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Evaluation of Dynamic Probing Testing Effect in Hand Excavated Pit on Test Results Using Numerical Modeling

S. M. S. Ghorashi*[a], M. Khodaparast[a], A. M. Rajabi[b]

[a] *Faculty of Engineering, University of Qom, Qom, Iran*
[b] *Department of Geology, University of Tehran, Tehran, Iran*

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ABSTRACT

In Iran, using the hand excavated pits (wells) have been more common compared to other countries. As a matter of fact, recent years, utilizing the dynamic probing test (DPT) in these types of pits has been significantly developed in Iran. This is while the standard state of doing this test is from the ground level. In this work, the dynamic probing test is carried out in two similar wells with diameter of 1 m and the depth of 10 m in two areas in city of Qom in Iran; one has silty sand soil and the other is clay. Then, both tests are simulated using numerical modeling in Abaqus software and the results are compared and calibrated with the values obtained at the mentioned sites. The results show a good agreement between the simulation data and tests done in the sites. After calibrating the simulated values with the values obtained from the site, we perform another simulation, this time, for the standard state (It means that the test is done from the ground level or with the assumption without well), as deep as 10 m and for both areas and with the mentioned soils specifications. The results show 35 and 22 percent difference in the dynamic resistance of cone's tip between the testing in standard state and hand excavated pit, for silty sand and clay soils, respectively. Finally, using the simulation, we present the relations between the depth of the test point and dynamic resistance of cone's tip for both states and both types of the soils studied in this paper.

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1. INTRODUCTION

The dynamic probing test (DPT) is one of the in-situ tests that has wide application in identifying soil properties. The primary type of this test was developed by Nicolaus Goldman in 1699, and one of the first standards was facilitated by the Germans in 1977, named DIN [1, 2]. In this test, depending on the type of the dynamic probing, the soil strength is estimated from number of blows needed for specified penetration between 10 to 20 cm. Thereafter, other standards were developed from this test [3–5]. In addition, in 2014 the first national standard was provided for this test in Iran [6]. Figure 1 shows different parts of a dynamic probing.

Boring and drilling are considered as the oldest method for the site investigation. In Iran, due to the existence of domestic expertise, and also considering the long-time experience of the Iranians in hand excavated pits (pits or wells), this method is less costly and hence, more practical. Also, advantages of pits compared to the boreholes such as exact determination of the log, possibility of in-situ density testing, in-situ shear testing and undisturbed sampling of soil, help a lot to identify the foundation of sites, specifically in big and sensitive civil projects. On the other hand, DPT has shown its advantages to use through its enormous applicability in different situations such as pits, which is why are motivated to use it here as well. Since in the standard state, DPT is continuously done from ground level, the direct use of the test result in pit bottom does not give an accurate estimation of soil resistance. Diameter of the pit can have a significant impact on the results of the DPT.

Numerical modeling with a valid computer software which is based on numerical methods is a simple, cheap and accurate way to evaluate the in-situ tests results and other geotechnical phenomena.

Most studies on numerical modeling of penetration tests so far are about the cone penetration tests (CPT) [7–13]. However, so far, only the empirical researches have been presented on DPT [14–18]. In this study, the aim is to evaluate the effect of the pit (well) on DPT (DPL type) results using numerical modeling. With the help of this method, without any cost, the difference of the cone's tip strength values in the standard state and the test in the pit

*Corresponding Author's Email: *mr.sms.ghorashi@gmail.com*
(S. M. S. Ghorashi)

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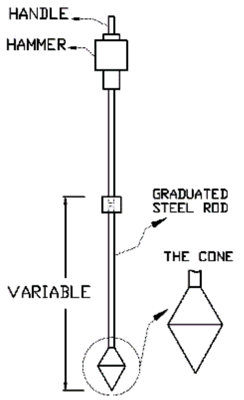

Figure 1. Schematic device structure of a dynamic probing

can be obtained. Therefore, after testing in the pit bottom, by modifying the values, the values of the foundation of the construction site are accurately measured. This study is novel and has been carried out for the first time.

2. TEST SPECIFICATIONS IN THE STUDY AREA

In this study, the DPT has been implemented using light dynamic probing (DPL) in two areas of Qom city in Iran. The site of performing the DPT is shown in Figure 2.

