Civil Engineering and Technology
December 2015, Volume 4, Issue 4, PP.40-47

The Study on Large-tonnage Pile Static Loading Test

Haifeng Deng, Wenran Cao, Xiaoqiao Luo

CNPC Research Institute of Engineering Technology, 300451, China

Email: denghaifeng@cnpc.com.cn

Abstract

Nowadays large tonnage pile foundation has been used in a large number of high-rise buildings and bridges, so it is necessary to design more secure and reliable loading system for ascertaining the pile bearing capacity in the pile static loading test. This paper adopts the stack method for large tonnage (75000kN) pile static loading test to further study the loading system with combining the theoretical and numerical methods, which can provide scientific basis for testing of large-tonnage pile foundation to some extent.

Keywords: Large-tonnage; Static Test; Loading System; Numerical Calculation

1 INTRODUCTION

Pile static loading test is still recognized as one of the most intuitive and reliable methods up to now. With the rapid development of economic construction, large buildings raise higher demand on the pile bearing capacity. Furthermore, the maximum load undertaken domestic is around 5000kN, and testing apparatus need to be improved instead of traditional loading system, resulting in the test into a kind of practical project, in addition, it is time-consuming and of very high cost. Therefore further work for large-tonnage pile static load test needs to be carried out gradually. The promotion and application of this research results will have significant social and economic benefits.

Stack method is applied to the pile static loading test in this paper. Combined with theoretical and numerical method, a set of loading system is designed in order to provide a scientific basis for the actual testing work.

2 LOADING SYSTEM

Loading is controlled by jack, and based on relative blocks automatic loading can be carried out. Device which can provide counterforce is called weights platform reaction force device, constituted by buttresses, main beam, secondary beam, concrete test blocks as shown in Figure 1.

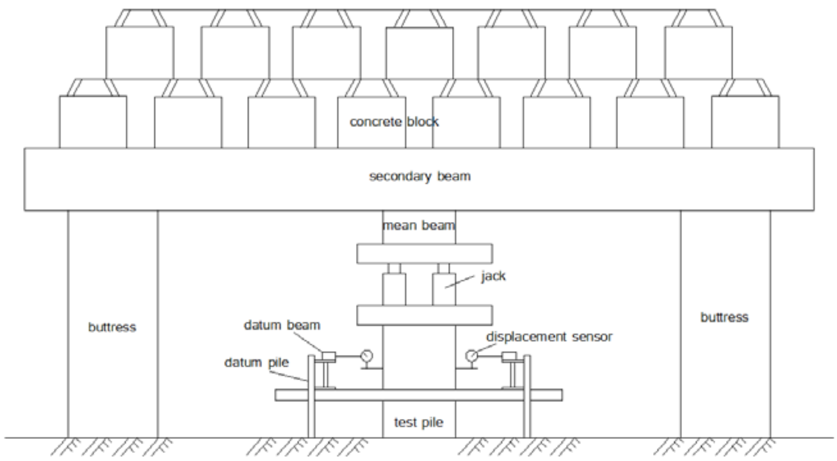


FIG. 1 FIGURE OF LOADING PLATFORM TESTING APPARATUS

3 DESIGN AND CHECKING OF PRIMARY AND SECONDARY BEAMS

The maximum loading is 75000kN, which is 1.2 times the practical loading.

Primary and secondary beams are Q420 steel beams, with the cross section H-type. In order to ensure the overall stability of the secondary beam, we weld the pair of adjacent secondary beams together.

The height, width, plate thickness, stiffener spacing and length of girder are 1500mm, 500mm, 40mm, 60mm, 150cm and 8m, respectively.

The height, width, plate thickness, stiffener spacing and length of secondary beam are 900mm, 300mm, 20mm, 26mm, 50cm and 12m, respectively.

Five jacks are applied to work. According to the plate number and size of jack, we choose five main beams to bear the overlying load, and 26 secondary beams are used based on the beam size.

As follows, the checking of intensity, stiffness and stability for primary and secondary beams are presented based on "Steel Design Code" GB50017 -2003 respectively.

3.1 Checking of Intensity and Stability for Secondary Beams

The uniform loading acting on the secondary beam is 15000kN/(26×12m)=240.38kN/m. Caps length on both sides of the cantilever is 1m, as shown in Figure 2.

