have their own characteristics with complicated lateral soil pressure distribution along pile length. With slope existence, mechanical behaviors of piles foundation have more complications due to the complexity of horizontal resistance distribution regularity and influence scope.

According to model tests of bridge piles foundation in slope, lateral soil pressure and resistance distributions were achieved through earth pressure cells embedded around model piles, and the mechanical behavior of pile foundation were discussed in different load combinations and different pier column length.

DESIGN OF MODEL TESTS

According to the engineering characteristics of bridge piles foundation in slope, aluminum tubes with 2.5mm wall thickness and 25mm diameter were adopted to simulate as pile foundation and the mixture with cement, gypsum, and sand were adopted to simulate rock mass around pile foundation. Model tests would have two groups and each tests group has four model piles in slope and one rigid pile that was adopted to calculate sub grade coefficient. The loading system included vertical load P and thrusting load q on slope top, and model test general planning was shown in Table.1.

Table.1 Model test general planning for bridge piles foundation in slop

Group	Pile	Pile length l (cm)	Pier column length l_0(cm)	Loading method	Test purpose
A	01#	100	40	$P = 0$, q multi-stage loading.	Different loading combinations have influence on model pile stress.
	02#	100	40	q suspends，P multi-stage loading.	
	03#	100	40	P suspends，q multi-stage loading.	
	04#	100	40	$q = 0$，P multi-stage loading.	
B	06#	90	30	q suspends，P multi-stage loading. Pier column length l_0 changes from 0.3m to 0.7cm.)	Pier column length variation has influence on model pile stress.
	07#	90	30		
	08#	110	50		
	09#	120	60		

Rock slope has two layers including weakly weathered rock and strongly weathered rock from bottom to top, in which their physical mechanical characteristics have great differences. Two types different mixtures including gypsum, cement, sand

and early strength accelerator were shown in Table.2, which was adopted to simulate as rock mass and components of mixture

Table.2 The amount of mixture

Rock mass	Volume (m^3)	Density (g/cm^3)	Weight (kg)	Gypsum (kg)	Cement (kg)	Water (kg)	Sand (kg)	Early strength accelerator (kg)
1	2.131	2.1	4475.52	389	196	582	3308	0.98
2	0.834	2.1	1752.91	62	119	254	1319	0.60
Summation			6228.43	451	315	836	4627	1.58

Tests were operated in a model box, which was divided into five independent regions with a wooden separator (to prevent thrusting load from influencing adjacent model piles). The loading method included vertical loads and thrusting loads on slope top, which simulated weight of construction service roadway and severe construction equipments in construction stages. The loading device adopted to inflict vertical load was placed on model pile top through spherical hinge. The loads layout is shown in Fig.1.

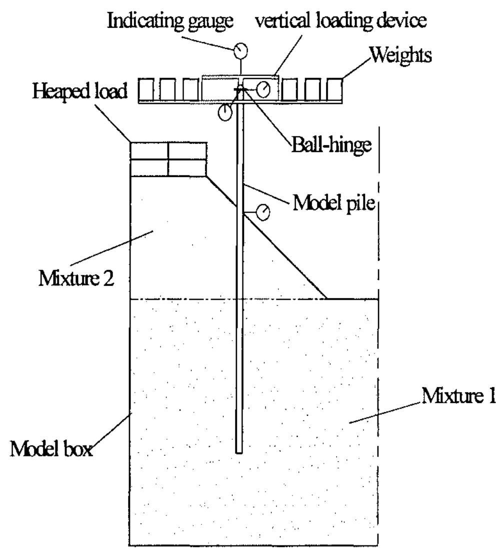


Fig.1 Loads layout

Both of pile top settlement and horizontal displacement of model pile were measured through dial indicator whose location was shown in Fig.2. Moment of pile shaft could be calculated by strain of pile shaft and flexural stiffness. In the process of loading, the distribution of thrust and resistance by rock layer around pile has been measured by earth pressure cells which were mainly embedded around NO.03# and NO.07# model pile. The layouts of the earth pressure cells are shown in Fig.2.

