequently attacked by strong typhoon during summer. So, the engineers have to solve the following problems with respect to the structural designs through careful layout of the structure and rigorous calculations: (1) heavy live loads; (2) long span space; (3) extraordinary length; and (4) flexible construction. Prestressed reinforced concrete members, tubular steel members, cables and thin tensile membranes are combined in an ordered manner according to form and structural behavior. The present paper discusses the issues involved during the structural design of this complex structure. Special attention was paid to the effective prestressing of the structure, the evaluation of stresses due to temperature changes and the design of the flexible membranes.

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Fig.1 Perspective view of the gymnasium

FLOOR SYSTEMS

For getting a view of open spaces, no columns are permitted on the ground floor except on the perimeter. The second floor beams span on columns 35 m apart. The large scale of the long-span structure and the heavy live loads (5 kN/m^2) on the floor require unique building configurations quite different from traditional forms and require more precise evaluation of loading conditions than just provided by codes. Considering the potential consequence of failure endangering a large number of students, special care was taken in the design of the long-span structure, including prestress technology, rigorous calculations of the stresses due to temperature changes, the placement of shrinkage joint, and a much more comprehensive field inspection and monitoring.

Prestressed floor design

Steel strands with high strength and low relaxations were employed as prestressing material in order to ensure the construction quality and were arranged in a curved manner with both end tendons anchored at the upper part of the beam. The inflection points and the vertical space between steel strands were determined after comparing the prestress effects of various schemes. The configurations shown in Fig.2 follow the tensile stress flow in the typical interior reinforced concrete beam under uniformly distributed loads.

Besides the general considerations on the arrangements of the steel strands, careful calculations were also performed to investigate the effects of the following factors on: (1) the ratio of the live load to dead load; (2) the upper limit of arch camber during the process of construction and use; (3) loss of prestress in the steel strands due to shrinkage and creep; (4) sequence of prestressing and the interaction of prestressed members; (5) efficient transfer and establishment of prestressing forces.

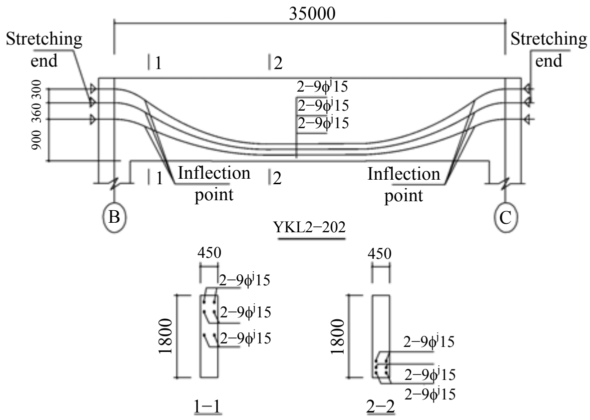

Fig.2 Configuration of the typical interior prestressed reinforced concrete beam

Apart from the determination of the size of the steel strand, the application of hoop steel in prestressed beam is necessary to avoid failure due to the shear forces. The amount of the hoop steels should not only meet the minimum values specified by the design code, but also to ensure that the calculated shear forces are greater than the shear resistance of the beams. According to the concept of balancing constant load or the concept of cable-beam bearing load separately (Xie and Zhuang, 2001), part of the reverse balancing loads engendered by prestressing forces are transferred to the columns. The beams only need to resist the remainder of the not yet balanced exterior loads. On the other hand, in small strain theory, frames subject to equivalent uniform vertical loads can be divided into two sub-structure systems: a prestressed cable framework and a normal framework, which can be analyzed with respect to the shared loads separately. The prestressed cable bears the load of $\lambda_p(g+q)$, and the load transfers from the cable structure to the beam-column connections, while the normal framework carries the vertical loads $(1-\lambda_p)(g+q)$ and can be analyzed with well-known principles for ordinary frameworks. Therefore, the shear strength of the normal framework should satisfy

$$V \le V_{cr} \tag{1}$$

here, V is the maximal shearing force and V_{cr} is the shear resistance provided by the concrete and hoop steel.

Extra-long structure design

The maximum allowable length for multi-story buildings in the China design code is 55 m. Buildings that satisfy this specification can omit the stresses due to temperature changes, otherwise, special measures should be considered to avoid potential cracks and relaxing deformations, for example, providing movement joints at the critical locations, setting control joints and shrinkage joints at the specified sections, or, performing rigorous calculations to evaluate the loads (stresses) due to temperature changes. Obviously, the present gymnasium has exceeded the maximum length and cracks are expected if no measures are taken. On the other hand, the prestresses introduced along the span direction may have negative effects on the control of the cracks along the building length. Due to the Poisson's ratio not being equal to zero, compressive stresses on the member sections in the span direction produce tensile stresses in the length direction, which will in turn crack the concrete that is weak in tensile strength. It is necessary to check whether it is true. Numerical calculations as well as field-testing showed that this is only a conjecture, therefore this kind of stress is ignored during extra-long structure design.