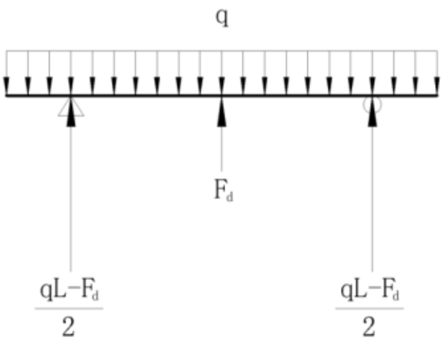


FIG. 2 COMPUTING MODEL DIAGRAM FIG

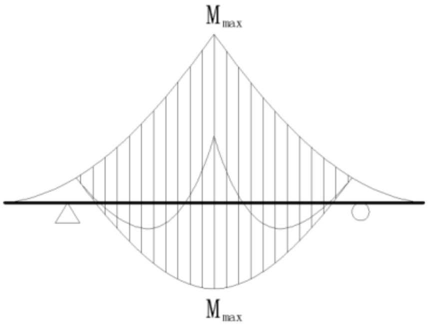


FIG. 3 BENDING MOMENT ENVELOPE DIAGRAM

Bending moment and shear force envelope diagram of secondary beam are shown in Figures 3 and 4.

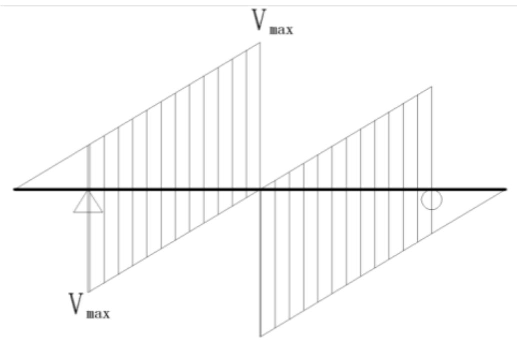


FIG. 4 SHEAR FORCE ENVELOPE DIAGRAM

The maximum moment of secondary beam is computed in two cases:

When no load is applied;

When all load is applied;

Case 1:

$$M_{\max 1} = \frac{ql^2}{8} = \frac{240.38 \times 10^2}{8} = 3004.75 kN \cdot m$$

$$V_{\max} = \frac{ql}{2} - ql' = \frac{240.38\times 12}{2} - 240.38\times 1 = 1201.90kN$$

Case 2:

$$M_{\max 2} = \frac{ql_1^{\,2}}{2} - Nl_2 = \frac{240.38\times 6^2}{2} - 240.36\times 5 = 3125.04kN\cdot m$$

$$V_{\max} = \frac{62500}{2\times 26} = 1201.92kN$$

1) Checking of bending strength

$$\frac{M_x}{\gamma_x W_{nx}} + \frac{M_y}{\gamma_y W_{ny}} = \frac{3125.04\times 10^6}{1.05\times 8.881\times 10^6} + 0 = 335.12MPa \le f = 360MPa$$

In which, $\gamma_x = 1.05$ is the plastic coefficient of development of section; $W_{nx} = 8.881\times 10^6 mm^3$ is net section modulus of secondary beam.

2) Checking of shear strength

Shear strength checking in mid-span.

$$\tau = \frac{V\cdot S}{I\cdot t_w} = \frac{1201.9\times 1000\times 5.206\times 10^6}{3.996\times 10^9\times 20} = 78.29MPa \le f_v = 210MPa$$

In which, $S = 5.206\times 10^6 mm^3$ is area moment of gross cross-section to the neutral axis where the shear stress is tended to be calculated at the secondary beam; $I = 3.996\times 10^9 mm^4$ is inertia moment of gross cross-section; $t_w = 20mm$ is the total thickness of the web.

3) Checking of reduced stress around the web edge

$$\sigma = \frac{M}{I}y_1 = 331.59MPa\,;\ \tau = \frac{V\cdot S_1}{I\cdot t_w} = 51.26MPa\,;\ \sigma_c = 0MPa\,;\ \beta = 1.1\,;$$

$$\sqrt{\sigma^2 + \sigma_c^{\,2} - \sigma\sigma_c + 3\tau^2} = 343.27 \le \beta f = 396MPa\,,$$

this indicator meets the requirements.