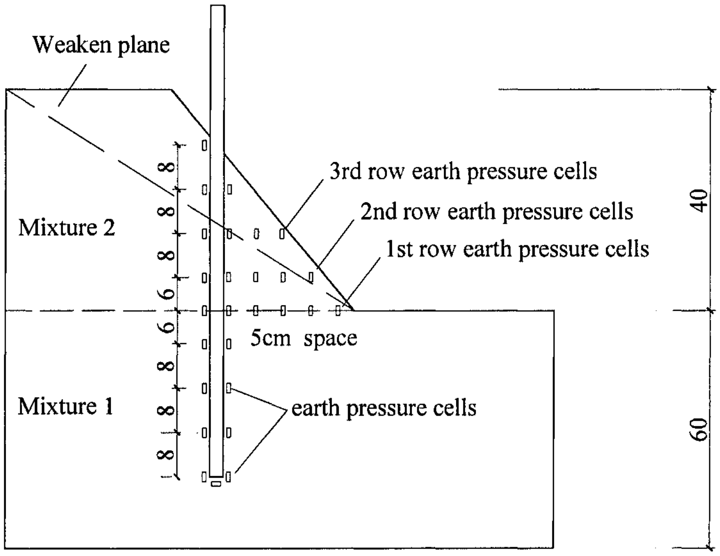
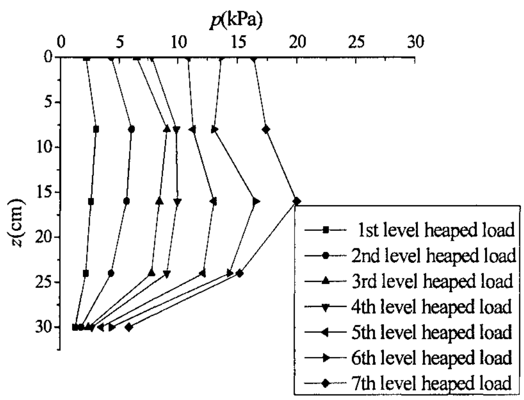


Fig.2 Layout of earth pressure cells

Slow maintenance loads method was adopted in vertical loading process. When two successive reading date variations, including vertical displacement and horizontal displacement, both less than 0.01mm, next loading would be applied. According to the references by Rase, rock around model pile occurred plastic failure when displacement of pile top reached 0.1 times diameter of model pile, and loading would be stopped.

ANALYSIS FOR TEST RESULTS

Thrust distributions of slope were measured by earth pressure cells around model pile and showed in Fig.3 and Fig.4 in each thrusting loads action.



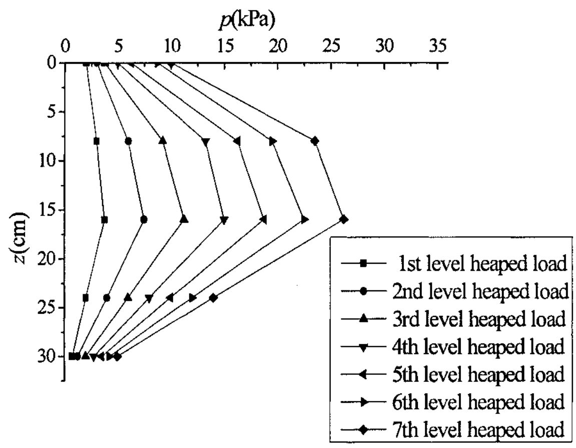
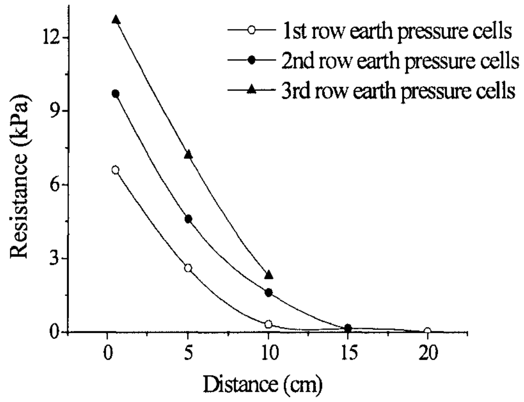


Fig.3 Thrust distributions on NO.03# pile **Fig.4 Thrust distributions on NO.07# pile**

Because deformation occurred on model pile shaft in ground surface, thrusts of slope in ground surface were comparatively small and increasing gradually from top to bottom. Thrust decreased on weak plane for friction existence between slope and weak plane.

