Stability Analysis of the Thin Concrete Walls of the Hangzhou Grand Theater

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Abstract

This paper presents the analysis of stability for a thin concrete wall in the Hangzhou Grand Theatre. In order to satisfy the service requirement, the walls bearing heavy vertical and eccentric loads are widely used with highly slenderness ratio of compressive members. The ratio of height to thickness is nearly 50. How to deal with the stability of the thin concrete walls becomes a difficult problem which may affect whether the architecture scheme can be rightly carried out or not. The planar and spatial finite element models have been constructed by the ANSYS program, and then linear and nonlinear buckling analysis of Hangzhou Grand Theatre are carried out by the ANSYS program. The practical suggestions are drawn from buckling analysis.

Introduction

Standing on the south-east corner of the New City Square and being contiguous to Qiantiao River, the Hangzhou Grand Theatre resembles a glittering crystal palace with a perfect harmony of glass and steel, as shown in Fig.1. The theatre is a multi-functional cultural complex where the public can witness world famous opera, ballet and orchestras or simply come at their leisure to eat and shop, and which can additionally offer the highest standard for international conventions and gathering.

Figure 1 - The Grand Theatre Aerial View
The total floor space within the complex is approximately 50,000 square meters, comprising four underground levels and ten above, as shown in Fig.2 and 3. The main interior includes a Lyric Theatre seating 1,719, a Studio Theatre seating 875, a Drama Theatre seating 458 and a number of multi-propose rooms, such as bars, restaurants, conference rooms.

**Figure 2 - The Grand Theatre General Plan**

**Figure 3 - The Grand Theatre 1-1 Sectional Drawing**

In order to satisfy the service requirement, especially to ensure first class presentation and performances, the deflection gap in architecture scheme is set between the Lyric Theatre, the Studio Theatre, the Drama Theatre and their ambient structure respectively. So compressive side walls with the highly slenderness ratio are widely used in these three theatres. The Lyric Theatre is shown in Fig.4. Its side walls are 24m in height, 31m in length, 0.5m in thickness. The ratio of height to thickness is nearly to 50. Along the side walls, the pipe channels (width 0.5m) are arranged every 3m. The side walls are divided to thin wall limbs (width 2.5m). There are audience grandstands directly stretched 3.5m out from the side walls. Considered the side walls bearing heavy vertical and eccentric loads, how to deal with the stability of the thin concrete
side walls becomes the difficult problem which may affect whether the scheme can be rightly carried out or not.

**Figure 4 - Service Load Drawing**

**Procedure**

In the paper, several planar and spatial finite element models have been constructed by the ANSYS program (Reference 1) in order to analyze concrete side wall buckling of the Lyric Theatre. All of these models bearing the service load are based 2-D or 3-D structure with rigid fixed ends, as shown in Fig. 5. The Young's modulus of the material is 30Gpa. The buckling load and buckling mode shape of the side walls are predicted by nonlinear buckling analysis and linear buckling analysis. The effects of boundary members for the unstable deformation on the buckling strength are investigated.

**Figure 5 - The Lyric Theatre Plan**
Models

2-D analytic model consists of single wall limb taken the whole side walls. The element type BEAM4 or SHELL63 is used as shown in Fig.5 and 6, denoted as S10 and S11 respectively. 3-D analytic model denoted as S2 is shown in Fig.7 using SHELL63. In practical design, the ambient walls sited at two ends of the side walls may act as boundary members. The calculated model built using SHELL63 is shown in Fig.8, denoted as S3.
Analysis Results & Discussion

Linear buckling analysis

Linear buckling analysis predicts the theoretical buckling strength of an ideal linear elastic structure. It is assumed that structure configuration has no change in the process of loading. The buckling load is taken as the load when the determinant of stiffness matrix becomes zeros. Eigenvectors corresponding to unstable state are calculated.

Its buckling judgment guidelines is expressed as

$$\begin{vmatrix} K_O + k_{cr} K_G \end{vmatrix} = 0$$

where,

$K_O =$ initial stiffness matrix.

$K_G =$ geometrical stiffness matrix.

$k_{cr} =$ buckling load factor, $k_{cr} = \frac{P_{cr}}{P}$.

$P_{cr} =$ buckling load.

$P =$ design service load.

Buckling load factors obtained from linear buckling analysis are presented in Table 1.