4) Checking of integral stability for secondary beam

In order to strength the integral stability of secondary beam, a pair of beams is weld into box cross-section, in which h/b_0=900/320=2.5125<6, and l_1/b_0= 10000/ 320 =31.25<95(235/f_y) =53.15. Based on the requirements specification the beams from time to calculate the overall stability.

5) Checking of local stability of secondary beam

Based on the stiffening ribs of secondary beam, the following results can be obtained:

$$\sigma = \frac{My_1}{I_x} = 301.30MPa\,;\ \sigma_{cr} = f = 360MPa\,;\ \tau = \frac{V}{t_w\times h_w} = 51.79MPa\,;\ \tau_{cr} = f_v = 210MPa\,;$$

The top flange of secondary beam isn't subjected to concentrated load:

$$\sigma_c = 0MPa\,;\ (\frac{\sigma}{\sigma_{cr}})^2 + (\frac{\tau}{\tau_{cr}})^2 + \frac{\sigma_c}{\sigma_{c.cr}} = 0.898 \le 1\,;$$

In which, σ is the bending pressure stress on the edge of calculated height induced by the average bending moment between the calculated web segment. τ represents the average shearing force of web induced by average shearing force in the calculated web segment. σ_c is the local pressure stress on the edge of web.

The ratio of width and thickness without support about box-section on the secondary beam:

$$b_0/t = 320/20 = 16 < 40\sqrt{\frac{235}{f_y}} = 29.92$$

The result shows that the local stability of secondary meets the requirement.

3.2 Checking of the Strength and Stability about Girder

When the jack is subjected to the maximum loading, the loading on girder is about 62500kN. The loading transmit from the secondary beam is uniform, and the equivalent value on each one is 62500kN/(5×8m)=1562.5kN/m. In addition, the bearing form of girder is similar to the secondary.

The maximum bending moment in mid-span is:

$$M_{\max} = \frac{ql^2}{8} = \frac{1562.5 \times 8^2}{8} = 12500kN \cdot m$$

The maximum shearing force is:

$$V_{\max} = \frac{ql}{2} = \frac{1562.5 * 8}{2} = 6250kN$$

1) Checking of bending strength

$$\frac{M_x}{\gamma_x W_{nx}} + \frac{M_y}{\gamma_y W_{ny}} = \frac{12500 \times 10^6}{1.05 \times 5.318 \times 10^7} + 0 = 223.86MPa \le f = 350MPa$$

In which, $\gamma_x = 1.05$, $W_{nx} = 5.318 \times 10^7 mm^3$.

2) Checking of shearing strength

The shearing strength in mid-span:

$$\tau = \frac{V \cdot S}{I \cdot t_w} = \frac{6250 \times 1000 \times 3.112 \times 10^7}{3.988 \times 10^{10} \times 40} = 121.92MPa \le f_v = 185MPa$$

In which, $S = 3.112 \times 10^7 mm^3$, $t_w = 40mm$, $I = 3.988 \times 10^{10} mm^4$.

3) Checking of reduced stress on the web

$$\sigma = \frac{M}{I} y_1 = 216.27MPa \quad \tau = \frac{V \cdot S_1}{I \cdot t_w} = 84.63MPa ; \sigma_c = 0MPa ; \beta = 1.1$$

$$\sqrt{\sigma^2 + \sigma_c{}^2 - \sigma\sigma_c + 3\tau^2} = 261.27 \le \beta f = 385MPa$$

The result shows reduced stress meets the requirement.

4) Checking of stiffness

$$v = \frac{q \cdot l^4}{8EI_x} = \frac{1562.5 \times 1000 \times 4^4}{8 \times 2.06 \times 10^{11} \times 3.988 \times 10^{-2}}$$
$$= 0.00609m \le \frac{l}{400} = 0.01m$$

5) Checking of girder stability

l_1/b_0=4000/500=8<9.5, based on the standard requirement, the total stability doesn't need to be checked.

6) Checking of girder local stability

Based on the stiffening ribs of girder, the following results can be obtained:

$$\sigma = \frac{My_1}{I_x} = 150.38MPa ; \sigma_{cr} = f = 350MPa ; \tau = \frac{V}{t_w \times h_w} = 113.22MPa ; \tau_{cr} = f_v = 185MPa$$

The top flange of girder isn't subjected to concentrated load:

$$\sigma_c = 0MPa ; (\frac{\sigma}{\sigma_{cr}})^2 + (\frac{\tau}{\tau_{cr}})^2 + \frac{\sigma_c}{\sigma_{c.cr}} = 0.559 \le 1$$

σ 、 τ and σ_c are the same as above.

