temperature difference between day and night or the one experiencing cross seasonal construction. The reinforcement amount of circle beam support should be increased to resist the adverse effects of the axial force increasing with temperature raising and the bending moment increasing with temperature decreasing. During construction process, on the other hand, the support can be protected against the sun or be drenched with water to lower the temperature of support with the arrival of summer while the warm-keeping measures, such as covering, should be taken with the arrival of winter to ensure the safety of excavation and surrounding environment.

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Finite Element Analysis on the Influence of Unloading Effect and Rebound Effect on Load and Settlement of Single Pile

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ABSTRACT

At present, the effect of deep excavation on piles about the force and deformation had been paid more and more attention, but the mechanism of deep excavation on load transfer and settlement of pile was not very clear. Using ABAQUS software, a two-dimensional axisymmetric finite element model, considering pile length, excavation depth, and over consolidation, was modeling under three different conditions to analyze the influence of unloading effect and rebound effect on the engineering piles stiffness. Comparative analysis showed that vertical stiffness loss of pile was mainly caused by rebound effect before reaching inflection point load and the rule was all the same as pile length changed. Then, unloading effect gradually becomes the main cause of vertical stiffness loss. In addition, when the size of pit reached certain limit value, continuing to increase the size had less influence on the stiffness and ultimate bearing capacity of pile. The over consolidated effect significantly increased the vertical stiffness of pile.

KEYWORDS: unloading effect; rebound effect; pile; foundation pit; FEM

1 INTRODUCTION

The development of underground cavity in Tianjin was very rapidly in recent years and the depth of pit engineering becomes deeper and deeper. Many engineering examples reflected the problem of the influence on engineering pile with the excavation of pit and it was distracted more and more scholars' attention.

Iwasakietal (1994) found that with the increase of excavation depth, counterforce on the pile head and axial force of pile body continued to increase, the pile tensile stress appears with the increase of excavation depth by monitoring the changes of axial force on the excavation pile body; Zhu Huogen (2005) introduced that pile was pulled in pit engineering because of rebound of foundation pit in the Shanghai accident. The pit was 13 m deep and effective pile length was 30~37 m long, reinforcing cage was 13 m long. After the pit was finished, the low strain dynamic test found that 30% of piles broken at the bottom of the reinforcing cage. Chen Xiaoxian (2006) reported a foundation pit engineering in Xiamen, the pit was 7 m deep and the water level was decrease to 1 m under the bottom of pit, the 25% of engineering pile occurred fracture when excavation and foundation pit finished because of the pit rebound. Mao-song huang (2007), analyzed the influence of deep excavation on uplift piles include constant cross section and pedestal pile by the method of finite element (FEM). Some papers studied the mechanical characteristics of engineering piles by the finite element method and model test. Zheng Gang (2009) studied the mechanism of load transfer and settlement of the pile loaded after deep excavation with FEM, but without considering the consolidation of soil, pile length, the influence of other factors such as the width and depth of pit. In this paper, these factors and the excavation unloading and the rebound effect was coupled, further revealed the load transfer and settlement mechanism of pile in the deep excavation.

2. INTRODUCTION OF THE FINITE ELEMENT MODEL

2. 1 Finite element model

In the finite element analysis, a two-dimensional axisymmetric model was created to simulate the behavior of the excavation using the commercial finite element software, Abaqus. Entity element was used to simulate the pile, soil and the retaining wall. Four-node axisymmetric reduced integral element (CAX4R) was used for the pile and the retaining wall, while the four-node axisymmetric element (CAX4) was for the soil. The struts adopt the two-node linear axisymmetric element (CAX1). As shown in Figure 1, The pit was 40 m wide with the excavation depth of 20 m. Retaining wall was 40 m long and 1.0 m thick. Inner pile was 40 m long and 1.0 m in diameter and installed below the bottom of the pit. Retaining wall and pile were constructed prior to excavation. The soil layers were extended to 140 m deep, which was 80 m from the bottom of pile to the bottom boundary and was approximately 2 times length of the pile. The horizontal distance from the margin of the pit to the vertical of the model was 120 m, which is approximately 6 times half-width of the excavation. It was generally believed that when the size of the numerical model was 5 times larger than the loading area, the boundary effect can be ignored. This approach was also used in this study. Therefore, the influence of the bottom and horizontal boundaries on the numerical results can be ignored. The mechanical boundary conditions were set as follows: on the left and right sides, the horizontal displacements were set to zero while the bottom boundary was fixed in horizontal and vertical directions.