Because of slope existence, resistance distribution was different with pile in flat ground. Because of slope free surface effect, infinite boundary doesn't exist beside pile shaft. Rock around model pile was not a symmetrical structure and resistance of pile shaft was not consistent with common pile. Slope resistance shown in Fig.5 and Fig.6 and basic regularity of slope resistance was as follows:



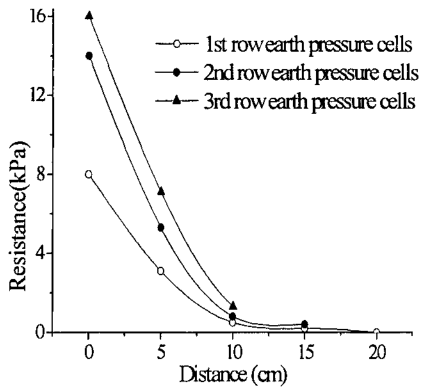


Fig.5 Resistance distributions on NO.03# pile Fig.6 Resistance distributions on NO.07# pile

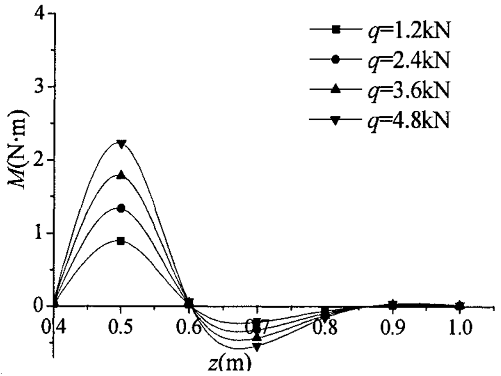
Resistance of rock around pile shaft decreased with distance increasing and earth pressure variation was small when horizontal distance exceeded 10cm. When horizontal distance exceeded 20cm, the earth pressure cells reflected weakly. Slope resistance was small because of slope free surface existence. Slope resistance was fully brought into play when slope thickness reached 10cm~15cm (just four times or six times diameter of pile), and influence of slope free surface was relatively small exceeding that thickness.

Measured moment of model pile shaft in Group A tests under different load levels are shown in Fig7~Fig.10, and basic regularity of model piles shaft were as follows:

(1) Moment of pile shaft only under heaped loads on slope top is shown in Fig.7. Because of deficiency constraints on pile top, the moment in pile top was small and moment distribution type of pile shaft was different with other three types loading method. General moment of pile shaft was small only under heaped loads condition.

(2) The maximum of pile shaft moment occurred in 0.5m scope under pile top. With buried depth increasing, moment of pile shaft decreased rapidly. When moment of pile shaft reached zero, reverse moment occurred. Reverse moment decayed zero at last with buried depth.

(3) Moment of pile shaft only under vertical load on pile top as shown in Fig.10. Because of slope free surface existence, rock around model pile was not symmetric semi-infinite body. Resistance of rock has been influenced by slope free surface and moment of model pile shaft occurred only under vertical load lonely action.



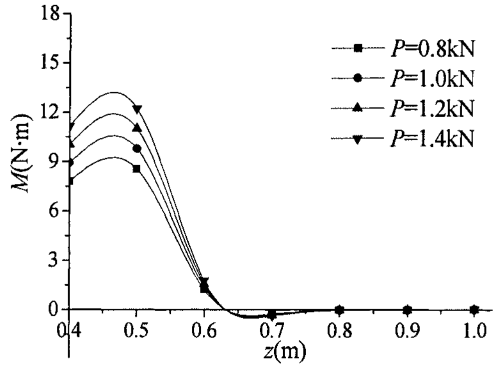
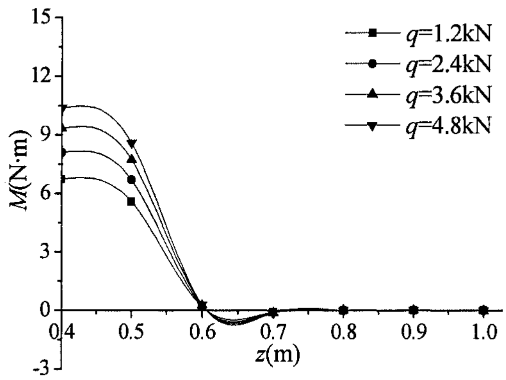