The ratio of width and thickness without support about the compression flange on girder is:

$$b/t = 230/40 = 5.75 < 13\sqrt{\frac{235}{f_y}} = 10.65$$

The result shows local stability meets the requirement.

4 Numerical Calculation

4.1 Numerical Calculation of Secondary Beam

Finite element model is established based on the force diagram, as shown in Figure 5.

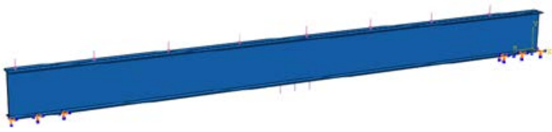


FIG. 5 FINITE ELEMENT MODEL OF SECONDARY BEAM

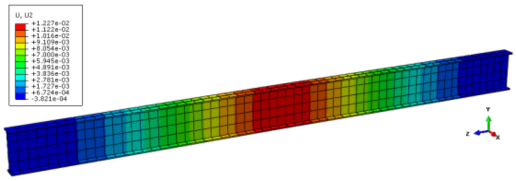


FIG. 6 THE NEPHOGRAM OF VERTICAL DISPLACEMENT

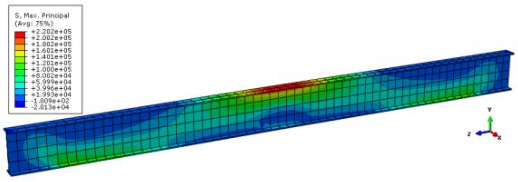


FIG. 7 THE NEPHOGRAM OF THE FIRST MAIN STRESS

Numerical results show that the maximum shearing force in dangerous section is 108MPa, maximum normal stress is 220MPa, angle deformation is 0.117°, as shown in Figures 6 and 7.

4.2 Numerical Calculation of Girder

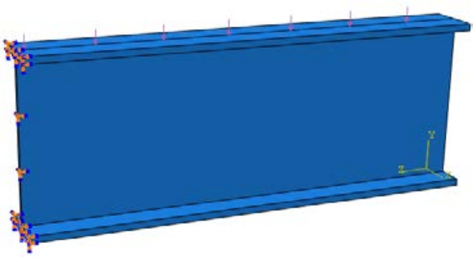


FIG. 8 FINITE ELEMENT MODEL OF MAIN BEAM

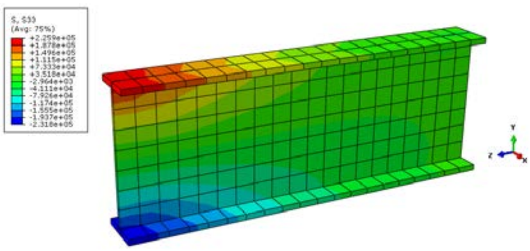


FIG. 9 THE NEPHOGRAM OF VERTICAL DISPLACEMENT

Finite element model is established based on the force diagram of girder, as shown in Figure 8.

The numerical results present that the maximum shearing force, normal force and angle deformation of girder dangerous section are about 85.13MPa, 226MPa and 0.129° respectively.

It can be found that numerical and theoretical results are slightly different. The main reason is that a simplified model is used in the theoretical calculation, in which numerical simulation simplifies the boundary conditions, while meshing form and that the plane hypothesis in three dimensional models does not hold both have a certain influence on the results, leading to a difference between the two.

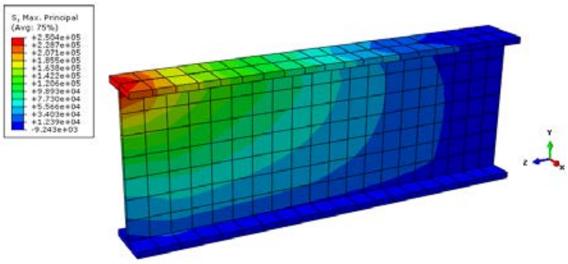


FIG. 10 THE NEPHOGRAM OF THE FIRST MAIN STRESS

4.3 Numerical Simulation of the Whole System

In order to check the safety and rationality of the loading system, a three-dimensional numerical simulation of the system is employed.

