Numerical and Theoretical Study of Plate Load Test to Define Coefficient of Subgrade Reaction

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Abstract

One of the important parameters required to evaluate the behavior of soils under loading condition is coefficient of subgrade reaction ($K_s$), which is being used widely to determine the kind of favorable foundation. There are many theoretical and laboratory approaches that released some relations to achieve the value of $K_s$. One of the most effective and fastest in-situ procedures to find $K_s$ is plate load test (PLT). In this test a plate with 30 to 45 cm diameter is loaded through incremental multi-stage and the corresponding soil settlement is monitored stage by stage. In recent years numerical methods have been significantly used to simulate some geotechnical tests. This paper presents the three dimensional simulation of PLT, investigated by using finite element code, and compares the results obtained from site studies with the results of the numerical modelling. During verification, it was found that a constant number must be used to be multiplied in the modulus of elasticity as an input of finite element code (ABAQUS). This constant is estimated to be 3. Then, the obtained constant was used to estimate the $K_s$ of another site. The results show that the relation has sufficient accuracy in soft soils but it cannot be reliable in coarse one. Also, the goal of this research was to explore the ability of numerical modeling to evaluate the value of $K_s$ without performing some plate load tests, was achieved.

Keywords: Numerical analysis, plate load test, settlement, coefficient of subgrade reaction.

1. Introduction

Ground-foundation interaction has been one of the challenging issues in geotechnical engineering since late nineteenth century. Because of the complexity of soil behavior, subgrade in soil-structure interaction problems is replaced by a much simpler system called subgrade model. One of the most common and simple models in this context is Winkler hypothesis. Winkler idea represents the soil medium as a system of identical but mutually independent, discrete and linearly elastic springs and ratio between contact pressure, $P$, at any given point and settlement, $y$, produced by it at that point, is given by the coefficient of subgrade reaction ($K_s$). Geometry and dimensions of the foundation and soil layering are assigned to be the most important effective parameters on $K_s$.

Terzaghi (1955) studied at $K_s$ and stated that the theories of vertical and horizontal subgrade reaction are based on the simplifying assumptions that the subgrade obeys Hooke's law, and that the subgrade reaction on the base of a rigid centrally loaded plate resting on the horizontal surface of the subgrade has the same value at every point of the base. He made some recommendations where he suggested values of $K_s$ for 1×1 feet rigid slab placed on a soil medium; however, the implementation or procedure to compute a value of $K_s$ for use in a larger slab was not specified [1]. Vesic (1961) tried to develop a value for $K_s$, by matching the maximum displacement of the beam in both aforementioned models. He obtained the equation for $K_s$ for using in the Winkler model [2]. Biot (1973) solved the problem for an infinite beam with a concentrated load resting on a three dimensional (3D) elastic soil continuum. He found a correlation of the continuum elastic theory and Winkler model where the maximum moments in the beam are equated [3]. Coefficient of subgrade reaction at every point under the foundation obtains from relation (1):

$$K_s = \frac{q}{y}$$  

Where $q$ is amount of applied stress in kN/m² and $y$ is the obtained settlement in meter. Winkler model was suggested for rigid plates, at first, then it was developed to calculate the stresses under flexible foundations [4-8].

Studies about $K_s$ refers to the past few years that some software programs show the ability to model soil settlement and estimate the amount of $K_s$ [1,9]. Ziaei and Janbaz (2009) studied at effective parameters on modulus of subgrade reaction in clayey soils through PLT. They declared that by increasing the depth of embedment of foundation, the $K_s$ increases. Also, flexural rigidity
of foundations could improve the status of subgrade reaction modulus. They set 3D FE model by means of Plaxis code and investigated effects of shape of foundation, depth of embedment of foundation, rigidity or flexibility and size of foundation. The results of these simulations showed that:

a) $K_s$ is decreased as the side dimension of foundation is increased.

b) As the consistency of clayey soil is decreased the difference between obtained modulus and Terzaghi one is increased.

c) With constant width and load intensity, $K_s$ in strip foundation has the lowest value while the square foundation has the highest value.

d) Terzaghi relation for estimating $K_s$ of rectangular foundations is not recommended for strip foundations [10].

Marto et al. (2012) studied the effect of size of foundation on sandy subgrade using finite element (FE) software (Plaxis, V.8.2) to investigate the validation of Terzaghi’s formula on determination of subgrade reaction modulus. Also the comparison between Vesic’s equation, Terzaghi’s one and obtained results were presented. They found that the modulus of subgrade reaction is decreased as the side dimension of plate increased. This was due to increasing load area, which consequences in increasing settlement. Moreover, they figured out that the effect of water in the soil is very significant, and $K_s$ obviously has larger values in dry soil. So, this effect should be very significant until 10 times of plate dimension [11].