Setting shrinkage strips (temporary joints) that are left open for a certain time to allow the early drying shrinkage for concrete is a well-accepted method to control temperature shrinking cracks, but the effect is influenced by some uncertain factors, such as the level of construction management. In the present project, two ordinary shrinkage strips are provided, but the cracks are controlled mainly by unbonded prestressed forces in the longitudinal direction. The magnitudes of the prestress forces are determined according to rigorous calculations of the stresses due to temperature changes and drying shrinkage in the span direction.

1. Stresses due to temperature changes

The temperature changes can be roughly grouped into: daily changes and seasonal changes. As there are roof structures, the daily temperature changes are assumed to be less critical than the seasonal changes. According to the weather data, the temperature changes can be conservatively assumed as ΔT=40 °C. The corresponding stress is estimated as

$$\sigma_T=\pm\alpha_c\Delta T E_c\alpha_1 \tag{2}$$

Here: $\alpha_1=1-\dfrac{1}{ch(\beta L/2)}$; $\beta=\sqrt{2C_x/H_2E_c}$; α_c=the coefficient of linear thermal expansion; E_c=Young's modulus of the concrete; H_2=the span; L=the length of the floor slab, taking as the distance between the shrinkage strips (30 m); C_x=1.0 N/mm^3.

Substituting the values for the C40 concrete, the calculated stress is ±2.22 MPa, which exceeds the concrete's tensile strength f_t =1.71 MPa.

Rigorous numerical analysis showed that apart from the membrane stresses with magnitudes very similar to the above-mentioned values, there are also bending stresses that vary linearly with the slab thickness; but much less than the membrane stresses, so they are neglected for simplicity.

Tensile stresses result from shrinkage in wintertime (negative temperature changes), while compressive stresses result from expansion in summertime (positive temperature changes).

2. Stresses due to concrete shrinkage

The shrinkage stresses can be estimated as

$$\sigma_{tr}=\varepsilon(t)E_cH(t) \tag{3}$$

here, $\varepsilon(t)=\varepsilon_y^0 M_1M_2...M_{10}(1-\mathrm{e}^{-0.01t})$.

The definition of each variables can be found in Wang (1997). The parameter values are: M_2=1.35, M_6=1.11, M_{10}=0.86, and the other equal to 1.0. The calculated strain (ε_y) due to the concrete shrinkage is 3.61×10^{-4}. If concrete additives were used to prevent the occurrence of shrinkage cracks during the construction, the possible expanding strain is estimated as 3.85×10^{-4} for gentle 10% volume expansion, so the final shrinking strain is –0.24×10^{-4} and the corresponding stress is –0.39 MPa (compression).

3. Prestressing load

The tensile stresses just discussed may act simultaneously and should be combined, as they are superimposable. The maximum tensile stresses due to temperature changes and the concrete shrinkage are

$$\sigma_{wcr}=\sigma_T+\sigma_{cr} \tag{4}$$

Assuming this tensile stress should be balanced by the compressive stresses induced by prestressing the floor slab along the building length, the required prestress strength is

$$\sigma_p=\sigma_{wcr} \tag{5}$$

Substituting the values for σ_T and σ_{cr} into Eq.(4) for σ_{wcr}, yields the total prestressing force. In this case, concrete cracks associated with the temperature changes and shrinkage can be avoided. Careful inspections after the erection of the structure confirmed that the analysis results are right.

DESIGN OF TENSIONED MEMBRANE ROOF STRUCTURE

As tensioned membrane structures provide a light, elegant, and efficient structure spanning over a large clear space, they are most suitable for use as roof structure for a variety of building types. Tensioned membrane structures are constructed from coated fabric, rigid beams/frames, and flexible cables. The design of tensioned membrane structures consists of three stages: shape finding analysis, cutting pattern analysis, and analysis of their structural behavior under normal climatic loads (e.g. snow load, wind load). The purpose of the shape finding analysis is to find the structure form that can satisfy the pre-defined stress system, to seek a prestress system that satisfies the configuration required by architecture. The prestressed system and the structure form are inextricably linked. Any adjustments made to either one will inevitably affect the other. Many technical procedures and algorithms have been developed to find the structure form and to determine the prestressed system. They can be roughly sorted into three groups: force density methods; dynamic relaxation method; general non-linear finite element methods. In the present project, the dynamic relaxation method was employed for shape-finding analysis.