Fig.7 Measured moment of NO.01# pile Fig.8 Measured moment of NO.02# pile



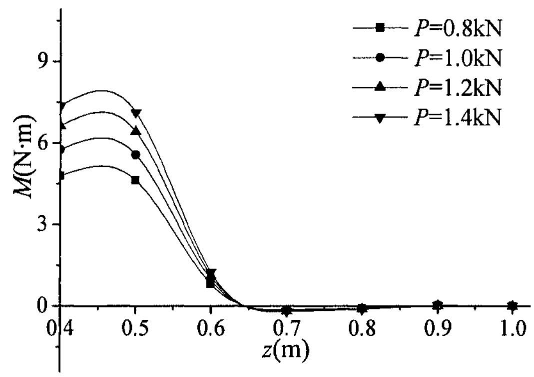
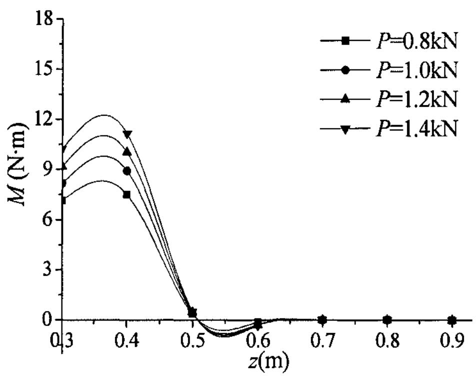


Fig.9 Measured moment of NO.03# pile Fig.10 Measured moment of NO.04# pile

The loads applying method was same in Group B which all firstly heaped load to 4.8kN, then applied vertical load. Piers lengths on model pile were respectively 0.3m~0.7m and stress of model pile shaft in slope was researched with different piers length. The measured moment of model pile shaft under different load levels in Group B is shown in Fig11~Fig.14. Basic regularity of model piles shaft were as follows:

(1) Stress distribution regularity of model pile in rock slope with different piers length was basically consistent.



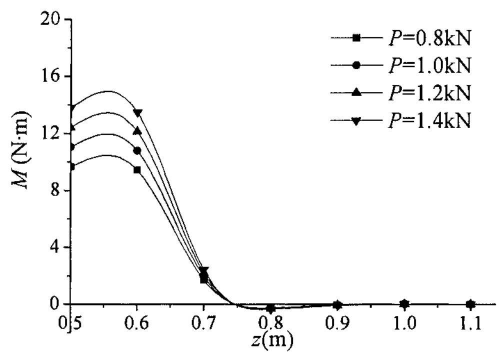


Fig.11 Measured moment of NO.06# pile Fig.12 Measured moment of NO.07# pile

(2) With piers length increasing, moment of model pile shaft has great increasing. When piers length increased from 0.3m to 0.7m, the maximum moment of model pile shaft increased to 130%.

(3) P-Δ effect of bridge pile foundation in slope was indispensable. Because of slope free surface existence, P-Δ effect of pile in slope was greater than in flat ground and long piers should not be adopted in bridge pier column design.

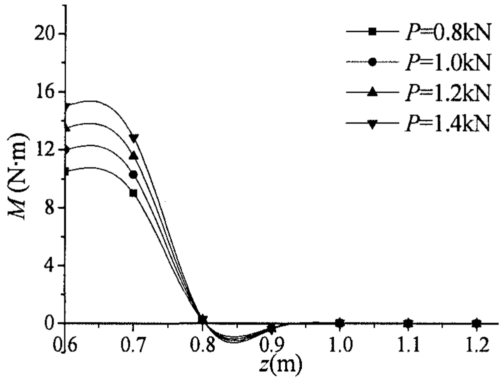


Fig.13 Measured moment of NO.08# pile

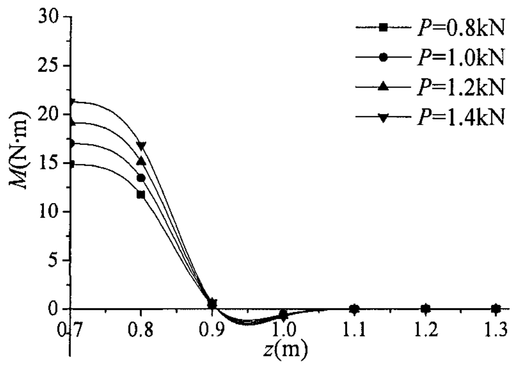


Fig.14 Measured moment of NO.09# pile

CONCLUSION

Model tests of bridge pile foundation were carried out and eight model piles of two groups in slope under vertical load and thrusting load were researched respectively. Thrust and resistance distribution of rock around pile in slope were discussed. The influence of model pile stress in different loading methods and different piers length condition were discussed. According to test results, the conclusion was showed as follow:

(1) The bridge pile foundation in slope has a double function that will not only bear load from a superstructure, but also counteract landslide-thrust from pile foundation side. Thrusts of slope in ground surface were comparatively small and increased gradually from top to bottom. Because of the friction existence between slope and weak plane, thrust decreased on weak plane. Slope resistance was small because of slope free surface existence. Slope resistance was fully brought into play when slope thickness reached four times or six times diameter of pile, and influence of slope free surface was relatively small exceeding that thickness.

(2) Lateral loads from thrusting loads on slope top have influence on pile shaft stress. Because of deficiency constraints on pile top, the moment in pile top was small and displacement in pile top was great. Because of slope free surface, rock around model pile was not a symmetric semi-infinite body. Resistance of rock has been influenced by slope free surface and moment of model pile shaft occurred only under vertical load respectively action.

(3) Because of slope free surface, piers column length increasing would greatly increase moment of pile shaft and P-Δ effect of pile in slope was greater than in flat ground. Long piers should be avoided to adopt in bridge pier column design.

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Influences of Construction Methods and Working Mechanisms on Stress Distribution in Deep Foundation Design

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ABSTRACT: In foundation design, stress distribution is crucial for determination of bearing capacity and settlement. Stresses are generally calculated using elastic models proposed by Boussinesq and Mindlin which mainly consider foundation size and depth. In fact, stresses are also dependent on construction methods and mechanisms of stress distributions, but these factors are rarely considered in engineering design. This paper used the FEM to simulate the construction method and analyzed stress distributions in diaphragm wall and open caisson foundation. The study showed that the additional stress under a caisson foundation is always greater than that beneath a diaphragm wall foundation, thus showing the importance of the effect of construction method on stress distribution. It was also found that Mindlin's solution is more appropriate than Boussinesq's solution for calculation of total settlement of deep foundation even though the latter may provide better stresses near the bottom of the foundations. In addition, the settlement obtained using Mindlin's approach is between the settlement of caisson foundation and diaphragm wall foundation, and the discrepancy goes up with an increase in foundation depth.

INTRODUCTION

Settlement under a foundation is generally determined by summing the settlement of layers with uniform or varying thickness. The reason for dividing the soil into layers is that the net pressure available to cause consolidation settlement reduces with depth. The reliability of this method depends solely on the precision in determining the additional stress beneath the foundation. The Boussinesq solution (1885) for a point load applied on the ground surface is usually integrated to determine the stresses within a semi-infinite half space. On the other hand, Mindlin's solution (1936) is utilized to determine the stress in a semi-infinite space under point load applied beneath the ground surface. Since the Boussinesq solution is a particular case of the Mindlin solution, the latter has a much wider field of application and a lot of research has been done to get their characteristics and application. Wang and Jia (2006), compared in situ data with vertical stress of a circular foundation and concluded that the Boussinesq method is not suitable for deep foundations. It is commonly accepted that the Mindlin solution is more applicable in computation of settlement of deep foundations. However, the influence of construction methods and working mechanisms on stress distribution in deep foundation design has rarely been studied. This study based on FEM computation found that settlement obtained with Mindlin condition is smaller for caisson foundation but greater than that of diaphragm wall foundation, and this trend goes up with an increase in foundation depth. This proved that construction methods and working mechanisms of foundations are major factors that affect stress distribution and should be considered during analysis of foundation settlement calculation.

COMPARISON OF BOUSSINESQ SOLUTION AND MINDLIN SOLUTION

Skopek (1961) gave the solution for vertical stresses beneath corners of rectangular area by integrating the Mindlin point load. The stress at other point within the mass may be obtained by using superposition. This solution can also be used to calculate stress in semi-infinite space with rectangular load applied on ground surface if takes the foundation depth as zero. Stress distribution under the center point of square foundation is shown in figure 1, where z represents the distance beneath foundation, and b, h represent the length and depth of the foundation.