In this study, the soil stiffness modulus has been calculated by using a finite element code, and then, the obtained results has been compared with the in situ results. The result validation has been done by using PLT data from combined cyclic power plant of Hormozgan and co-utility project of Qeshm Island. In fact, this paper does not need to investigate older researches and the main attempt is feasibility study of plate load test (PLT) simulation and to gain proper output by some simplification, which make the finite element code useful. The minor goal is to approximate $K_s$ of a site without doing plate load test.

2. Computational examples and analysis

2.1. Site specification

Combined cyclic power plant of Hormozgan is near mountain Geno and Persian Gulf that is mainly consist of grain size levee. Another Site is co-utility project of Qeshm Island in Iran, situated in the vicinity of Luft seaport by silty clay soil with or without sand. This study benefits from the data of Hormozgan site for initial estimation and uses the data of Qeshm Island for validating the estimation. The specification of soils, test as plate load test are shown in Tables 1 and 2. As an example, TP1 is the type of soil, introduced by number 1.

| Table 1: Mechanical specification of Hormozgan site |
|-----------|-----------|-----------|-----------|-----------|
| PLT location | TP1 | TP2 | TP3 | TP4 |
| $\gamma$ (Kg/cm$^3$) | 2 | 2.12 | 1.89 | 1.83 |
| $C$ (Kg/cm$^2$) | 0.1 | 0.1 | 0.1–0.3 | 0.1–0.3 |
| $\phi$ (degree) | 32–34 | 32 | 26–28 | 27–29 |
| $\nu$ | 0.3 | 0.3 | 0.35 | 0.35 |
| $E$ (Kg/cm$^2$) | 470 | 215 | 210 | 150 |
| Classification | GP-GM | GP | Clay | Clay |

By comparison of two groups of data, attained from Hormozgan and Qeshm site, it is noticeable that the elasticity modulus ($E$) of Table 1 generally are higher than corresponding parameter of Table 2.

| Table 2: Mechanical specification of Qeshm island site |
|-----------|-----------|-----------|-----------|-----------|
| PLT location | TP1 | TP2 | TP3 | TP4 |
| $\gamma$ (Kg/cm$^3$) | 1.95 | 1.95 | 1.95 | 1.95 |
| $C$ (Kg/cm$^2$) | 0.2 | 0.2 | 0.2 | 0.2 |
| $\nu$ | 0.35–0.4 | 0.35–0.4 | 0.35–0.4 | 0.35–0.4 |
| $E$ (Kg/cm$^2$) | 80 | 110 | 95 | 120 |
| Classification | CL-ML | CL-ML | CL-ML | CL-ML |

2.2. Procedure and Methodology

The Philosophy of PLT simulation is to enable a computer program as a helpful and useful method to estimate the settlement of foundation when there is no access to in situ data. The procedure of this research to reach the mentioned goal is here:

Step1: Four numerical models were set to run by the same data of Hormozgan site as input data.

Step2: The results of model of step1 were compared with the results of in situ PLT and approximated which parameter should be changed to converge two group of results together.

Step3: Models were set to run by new changed data and were compared with the results of first step.

Step4: In the case of achieving the desired results, the changed parameter in step2 were applied on a new project to reassure about the software reliability.

Step5: Finally, the results of numerical modelling were compared with the results of some classical relations and were reported.

2.3. Numerical modeling

PLT has simulated in three dimensional finite element code (ABAQUS. V. 6.9) by following considerations [12]:

a) The model was built in axisymmetric type by free mesh which has been grown from the loading line to upper and lower boundary condition (Fig. 1). The elements has been chosen as a 4-node axisymmetric quadrilateral, bilinear displacement, bilinear pore pressure and as a pore fluid/stress element.

Fig.1. Mesh and geometry of model

b) Mohr Coulomb plasticity constitutive model was chosen.
c) Initial elasticity modulus \((E_0)\) and elasticity modulus of 50% strain \((E_{50})\) used to calibrate the model and to validate the results of numerical modelling with the results of site investigations.
d) Initial condition set to be drained.
e) A plate with 30×30 cm dimension was used and the depth of test set to 2 meter below the surface. Then height and diameter of model were considered 5 m and 1 m respectively.
f) Evaluating the settlement of plate under incremental load was done by catching average of settlements on the whole loaded area.
g) Proper boundary condition was set. Bottom of the model was closed in every direction. Sides of the model were closed in horizontal direction and were let it to settle down in vertical direction.
h) Loading of model was pressure type.
i) In this research, effects of soil-plate friction (interface) were neglected and the load applied to the surface of soil in borehole directly.
j) Eleven and seven models have been run for Hormozgan site and Qeshm site, respectively.

3. Results

The results show that the settlements obtained from numerical simulation are too high to be compared with the corresponding results of in situ PLT; because of changing in initial and surrounding condition such as in situ stress and different behaviour of soil in site. The result has been shown in Figs. 2 to 5.