Tables 1 and 2 show the specification of DPL used for the calculation of dynamic resistance of the cone's tip (q_d) and properties of soil types in the study areas, respectively. Soil properties at the sites have been obtained in the laboratory in Qom city.

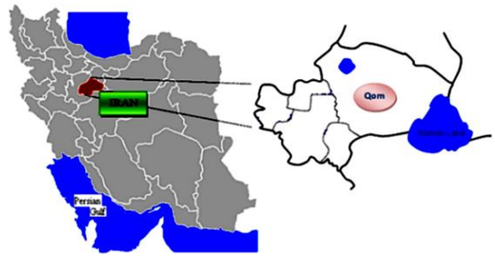

Figure 2. The site of study area

Table 3 shows the test results for the number of hammer blows in both types of soils in the site. Meanwhile, according to the standard of DPL, number of hammer blows is for specified penetration of 10 cm (N_{10}) [5].

The dynamic resistance of cone's tip (q_d) is achieved using the number of hammer blows, according to the following relationship [19]:

$$q_d = (\frac{m}{m+m'})r_d \quad (1)$$

$$r_d = \frac{mgh}{Ae} \quad (2)$$

where, m is hammer mass in kg, m' the total mass of penetrating cone, rod drive, anvil and guide rod (kg), g gravity acceleration in the unit of m/s^2, h the height of hammer fall in meter (m), A nominal base area (m^2) and e the average of penetration value in each blow (m) equal to $0.1/N_{10}$ based on DPL type.

As a result, according to the above relationship, the value of (q_d) in depth of 10 m for both types of clay and silty sand soil, equal to 2.56 and 4.26 MPa, respectively.

TABLE 1. The Specification of DPL used in study area

Soil types	Specific weight, γ (KN/m^3)	Elasticity modulus, E (MPa)	Poisson ratio, ν	Internal friction angle, ϕ' (º)	Cohesion, C' (kPa)
Silty sand	18.5	85	0.3	34	0
Clay	17	20	0.4	23	3.5

TABLE 2. The properties of soil types in the study areas

Specification	Value	Unit
Hammer mass	10	Kg
Height of hammer fall	0.5	m
Anvil, guide rod and penetrating-cone mass	6	Kg
Drive rods mass	3	Kg/m
Cone diameter	36	mm
Nominal base area	10	cm^2
Cone angle	90	(º) degree
Specific work per blow	49	(kJ/m^2)

TABLE 3. The results of DPT test in the study areas

Soil type	Number of hammer blows for penetration of 10 cm	Standard range of hammer blows in DPT [6]
Silty sand	40	3-50
Clay	24	3-50

3. NUMERICAL MODELING

3. 1. Geometry and Meshing of the Model

In this study, the aim is to use Abaqus finite element software for numerical modeling of the DPL penetration in soil. The cone of dynamic probing and soil environment are two main parts of the desired modeling. Thanks to the axial symmetry of the cone and soil environment, the simulation has been performed in two dimensions and for half of the cone and soil environment. The objects or environments that have axial symmetry can be simulated in two dimension and for half of them. Therefore, the elements are used in the model must be axisymmetric elements. With the help of this modeling, we can decrease the time of computing in Abaqus software without the minimum computational error.

Because of the higher stiffness of the cone compared to the soil, the cone and soil have been modeled as the rigid and deformable systems, respectively.

The soil environment used in the modeling has been assumed to have 1.5 *m* width and 3 *m* depth. The soil model has been meshed using 1811 elements, including CAX4R element (4-node, reduce integration, axisymmetric element) and the right and the bottom boundaries have been meshed using CINAX4 element (4-node, axisymmetric, infinite element).

Also, in order to increase the accuracy of simulation, the elements size is reduced, as it gets closer to the cone. The infinite elements have been used in the right and bottom boundaries of soil environment to reduce the effect of the boundary conditions. On the other side, there is an axis of symmetry in the left boundary and an overburden pressure is applied on the top of the soil model which is equivalent to the pressure value in the depth that DPT is performing. Also in the pit model, vertical displacement has been blocked in the area of non-overburden pressure.