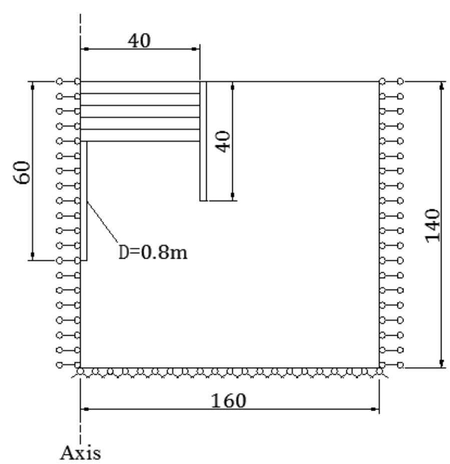


Figure 1. Model of finite element method.

2.2 Material properties

The properties of all the materials used in the numerical model were provided in Tables 1. The master-slave surface-surface contact without thickness and penalty function algorithm were adopted to simulate pile–soil and retaining wall-soil. The Coulomb frictional law was implemented in interface modeling and friction coefficient of 0.35 was adopted according to Ou C. Y． and Roscoe KH. The piles, struts and the retaining wall adopted the linear elastic material and Elastic modulus of 3×10^4 MPa and actual density of 2500 kg/m^3 were used to define the mass property of concrete. The soils were modeled as Modified Cam–Clay (MCC) materials. The MCC model included five material parameters: slope of swelling line, κ; slope of virgin consolidation line, λ; the void ratio, e; slope of the critical state line, M; and Poisson's ratio, v. The values of λ, κ, and e were obtained from one-dimensional consolidation tests. The value of M was obtained from the triaxial consolidated undrained tests with measured pore water pressure. The permeability values of the soils were assumed to be consistent with the empirical values in the Tianjin area, and their values were assumed to be equal in vertical and horizontal directions in the numerical analysis.

Figure 2 shows the measured and computed curves of Q-S. It is shown that the computed results agree reasonable well with the measured data. Therefore, the finite element model and parameter mentioned above provide a reliable way to investigate different working conditions

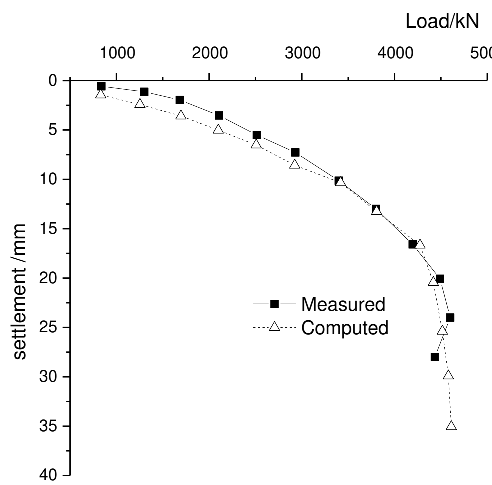


Figure 2. Comparison of results from computer and measured.

Table 1. Parameters of Subsoil

Layer	Soil type	Depth /m	Unit weight kN/m^3	λ	κ	M	e
1	Fill	2.1	19.1				
2	Mucky clay	6.2	18.7	0.089	0.013	0.43	0.9
3	Silty clay	10	20.2	0.076	0.0064	0.46	0.8
4	Mucky clay	16.3	17.9	0.0792	0.0065	0.46	0.85
5	Silty clay	24.3	20.6	0.032	0.0041	0.91	0.72
6	Silty sand	51.5	20.1	0.0221	0.0035	1.21	0.62
7	Silt	55	20.3	0.0267	0.0038	1.19	0.64
8	Silty clay	69.2	21.1	0.0334	0.0048	0.80	0.75

2. 3 Working conditions

In order to compare the effect of unloading and rebounding on the pile, three kinds of methods of pile tests were designed with controlling variable method. The first kind of methods of test pile was with sleeve(PTS), which added casing to isolate the friction of the pile and soil within the scope of excavation, and then loading. (see Figure 3(a)). The second method was carrying out pile test at the pit-bottom(PTPB), which reactive the pile after finishing the excavation and then loading at the bottom of the pit. (see Figure 3(b)). It could eliminate the effect of the rebounding of soil which made the whole pile be in tension and determined alone the effect of the unloading of soil on the rigidity of the pile. The last pile test was conducted while excavating(PTE). Compared with the second way, it could determine alone the effect of soil resilience on the rigidity of pile. (see Figure 3(c)).