1) Finite element model

In the calculation, the cap is in the condition of full restraint, lateral restraint is applied to the primary and secondary beams, various parts of the model are used in contact with the contact algorithm, with the contact surfaces “hard contact”. 8-node reduced integration unit - C3D8R is used throughout the model.

2) Result and analysis

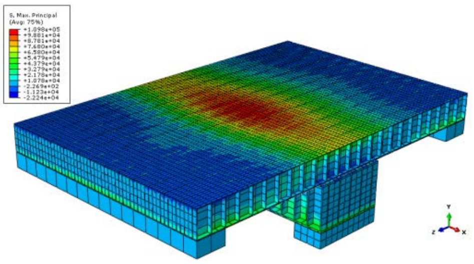


FIG. 11 THE NEPHOGRAM OF THE FIRST MAIN STRESS

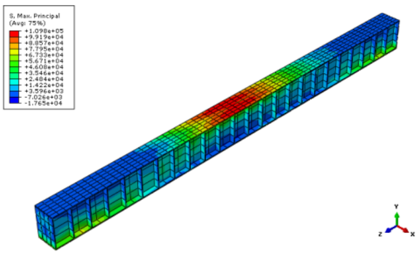


FIG. 12 FIRST MAIN STRESS FOR SECONDARY BEAM

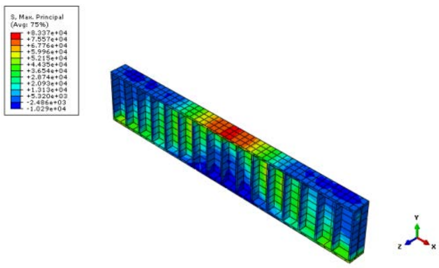


FIG. 13 FIRST MAIN STRESS FOR MAIN BEAM

It can be got from the major principal stress (as shown in Figures 11-13) that the maximum normal stress of secondary beam and girder are 110MPa and 90MPa respectively, which conforms to the requirements of the specification.

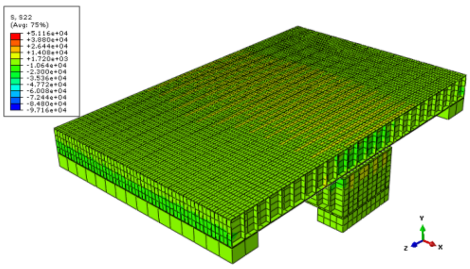
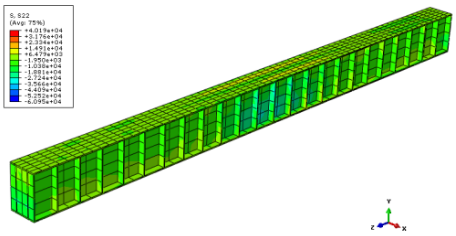


FIG. 14 STRESS NEPHOGRAM OF MODEL IN Y DIRECTION



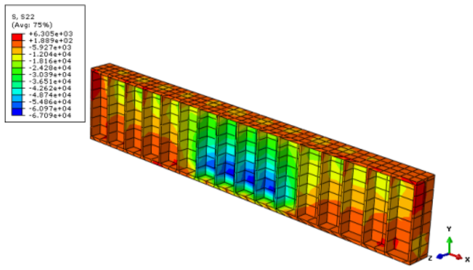


FIG. 15 STRESS OF SECONDARY BEAM IN Y DIRECTION FIG. 16 STRESS OF MAIN BEAM IN Y DIRECTION

Based on the stress nephogram of main beam in Y direction, the maximum shearing forces of secondary and main beams are extracted with 69MPa and 97MPa respectively, which conforms to the requirements of the specification.

Compared with the numerical results of major and secondary beams, it can be found that the simulated internal force is lower than the theoretical result mainly due to the adding stiffeners which can strengthen the cross-sectional moment of inertia. Meanwhile the aforementioned simulation of a single beam simplified boundary and load conditions more dangerous. In addition, meshing generation can induce the local shear stress mutation. The factors above lead to the difference between the two methods.