It is clearly seen that internal friction angle of particles doesn’t change in the range of stress bulb. Also Poison’s ratio and specific weight of soil body do not change tangibly. According to the slow rate of loading and the level of the water table, which provides the drained condition; the parameter of cohesion \((C)\) remains constant through testing. The only effective parameter to converge the results of site and model should be elasticity modulus. By applying pressure and acceptance of settlement, the elasticity modulus of soil changes slightly.

With change in \(E_0\) of Hormozgan data, the model was run again and the results were derived as shown in Figs. 6 to 9.

It seems to be better to use a coefficient for \(E_0\) as an input of ABAQUS to gain better results. Also, it should be interpret that for such a cohesionless soil, there is not good match between numerical modeling and in situ test results. With respect to average of all applied coefficients for modulus of elasticity, it can be said that number 3 is proper to use as regulator coefficient. Of course the best range of this coefficient is between 2.5 to 3.25. Validating of software has been done by applying 3\(E_0\) for the data from Qeshm island site. The results have been shown in Figs. 10 to 13.

As can be seen in Figs. 10 to 13, by considering 3\(E_0\) as input value instead of \(E_0\); the obtained results from FE code are very close to and in some cases match the results of PLT on fine aggregate soils. It was seen that simulation of plate load test by using the mentioned code, generates better results (curvature and pattern) for cohesive soils than cohesionless one but the code should be calibrated for every project and then generalized.
The final step was allocated to compare the results of some classical formula with the real results and model one on the basis of Qeshm island results (validating model). Table 3 shows attained $K_s$ from three mentioned approach.

According to Table 3, the results obtained from ABAQUS, have better agreement and accuracy to the site investigation than the theoretical one for fine soils. This is because of following reasons:

1) Soil body for fine particles are more homogenous than coarse one. For this reason prediction of fine soils behavior is easier and more harmonious with the made model, assumed homogenous mass as soil.

2) Often, presence of coarse gravel and stone in cohesionless soils lead to change the results of site loading test.

3) There is a fraction between 0.85 to 1.15, represents the ratio of in situ $K_s$ to attained $K_s$ from the software, for cohesive soil whereas this ratio cannot be in a certain range for cohesionless soils.

4) The larger elasticity modulus of coarse particle in comparison with the elasticity modulus of fine particle, may be the main cause
of discrepancy of model results with investigation results; even after modification of \( E \) to 3\( E \).

Table 3: Comparison of obtained \( K_s \) from in situ test, Model test and Klopple’s and Selvadurai’s and Meyerhof’s classical equation.

<table>
<thead>
<tr>
<th>Location</th>
<th>TP1</th>
<th>TP2</th>
<th>TP3</th>
<th>TP4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site; ( E = E_0 )</td>
<td>5.39</td>
<td>5.64</td>
<td>5.38</td>
<td>7.20</td>
</tr>
<tr>
<td>Model; ( E = 3E_0 )</td>
<td>5.00</td>
<td>5.00</td>
<td>5.55</td>
<td>7.50</td>
</tr>
<tr>
<td>Selvadurai [13]; ( K_s = \frac{0.65E_0}{B(1-v_0)^2} )</td>
<td>2.00</td>
<td>2.76</td>
<td>2.38</td>
<td>3.00</td>
</tr>
<tr>
<td>Klopple [3]; ( K_s = \frac{2E_0}{B(1-v_0)^2} )</td>
<td>3.89</td>
<td>5.35</td>
<td>4.62</td>
<td>5.84</td>
</tr>
<tr>
<td>Meyerhof [3]; ( K_s = \frac{E_0}{B(1-v_0)^2} )</td>
<td>3.09</td>
<td>4.25</td>
<td>3.67</td>
<td>4.63</td>
</tr>
</tbody>
</table>

4. Conclusions

With the studies carried out on samples of PLT and simulation of the mentioned soils in FE code, the conclusions can be explained as below:

1. The results of numerical simulation of PLT on the basis of elasticity modulus of site, have significant difference with in situ results (about 400%). It has been shown that use of 2.5 to 3.25 of \( E_0 \) as an input parameter of software can reduce the error rate, up to 15% for cohesive soils.
2. It is necessary to reach the specifications of in situ soil data to use the code for simulating plate load test.
3. The attained accuracy from the results of cohesive drained soils model is high and close to the actual value. Then it is recommended that the use of the numerical modelling should be unique for cohesive and drained soils.
4. It is recommended that PLT modelling be done by applying 3\( E_0 \) instead of \( E_0 \) to reach rational results.
5. The model has been set in depth of 3.5m and the provided relation is reliable for shallow depth. Then, the validity of the relation should be examined in greater depths.
6. The results of this project confirm the obtained results of Ziaei and Janbaz [10], about efficiency of FE codes in calculation of \( K_s \) in fine soils. In other words, Plaxis and ABAQUS, the FE codes, exhibit better PLT results in fine soils.

Acknowledgment

The authors wish to thank M. Rezaei Nouri because of providing the required data from the sites.

References