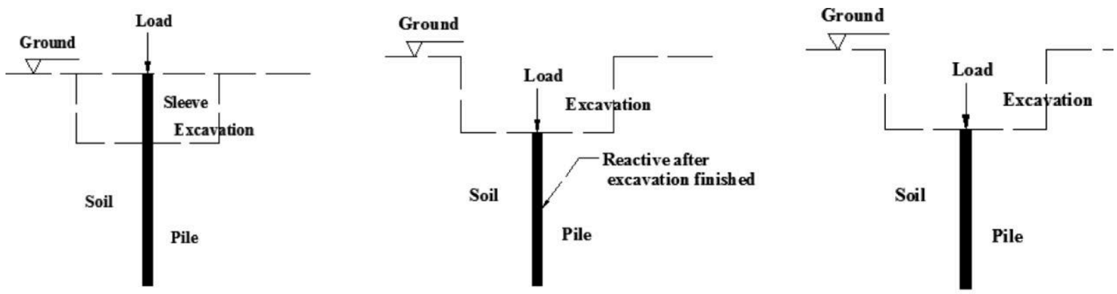


Figure 3. Plots of different methods.

3. RESULTS OF DIFFERENT CASES

3.1 Comparison of Q-S curves

As shown in Figure 3, there was three curves of Q-S by different kind of pile tests. Because of loading on the top of pile which flushed with the ground, the settlement of the PTS deducted the amount of pile compression within the length of excavation. The ultimate bearing capacity could be defined as the load corresponding the settlement which up to 0.1 times pile diameter and the knee point load could define as the load corresponding the start point in the period of the sharp drop at the Q-S curve.

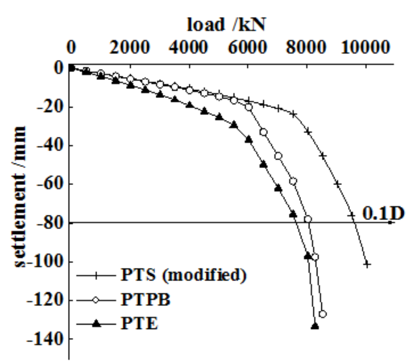


Figure 4. Q-S curves of different methods.

It could be found from the figure 4 that, the Q-S curves of PTS and PTPB were very close before the inflection point load, as a result, the unloading effect on the stiffness loss of engineering pile didn't do too much. At the same time the knee point load of the PTPB's Q-S curve was same as the PTE's, and the settlements corresponding the knee point load reached to maximum between both. Conclusion could be drawn that, before reaching the knee point load the vertical stiffness loss of the pile mainly derived from the soil resilience. After the knee point load, with the increase of the loading on the top of pile, the resilience effect wore off and the unloading effect grown up as the primary reason of the vertical stiffness loss of piles.

Compared the Q-S curves of PTPB and PTE, the stiffness loss of the PTPB reached the maximum value, in other words, the soil resilience because of excavation on the stiffness loss had the biggest impact. Compared to the PTS, the ultimate bearing capacity was reduced by 16.5%, the knee point load was reduced by 20%. And so that the unloading effect and resilience effect decreased the vertical stiffness of pile obviously.

3.2 Impact analysis of soil unloading and rebound on pile's stiffness

3.2.1 Impact of different pile length

The Figure 5 presented Q-S curves of three kind of pile tests at different pile length case. The marked pile diameter was 0.8 m, the length of the pile was 30~70 m (added a model per 10 m). The knee point load(KPL), the settlement corresponding the knee point load(SKPL) and the ultimate bearing capacity(UBC) of three kind of pile tests at different pile length cases were shown in the Table 2. The unloading effect and soil resilience effect could be studied from the fig and the table at different pile length on the stiffness and ultimate bearing capacity of pile.