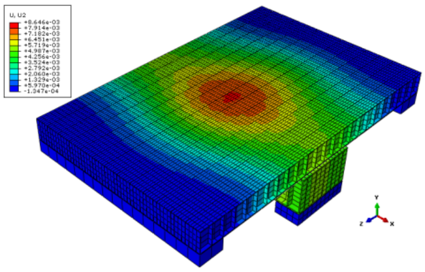
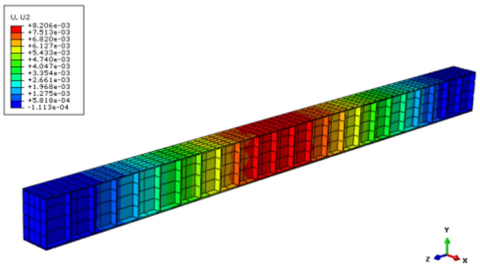


FIG. 17 VERTICAL DISPLACEMENT OF MODEL



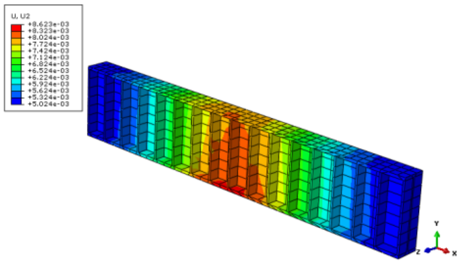


FIG. 18 VERTICAL DISPLACEMENT OF SECONDARY BEAM FIG. 19 VERTICAL DISPLACEMENT OF MAIN BEAM

It can be seen from Figures 17-19 that the maximum vertical displacement of the structure is about 0.0086m, which meets the requirements. Combined with the deformation of overall system, we can also find that the vertical displacement of the loading system is mainly caused by the deformation of the main beams.

5 LOADING HEIGHT AND ANTI-OVERTURNING MEASURE

Stacked cement blocks are chosen to lie on the loading system, with each one five tons and 1m×1m×2m. Combined with the platform area, the calculated height is about 31m. Furthermore, the crack angle is about 0.129° based on the deflection of beam, resulting in the crack distance is d=31m×tan(0.129°)≈7cm. The upper crack width is not great, while the height of the heap load is too high, which leads to the construction difficult and prone overturning.

In this paper we choose the way of increasing the platform size to reduce the height, in which a 12m long steel beam is overlapped in the axial direction perpendicular to the secondary beam uniformly. The cross-sectional dimension of steel beam above is same as the secondary beam, by this way the loading height and the cracking distance reduce to 20m and 4.5cm respectively.

Although the heaping height decreases to some extent, it is still higher than conventional test. So additional retaining structure is required to prevent the occurrence of overturning, meanwhile in the stacking process, the testing blocks need to be placed uniformly combined with cable controlled method.

6 CONCLUSIONS

This paper concentrates on the loading system of large tonnage (7500t) pile static loading test, the main conclusions are as follows:

(1) An existing beam with maximum cross-sectional dimension is selected as the primary and secondary beam, and stiffeners are added. The adjacent secondary beams are welded into integrity in order to increase the stability of the global and local secondary beams. Based on the structural behavior, the strength, deformation and stability of major beam are analyzed in a simplified form, which shows that the chosen cross-sectional size is reasonable and reliable.

(2) Numerical method is employed to simulate the whole loading system. By comparing the numerical and theoretical results of each part and the overall structure, we further validate the rationality of the loading system.

(3) Considering security problem induced by the overlying stack height due to excessive load, secondary beams are designed to be stacked up, resulting in the platform area transforms from $8\times12m^2$ to $12\times12m^2$, heaping height is down from 31m to 20m, and the cracking distance is reduced from 7cm to 4.5cm.

REFERENCES

[1] Dai G L, Gong W M, and Jiang Y S. “Experimental Research on Large Tonnage Static Test of Bridge Bored Pile”. Special Structures, 2001(6): 38-41

[2] “Design Code for Steel Structures” GB500 17 -2003 CHN. China State Construction Engineering Press, 2013

[3] Zhou M X, Cheng B H. “Detection Technology Development Status and Outlook on Vertical Bearing Capacity of Bridge Pile”. Bridge Construction, 2009 (02)

AUTHORS

Haifeng Deng, borned in August 22, 1989 in Jilin province, China, and graduated at Dalian University of Technology, Dalian, China in 2014, master of geotechnical engineering. The main field is Marine survey and structure design.