Figure 3 depicts the model that has been created by Abaqus software for two testing cases: pit bottom and standard state with depth of 10 *m*. Since all tests have been done in depth 10 *m*, the same overburden pressure has been used for both soils.

To apply the effect of horizontal stresses in model, the coefficient of earth pressure at rest (k_0) has been estimated using Equations (3) and (4) [20]:

$$k_0 = 1 - \sin\phi' \quad (3)$$

$$k_0 = 0.95 - \sin\phi' \quad (4)$$

Equations (3) and (4) are for silty sand and clay soil, respectively.

3. 2. Loading

Based on the standard instruction of DPT, penetrating of the cone in DPT must be continuously conducted into the ground. Also, the penetration rate must be kept between 15 to 30 blows per minute [5]. According to Table 3, the values of N_{10} in the site are 40 and 24 for silty sand and clay soil, respectively. Also, based on the standard considerations of DPT, the penetration rate of 30 and 20 blows per minute has been considered for silty sand and clay soil, respectively.

Therefore, considering the proportional relation between the values recorded in the site (N_{10}) and the value considered of the test standard range for 10 *cm* penetration, the penetration velocity of penetrometer in silty sand and clay soil has been achieved 0.00125 *m/s* and 0.00139 *m/s*, respectively.

3. 3. Interaction of Parts of Model

The surface to surface contact [21] has been used in the definition of interaction between cone and soil. And the cone and soil are chosen to be master and slave surfaces respectively, based on the higher stiffness of the cone. In order to simplify the model, the friction coefficient between boundary surfaces of the cone and soil neglected to prevent interference of sleeve friction resistance in determining of the cone's tip resistance.

3. 4. Soil Behavior

In this study, in order to determine the soil behavior, the Drucker-Prager model has been employed. Figure 4 shows the failure line of Drucker-Prager model in *p*-*q* plane [22].

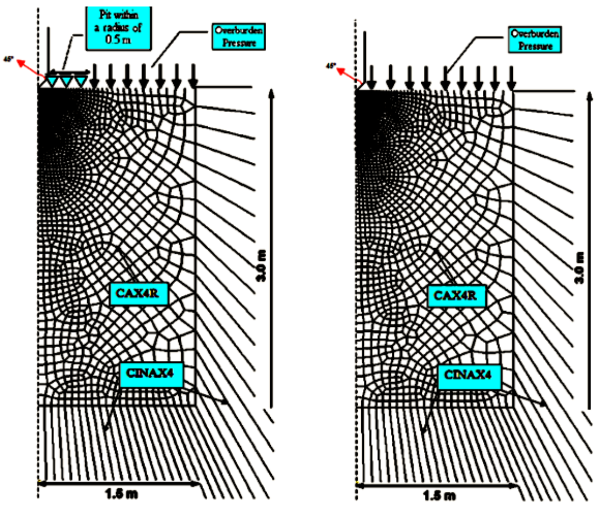

Figure 3. The model in state of testing at pit bottom (Left) and continuously from ground level (Right)

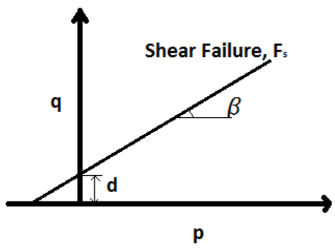

Figure 4. The failure line in yield criterion of Drucker-Prager model

To calculate the values of friction angle (β) and cohesion (d) in Drucker-Prager model using the soil parameters of ϕ' and C' in the site, following relationships can be used [23]:

$$tan\,\beta = \frac{6\,sin\,\phi'}{3-sin\,\phi'} \tag{5}$$

$$d = c'\,\frac{6\,cos\,\phi'}{3-sin\,\phi'} \tag{6}$$

Another parameter that is required to define the plastic area of soil in the Drucker-Prager model is the flow stress ratio, which ranges from 0.788 to 1 [23]. The parameters required in the definition of the Drucker-Prager model have been taken for the simulation in Abaqus from Table 4 for the silty sand and clay soil.

Another parameter needed in this model is the dilation angle. Because the experimental values of dilation angle are not available, it is calculated by using the values given in Table 5 employing following equation [24]:

$$\psi = \phi' - \phi_{cr} \tag{7}$$

where ψ and ϕ_{cr} are dilation and critical friction angles.