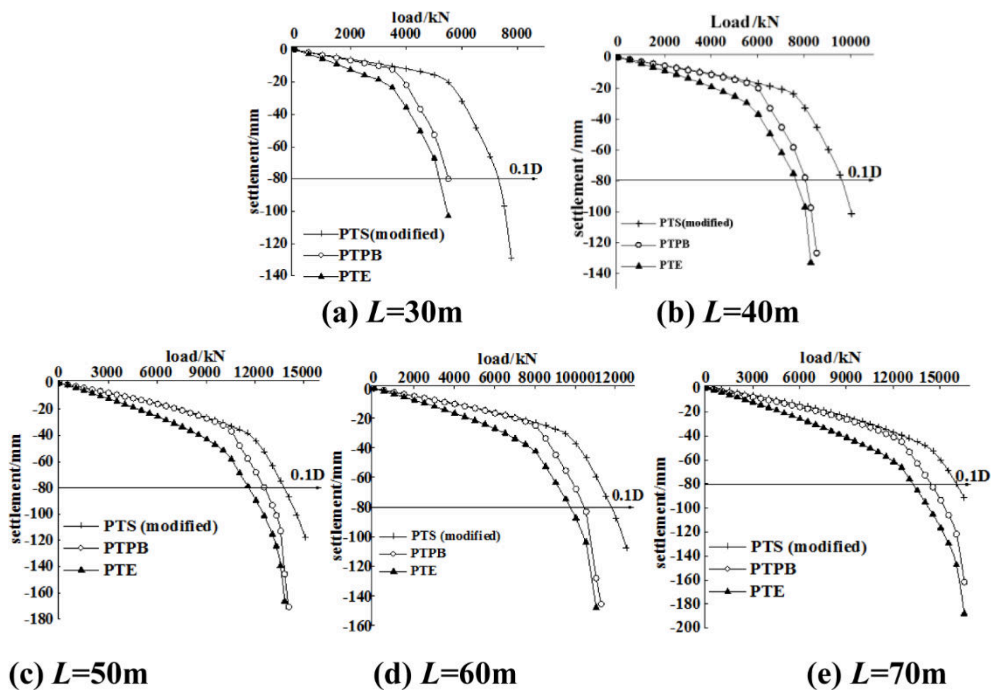


Figure 5. Q-S curves of different pile length.

Under the standard excavation condition (R=40 m, H=20 m), the Q-S curve of PTE was same as PTS's basically before reached the KPL with the change of pile length. And the KPL of PTE was same as PTPB's basically. So with the increase of the pile length, the vertical stiffness loss mainly derived from the soil resilience before the KPL. And then the unloading effect would be primary reason of the loss of ultimate bearing capacity and vertical stiffness of pile.

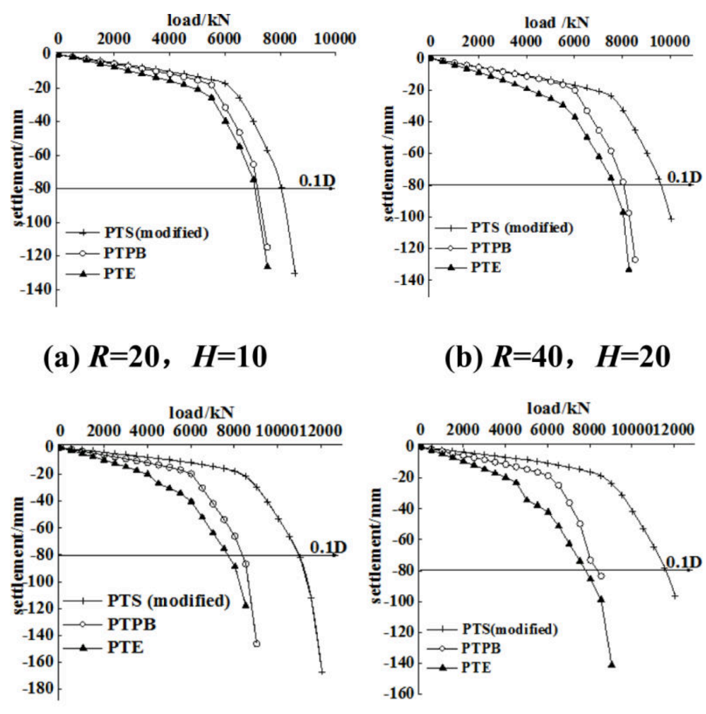
With the change of the pile length, the KPL remain equal between PTE and PTPB, but the settlement corresponding the KPL respectively increased by 90.3%、84.4%、70%、58.2%、43.8%. So the effect of the resilience decreased the vertical stiffness of pile obviously, and with the increase of the pile length, the effect of the soil resilience gradually decreased.