Rigid steel arches

The basic support unit is the two inclined steel arches connected laterally with three interim beams near and at the crown (Fig.3). The design of the arch concerns two issues (Zhang *et al*., 2002): stability and thrust forces at the hinged ends. A general nonlinear finite element program ABAQUS (Hibbit *et al*., 2003) was employed to perform eigenvalue buckling analysis and general nonlinear analyses. Two load cases were considered in the analyses: fully distributed vertical loads and half-span distributed vertical loads. The geometric parameters and the material properties as well as the preliminary design procedures are given in Wu *et al*.(2003). Only the analysis results of the structure subject to full span loads are presented in this paper: results for half-span loads are given by Wu *et al*.(2003). In order to consider the influences of the lateral deformations of the supporting members, the concrete columns are included during the analyses. The inclined arches are assumed to pin at the top of the concrete columns, while the bottom of the concrete columns are fixed.

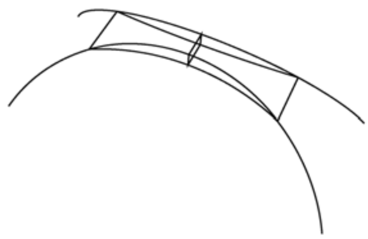

Fig.3 Perspective view of the inclined arch system

Fig.4 shows the post-buckling analysis results of perfect and imperfect arches. The vertical axis is the load factor, while the horizontal axis is the vertical displacements over the arch span. The analysis types included: geometrically nonlinear analysis, geometrically and materially nonlinear analysis. Both perfect and imperfect arches are involved. For imperfect arches, the imperfections are assumed to be in the form of (1) lateral in-plane displacement, (2) lateral out-of-plane displacements, and (3) general vertical displacements (Fig.5). These imperfections were obtained from linear elastic buckling analyses; and are thought to represent three possible imperfections of practical arches. In Fig.4, dashed lines represent the results of geometrically nonlinear analyses, while solid lines represent the results of material nonlinearity analyses. Only limited parts are shown in Fig.4 for geometrically nonlinear analysis curves labeled as ‘Elastic’, as the limit point loads for this kind of analysis are far above the linear buckling loads (i.e. q_{cr}^{L}). Fig.4 shows that the material nonlinearity has significant effect on the limit point loads. On the other hand, the imperfections only have moderate effects on the load carrying capacity, even if the imperfection amplitude is up to L/250. The load-deflection curves in Fig.4 show that the inclined arch system consisting of two individual arches connected by three horizontal beams is very stable. This in turn demonstrates that the structural arrangements are effective, especially from the viewpoint of the lateral out-of-plane buckling.

Design of the membrane

Membrane material used in this project was Ferraris PVDF1202T membrane of polyester fiber weaving, and resining 100% PVDF colophony. The whole roof was made up of 24 pieces of membrane that are grouped into there types: olive shaped, dumb-bell shaped and boundary-polygon. The key of design and analysis for the membrane roof structure is how to design the membrane, seeking the best way to satisfy the architectural and mechanical requirement as well as optimizing the configuration (Lewis and Gosling, 1993; Zhang, 2002). Therefore, dynamic relaxation method was first employed to perform the shape finding analysis followed by a general nonlinear finite element analysis.

One of the main load cases for the membrane design is the wind load. According to Load Code for the Design of Building Structures (GB 50009-2001, 2001), the wind load is calculated according to the reference wind pressure by multiplying the shape coefficient that represent the significant effects of the building shape on the design.

The present design considers the deformation compatibility between the flexible membrane and the rigid arches. After preliminary design of the membra-