3. 5. The Method of Analyzing the Model In order to analyze the model, dynamic explicit method has been used. In addition, due to the creation of large displacement when the cone is penetrating, the Arbitrary Lagrangian Eulerian (ALE) technique has been employed in the vicinity of the cone. ALE technique combines the features of pure Lagrangian and pure Eulerian analysis. In other words, the mesh networking moves independent of material and hence a high quality of the meshing can be possible even in large deformations. Moreover, the Volume Smoothing (VS) method has been used to implement ALE analysis [21]. The VS approach relocates the position of nodes by computing a volume weighted average of the center of elements which surrounding the node. This approach is shown in Figure 5. Based on Figure 5, new position of node M is determined from the position of the element C1 to C4. The node M approaches to C3 from the C1 as result of VS scheme.

TABLE 4. The parameters needed of Drucker-Prager used in modeling

Soil type	$\beta°$	d	k
Silty sand	54	0	0.788
Clay	42	7.409	1

TABLE 5. The range of friction angles for soils [24]

Soil type	ϕ'	ϕ_{cr}
Silty sand	27-35	24-32
Clay	20-30	15-30

4. RESULT AND DISCUSSION

The q_d values are taken from the simulation by dividing the vertical reaction force of the cone tip over the cross section area of the cone's dynamic probing. It is noteworthy that the values obtained from the numerical modeling in both soils have been considered for the penetration of about 20 *cm*. Figure 6 shows the penetration of the cone into the soil environment that is modeled in Abaqus.

Table 6 shows the q_d values obtained from the numerical modeling in pit bottom in depth of 10 *m* for the different values of ψ. As is shown in Table 6, the closest values of modeling to the values obtained from DPT (DPL type) in pit bottom are achieved at angles of 0º and 9º for silty sand and clay soils, respectively.

Table 7 shows a good agreement between the values obtained from the site and modeling for corresponding dilation angles. Therefore, these data have been employed to estimate the effect of pit diameter on DPT (DPL type) results for both soils.

In addition of the values obtained from pit bottom, it the q_d values in the standard state are needed as well (it means that the test is continuously done from ground level); then, we can model DPT for both cases (pit bottom and standard state) in similar conditions.

Table 8 shows q_d values obtained from numerical modeling in both states mentioned above.

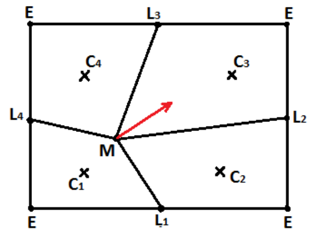

Figure 5. The VS approach used in modeling

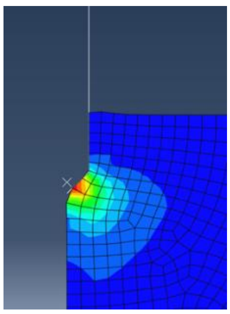
Figure 6. The penetration of cone in soil

TABLE 6. The q_d values versus ψ for soils

ψ(°)	$q_d(MPa)$	
	Silty sand	**Clay**
0	4.2	1.6
1	4.6	-
5	6.1	-
7	-	2.3
9	-	2.5

TABLE 7. The q_d values obtained in the modeling and the site

Soil type	ψ (°)	$q_d(Mpa)$	
		In the modeling	**In the site**
Silty sand	0	4.2	4.26
Clay	9	2.5	2.56

TABLE 8. The q_d values obtained at the case of testing in pit bottom and standard state

Soil type	$q_d(Mpa)$		Difference (%)
	In pit bottom	**In standard state**	
Silty sand	4.2	6.5	35
Clay	2.5	3.2	22

Figures 7 and 8 show the process of achieving q_d at the penetration depth (d_p) of about 20 *cm* in both the pit bottom and standard state for silty sand and clay soil.

To estimate the depth effect of DPL in both soils, in addition to the test modeling in depth of 10 *m* which was mentioned before, the modeling has been performed for depths of 2, 4, 6 and 8 *m* in both cases (pit bottom and standard state) and both soils.