Table 2. Impact Comparison of Different Pile Length

Length L/m	KPL/kN			SKPL/mm			UBC/kN		
	PTS	PTPB	PTE	PTS	PTPB	PTE	PTS	PTPB	PTE
30	5526	3517	3517	20.28	12.34	23.48	7310	5526	5300
40	7536	6029	6029	23.83	20.16	37.02	9658	8062	7680
50	9546	8038	8038	30.35	25.02	42.59	11807	10450	9750
60	12058	10048	10048	44.11	32.24	51.4	13778	12560	11618
70	14570	12560	12560	52.81	44.79	66.42	16077	14444	13364

3.2.2 Influence of the different width and depth of the pit

Figure 6 presented different *Q*-*S* curves of different types of standard excavation(R/H=2). Four models were built, and the depth of the excavations respectively were 10 m, 20 m, 30 m, 40 m. The knee point load(KPL), the settlement corresponding the knee point load(SKPL) and the ultimate bearing capacity(UBC) of three kind of pile tests at different pile length cases were shown in the Table 3.



(c) R=60，H=30 (d) R=80，H=40

Figure 6 Q-S curves of different pit sizes.

As shown in Figure 6 and Table 3， the ultimate bearing capacity .KPL and settlement corresponding KPL stay the same when the size of the excavation was over R=40 m， H=20 m. It shown that the effect of the pit size on the vertical stiffness and the ultimate bearing capacity was small.

Table 3. Impact Comparison of Different Pit Sizes

Size（m）: R/H=2	KPL/kN			SKPL/mm			UBC/kN		
	PTS	PTPB	PTE	PTS	PTPB	PTE	PTS	PTPB	PTE
R=20,	6028	5526	5526	17.46	18.27	26	8402	7181	7085
R=40,	7536	6029	6029	23.83	20.16	37.02	9658	8062	7680
R=60,	8541	6029	6029	21.48	19.47	40.35	10984	8357	7707
R=80,	8541	6029	6029	18.54	18.72	42.27	11605	8368	7800

3.2.3 Influence of the soil overconsolidation

The soil at the bottom of the pit was in state of the over-consolidated, but the finite element software only provided initial normal consolidation coefficient without considering the factor of the soil overconsolidation. In order to study the influence of the soil overconsolidation, the soil was divided into 8 layers. Over-consolidation ratio was calculated according to the following formula using the variation of effective stress located in neutral position of each layer. (α=0.56).

$$OCR = \frac{\sigma'_{v\max}}{\sigma'_v} \tag{1}$$

$$K_{0(OC)} = K_{0(NC)} \cdot OCR^{\alpha} \tag{2}$$

Table 4 presented the OCR and coefficient of earth stress at rest (K_0)

Table 4. OCR and Earth Stress Coefficient of Each Layer

Layer	Buried depth before /m	Buried depth after /m	$\sigma'_{v\max}$ /kPa	σ'_v /kPa	OCR	$K_{0(OC)}$
1	25	5	265	53	5	1.33
2	35	25	371	265	2.33	0.86
3	45	35	477	371	1.8	0.75
4	55	45	583	477	1.57	0.69
5	70	55	742	583	1.4	0.65
6	90	70	954	742	1.29	0.62
7	110	90	1166	954	1.2	0.6
8	130	110	1378	1166	1.18	0.59

Figure 7 presented the comparison of the earth stress coefficient between overconsolidation and normal consolidation. The KPL of the PTE was 6028.8 kN and the UBC was 8038.4 kN by adopting the overconsolidation earth stress coefficient. The KPL of the PTE was 5526.4 kN and the UBC was 7680 kN by adopting the normal consolidation coefficient. The KPL and UBC of the pile respectively decreased by 8.3%、4.5% with normal consolidation earth stress coefficient . This is due to that using the over-consolidated soil pressure coefficient increased the horizontal stress level of the pit soil, which in turn increased the normal stress of pile. So, the vertical deformation stiffness and ultimate bearing capacity of pile had improved. Figure 8 presented the comparison of the Q-S curves between PTS and PTE considering the soil overconsolidation effect. The loss of UBC decreased by 16.8% because of excavation. And the loss of UBC decreased by 20.5% without considering the soil overconsolidation effect.