Table 1. Buckling load factors obtained from linear buckling analysis
Some conclusions can be drawn as follows:

1) Boundary members can effectively increase the buckling load of the side walls and play a role in restraining the unstable deformation. The buckling load factor of S3 is nearly 350% higher than that of S2. In the buckling mode of S3, the deformations corresponding to the boundary members are shown as Fig. 9. However, such deformations do not appear in the buckling modes of S10, S11 and S2.

2) Compared with S11 and S2, the buckling loads are almost the same and both the buckling modes show the similar buckling behavior, as shown in Fig. 10. So 2-D analytic model can truly reveal mechanical performance of the real structure without boundary members.

3) The results obtained from using element type beam4 and shell63 have small difference from the result of 2-D analytic model. It is suggested that 2-D analytic model by using element type beam4 be used to calculated the stability of single wall limb.
Nonlinear buckling analysis

In linear buckling analysis, imperfections and nonlinearities prevent real-world structures from achieving their theoretical elastic buckling strength. Nonlinear buckling analysis is usually the more accurate approach and is therefore recommended for design or evaluation of actual structures. Using the nonlinear technique, the analytic model can include features such as initial imperfections, plastic behavior, gaps and large deflection response.

In addition to bearing service load in the 2-D calculated model S10, the walls also sustain the lateral force $F=1,5,10,115\text{kN}$ acting on the top of wall (where, $F=115\text{kN}$ is maximum horizontal seismic earthquake) or have initial slope value $e=0.002,0.004$ (where, $e=0.002$ is norm permitted limit value as Reference 2 noted). Then the relationship of horizontal displacement at the top of wall and the incremental loads $IL$ (where, $IL=$the ratio of applying load to design service load) is obtained, as shown in Fig.11. The buckling load factors can be concluded from point of inflexion, as shown in Table 2.

![Figure11 - Relationship of horizontal displacement of the wall and the increment load](image)

Table 2. Buckling load factors obtained from nonlinear buckling analysis and horizontal displacement of the top of wall

<table>
<thead>
<tr>
<th>Lateral Force or Initial Wall Slope</th>
<th>$F=1\text{kN}$</th>
<th>$F=5\text{kN}$</th>
<th>$F=10\text{kN}$</th>
<th>$F=115\text{kN}$</th>
<th>$e=0.002$</th>
<th>$e=0.004$</th>
</tr>
</thead>
<tbody>
<tr>
<td>horizontal displacement(mm)</td>
<td>IL=1</td>
<td>0.81</td>
<td>4.79</td>
<td>9.76</td>
<td>114</td>
<td>2.02</td>
</tr>
<tr>
<td></td>
<td>IL= kcr</td>
<td>28.8</td>
<td>70.9</td>
<td>90.6</td>
<td>8.1</td>
<td>39.8</td>
</tr>
<tr>
<td>Kcr (buckling load factor)</td>
<td>3.9</td>
<td>3.5</td>
<td>3.2</td>
<td>0.1</td>
<td>3.7</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Some conclusions can be drawn as follows:

1) The larger lateral force acting on the top of walls is, the faster the horizontal displacement increases. When the earthquake force is equal to 0.1 times design service load, the horizontal displacement...
suddenly increases and the wall limb appears buckling. So side walls without boundary members can not satisfied anti-seismic requirement.

2) The larger the initial slope of wall is, the faster the horizontal displacement increases. The incremental displacement in the case of $e=0.002$ is two times of that in the case of $e=0.004$. Applied buckling load on side walls with the initial slope value $e=0.002$, displacement at the top of the wall increases to about 40mm. The buckling crush of structure belongs to brittleness breakdown.

3) Considering the influence of the initial imperfections in the real structure and the non-vertical load, the buckling load factor is far smaller than that of linear buckling analysis. When $IL<4.09$ (Where, 4.09 is buckling load factor got from the linear buckling analysis), the tangential stiffness matrix has been decreased sharply and the displacement at the top of wall far exceeds the norm permitted value. So the nonlinear buckling analysis should be priority to applied in order to ensure the security of structure.

**Conclusion**

This paper presents the analysis of stability for thin concrete wall in Hangzhou Grand Theatre. Linear and nonlinear buckling analyses are applied. It is shown that the buckling load factor predicted by the linear buckling analysis is far higher than that of the nonlinear buckling analysis, and so linear analysis can not be applied in practice especially in the case of the structure loaded lateral force or moment and having the initial imperfections. It is important to apply the nonlinear buckling analysis. The linear buckling analysis can be used as assistant approach. It is suggested that to reinforce the structure unity be the most effective method to increase structure stability.

**References**