As is shown in Figures 9 and 10, by increasing the depth of the test, which is followed by increasing the overburden pressure, the values of q_d increase. However,

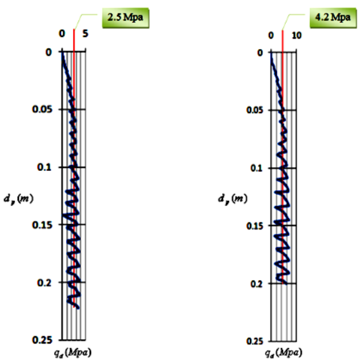

Figure 7. The q_d value for clay (Left) and silty sand (Right), in pit bottom

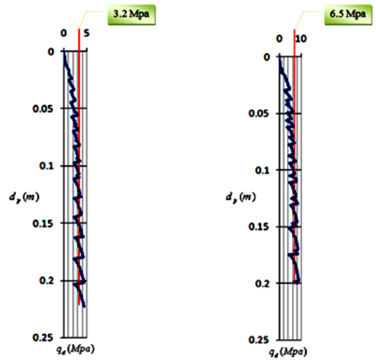

Figure 8. The q_d value for clay (Left) and silty sand (Right), in standard state

it should be noted that this increase is not linear, and in the higher depths, this increment is decreasing. It is also possible to obtain the relationship between the depth of the test (H_{dpl}) and the q_d values by fitting a second-degree curve for both soil types (Equations (7) to (10)).

In the following, Equations (8) and (9) render the values of q_d in terms of the depth of the test, at the pit bottom and the standard state for silty sand soil, respectively:

$$q_d = -0.03H_{DPL}^2 + 0.714H_{DPL}(R^2 = 0.999) \tag{8}$$

$$q_d = -0.062H_{DPL}^2 + 1.260H_{DPL}(R^2 = 0.996) \tag{9}$$

Similarly, the values of q_d are obtained using Equations (10) and (11) in terms of the depth of the test for clay soil:

$$q_d = -0.023H_{DPL}^2 + 0.48H_{DPL}(R^2 = 0.998) \tag{10}$$

$$q_d = -0.03H_{DPL}^2 + 0.621H_{DPL}(R^2 = 0.998) \tag{11}$$

As is clear, the fitted curves have a very good agreement with q_d values obtained from DPL test for both soils types in the pit bottom and standard state. In such way that the regression coefficient (R^2) is close to 1 for all equations. In addition, as shown in Figures 9 and 10, the difference in q_d values are significantly larger for silty sand in compare to clay soil. This is due to the lack of cohesion in silty sand.

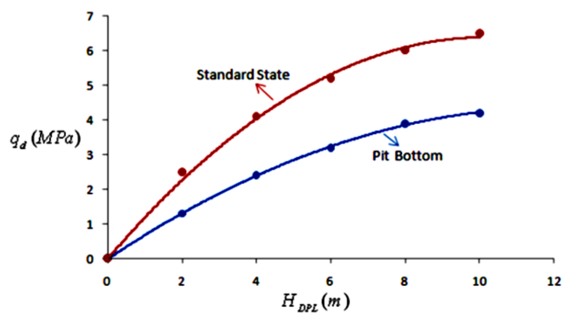

Figure 9. The relation of q_d values and H_{dpl} in pit bottom and standard state, for silty sand

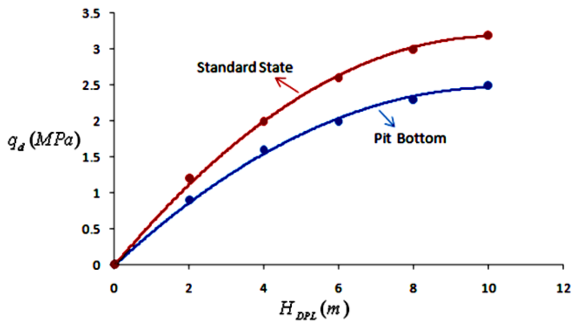

Figure 10. The relation of q_d values and H_{dpl} in pit bottom and standard state, for clay

Another point is that the q_d values on silty sand are higher than the q_d values of the clay soil in both standard state and pit bottom, due to. to the internal friction angle of the soil. In silty sand, although there is no cohesion, its internal friction angle is greater than the clay soil. Therefore, the q_d values of the silty sand are more.