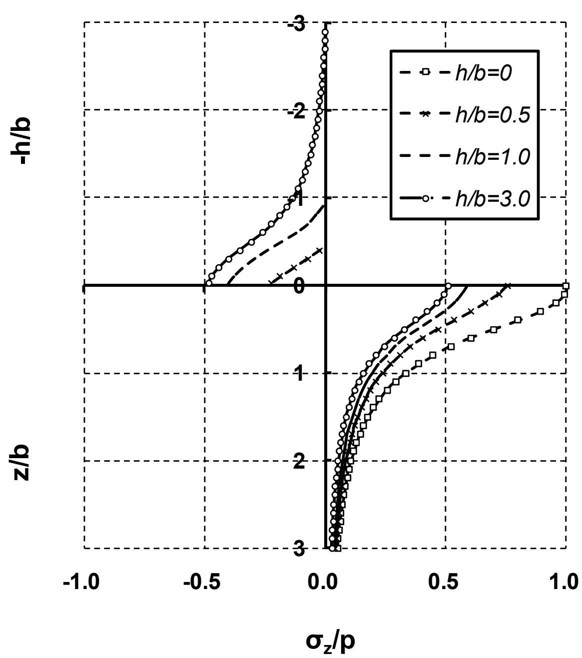

FIG. 1. Additional vertical stress calculated through Mindlin equation.

Figure 1 shows that:

(1) Mindlin solution always results smaller stress compared with Boussinesq solution. The discrepancy increases with depth, and it reaches almost 50% when the depth /width ratio equals to 3.

(2) Tensile stress exists above the base of foundation, which will never happen in real foundation and will introduce some errors into stress calculation.

CAISSON AND CLOSED DIAPHRAGM WALL FOUNDATION

Box-shaped deep foundation such as caisson and closed diaphragm wall may be constructed by different construction methods. They may cause variations in soil-structure interaction and consequently in additional stress distribution beneath foundation.

Open caisson foundation is constructed on ground surface and then sunk into ground. Sidewall friction may reach the peak and then drop to residual value because of large displacement during sinking. After construction, the sidewall friction cannot increase even the caisson foundation carries load from superstructure. If further consolidation of soil around the caisson is negligible, all the overburden load is passed to subgrade, as illustrated in figure 2.

Closed diaphragm wall foundation is constructed in deep trench cut, friction between soil and wall is quite small during construction. When foundation is loaded from superstructure, the downwards movement leads to increase in sidewall friction until it reaches the peak and finally reaches residual value if settlement is high enough. In either case, sidewall friction shares part of the load while the rest of the load is undertaken by subgrade as illustrated in figure 3.

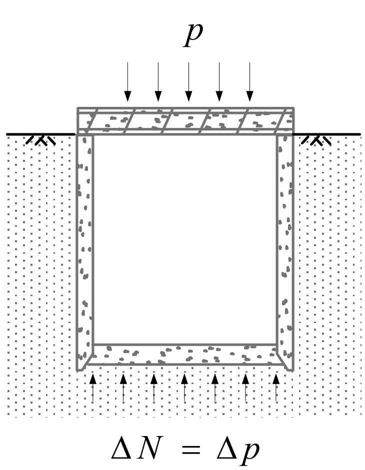

FIG. 2. Forces acting on caisson.

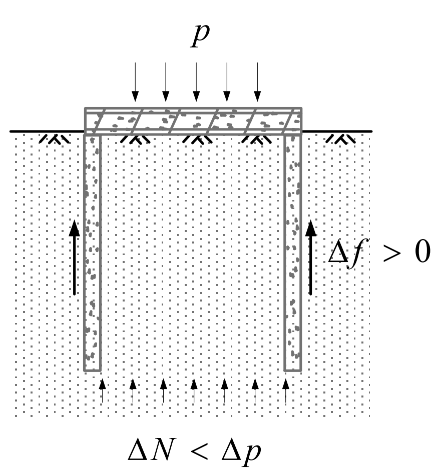

FIG. 3. Forces acting on diaphragm wall.

Figure 4 presents the developments of sidewall frictions of the two types of foundations during the process of construction. Here abscissa represents time, t_1, t_3 represent the ending time of foundation construction and superstructure construction