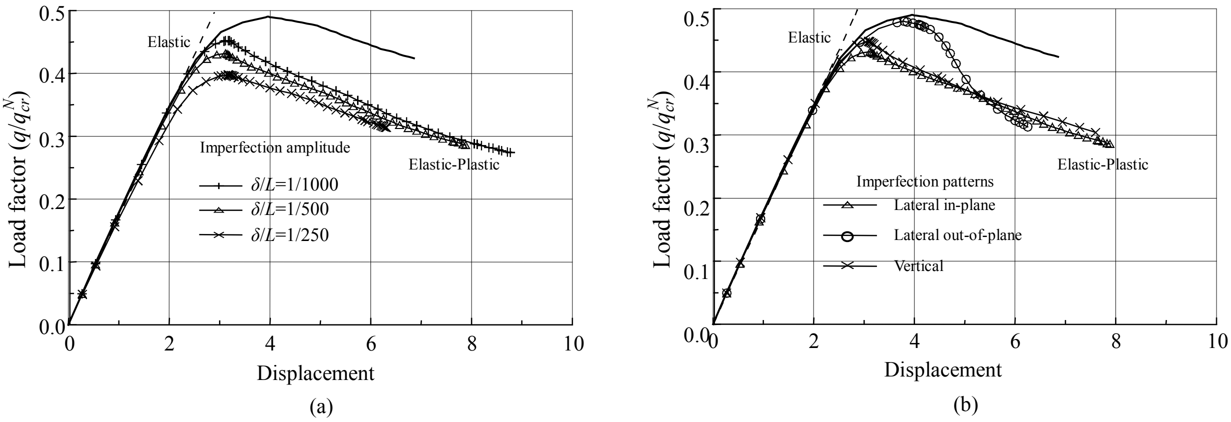

Fig.4 Load-deflection curves for perfect and imperfect arch systems
(a) Imperfection pattern: the lateral in-plane displacements; (b) Imperfection Amplitude: δ/L=1/500

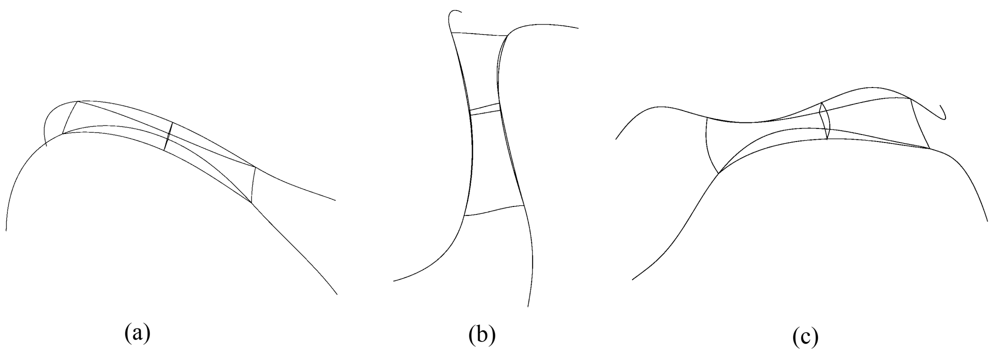

Fig.5 Three imperfection patterns (a) Lateral in-plane displacements; (b) Lateral out-of-plane displacements; (c) Vertical displacements

nes, for example, after defining the pre-stresses, the reaction forces are then fed back to the steel structures for further structural analysis. In addition, the design takes the interaction between the prestressed floor concrete structure and the tensioned roof structure into consideration. Several working stages are concerned during the design, i.e., the construction stage of prestressing tension, without considering the roof; and the stage of usage, the reaction from the roof steel structures acting on the top of the concrete columns as they are outside forces.

MONITOR

To control the construction quality and to verify the design considerations (Wu *et al*., 2003), field tests or monitoring were carried out. The monitoring content included the prestresses of the steel strands, concrete strains at the selected sections, observing concrete cracks, etc. Some conclusions were obtained from the field tests.

1. Anchoring forces of the pre-stressed cables stabilized before injection of concrete and satisfied the minimum values as required by the design;

2. Prestressed anchoring loss is a little larger than the theoretical value; prestressing loss due to duct friction is generally smaller than theoretical values, except for a few cases;

3. Beams camber upward when they are pre-stressed, with the measured magnitude of the camber being in good agreement with theoretical values. For example, the measured cambers of the beams in axis 17 and 23 were 9.91 mm and 11.33 mm, while theoretical value is 10 mm for the two beams;

4. Test stress results on the prestressed floor slab during the process of tension showed that the diffusibility of prestresses is rather good. Compressive stress distribution along the concrete floor sections were nearly uniform, as required by the design. At the same time, it was observed that the prestressing of the steel strands along the length had little influence on the perpendicular steel strands.

CONCLUSION

The design, construction and testing of this project yielded the following conclusions:

1. Prestressed reinforced concrete technology can solve many structural problems associated with large-span structures. Its feasibility, integrity, durability, economy and easy maintenance, make it worth to extending to other structures.

2. Through rigorous calculations, extraordinarily long seamless structure can be realized without serious structural problems. Prestressed technology can further improve the structural behavior and help avoid concrete shrink crack.

3. The combination of two inclined arches can greatly improve the stability behavior of arches. Imperfection sensitivity analyses showed that this kind of arch system is insensitive to imperfection. The load carrying capacity can be determined by performing a general geometrically and materially nonlinear finite element analysis.

4. The interaction between two different structural systems should be considered during the design. Appropriate testing or field monitoring can help establish modern structure design technology.

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