5. CONCLUSION

It was found that by using values obtained from the numerical modeling via Abaqus, it presents an accurate calibration of q_d values for the results performed from DPT (DPL type) in pit bottom (diameter of 1 m and 10 m depth) for silty sand and clay soil in the sites mentioned. The values of q_d in pit bottom are smaller than the standard state case, which means the penetration of the cone is simpler in the former case. Furthermore, the differences of q_d values between pit bottom case (diameter of 1 m and 10 m depth) and the case where test is continuously done from the ground level (standard state) are calculated to be 35 and 22 percent for silty sand and clay soils, respectively. Therefore, cohesion in clay causes the soil particles to be held together. As a result, it causes the difference in q_d values of the pit being smaller than the standard state. However, this difference is higher in sandy soil due to lack of cohesion.

The internal friction angle has a more important role than cohesion to increase the q_d values. Nevertheless, cohesion plays a more important role than internal friction angle to decrease the q_d values in both the standard state and pi bottom.

Finally, a set of second-degree equations for the q_d values have been obtained. These equations had the best fit with the q_d values. It is found that q_d increases by increasing the depth of test, whereas the value of this increment decreases by increasing test depth. These results are true for both cases in both types of soils.

The main innovation of this work is that it is the first time a dynamic probing test is simulated with numerical modeling. In addition, since the DPT has been performed in the pit and the standard state of this test is from the ground level, so with the help of this method, it can be done at any depth of the ground. However, performing the DPT in the pit leads to easier penetration of the cone in the soil, and therefore the values obtained from this test in the pit in the desired depth are not real values. Therefore, using numerical modeling, the values in the site have been calibrated with the simulated values. Then, using the calibrated model, this time the simulation is carried out with the assumption of testing in the standard state and at the same depth. With the help of this method, in any depth, the test can be done and the obtained values can be modified using the numerical modeling and with the lowest possible errors.

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Persian Abstract

چکیده

گمانه‌های دستی (چاه‌ها) در ایران نسبت به سایر کشورها رایج‌ترند. همچنین، اخیراً در ایران، استفاده از آزمون کاوشگردینامیکی در این گمانه ها به طور قابل توجهی توسعه یافته است. این در حالی است حالت استاندارد انجام این آزمایش از سطح زمین می باشد. در این مقاله، آزمون کاوشگر دینامیکی در دو چاه مشابه با قطر۱ متر و عمق ۱۰ متر در دو منطقه از شهر قم در ایران که یکی دارای خاک ماسه‌ای لای‌دار و دیگری رسی است، انجام شده است. سپس هر دو آزمون با استفاده از نرم‌افزار آباکوس شبیه‌سازی و نتایج به‌دست‌آمده از شبیه‌سازی با مقادیر به‌دست آمده از ساختگاه‌های ذکر شده مقایسه و کالیبره شده است. نتایج هم‌خوانی خوبی بین مقادیر به‌دست آمده از شبیه‌سازی و آزمون‌های انجام شده در ساختگاه‌ها را نشان می‌دهد. پس از کالیبره کردن مقادیر شبیه‌سازی شده با مقادیر به‌دست آمده از ساختگاه، این بار شبیه‌سازی را برای حالت استاندارد (به معنای انجام آزمایش به‌طور پیوسته از سطح زمین یا با فرض نبود چاه) در همان عمق ۱۰ متر و برای هر دو ناحیه و با مشخصات خاک‌های مذکور انجام می‌دهیم. نتایج به ترتیب، اختلاف ۳۵ و ۲۲ درصدی مقادیر مقاومت دینامیکی نوک مخروط به دست آمده از انجام آزمون کاوش‌گر دینامیکی در گمانه‌ی دستی با مقادیر به‌دست آمده در حالت استاندارد، برای خاک ماسه‌ای سیلتی و رسی را نشان می‌دهند. در نهایت با استفاده از شبیه‌سازی روابطی بین عمق انجام آزمون و مقاومت دینامیکی نوک مخروط برای هر دو حالت مذکور و هر دو نوع خاک مورد مطالعه، ارائه شده است.