COMPARISON OF STIFFNESS DERIVED FROM PRESSUREMETER AND MONITORING OF A HEAVY BUILDING

COMPARISON DES RIGIDITE′ OBTENUE PRESSIOMETRIQUESE ET DE LA RIGIDITE′ OBTENUE DE LA MESURE DES BATIMENTS LOURDS

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ABSTRACT - The settlement of Milad Tower of Tehran were monitored during the construction. Comparison of calculated and measured settlement of Milad tower proved that module of elasticity derived from the reloading section of pressuremeter tests are accurate to predict the settlement of the gravel of Tehran.

RÉSUMÉ– Le tassement de la tour Milad de Tèhèran qui est en train de construir a mesuré. La comparaison du tassement calcule ´ et le tassement mesure ´ a montre ´ que la module d′ elasticite′ de la test de pressiometrie est pre′cise pour l′ estimation du tassement.

1. Introduction

The stiffness of gravels has not been widely studied (Tatsuoka and Kohata 1995). The geotechnical exploration of coarse gravel is problematic because of difficulties in undisturbed sampling and conventional in-situ testing. The site of Milad tower of Tehran, Figure 1, have been selected for the research on the stiffness of such soils (Pahlavan, 2003). Coarse and angular cemented particles are important features of the ground. Seven boreholes excavated for the research. Table 1 summarizes geotechnical parameters of studied sites. The soil could be commonly classified as gravel (G). The results of SPT are generally above 50 and the average unit weight is 2.2 ton/ m^3 (Pahlavan, 2003).

Table1. Geotechnical parameters of the studied site.

Depth(m)	Classification	Gravel & Pebble (%)	Sand (%)	PI	Clay &Silt (%)
0-2	GP-GM	60	31	24	9
2-3	CL-ML	8	33	14	59
3-7.5	GP-GC	60	28	31	12
7.5-10	GW-GC	60	33	27	7
10-11.5	CL	8	41	19	51
11.5-12.5	SC	30	40	20	30
12.5-26	GW-GM, GC, GP	Variable			

2. Presssuremeter tests

The self–boring pressuremeter (SBP) is an ideal tool for determining the stiffness of many types of soils but it is difficult to use SBP in coarse alluvium deposits. This is attributed to difficulties in drilling, pushing and driving of test probes. Therefore, the pre–bored (Menard) pressuremeter (PMT) type GC (Baguelin et al., 1978) with slotted tube was used in the study. The tri-cell probe was 44 mm in diameter with a maximum pressure capacity of 6 MPa. The central measuring cell with length of 210 mm was inflated with water, and the two guard cells with nitrogen gas. Water was used for volume change measurement. An electronic device with resolution of 10kPa for pressure and 1cc for volume change, along with conventional measuring system was used. To prevent the bursting of rubber membranes, a slotted tube of 63mm diameter and 1.5m lengths with 6 longitudinal slots of 1m lengths was used.

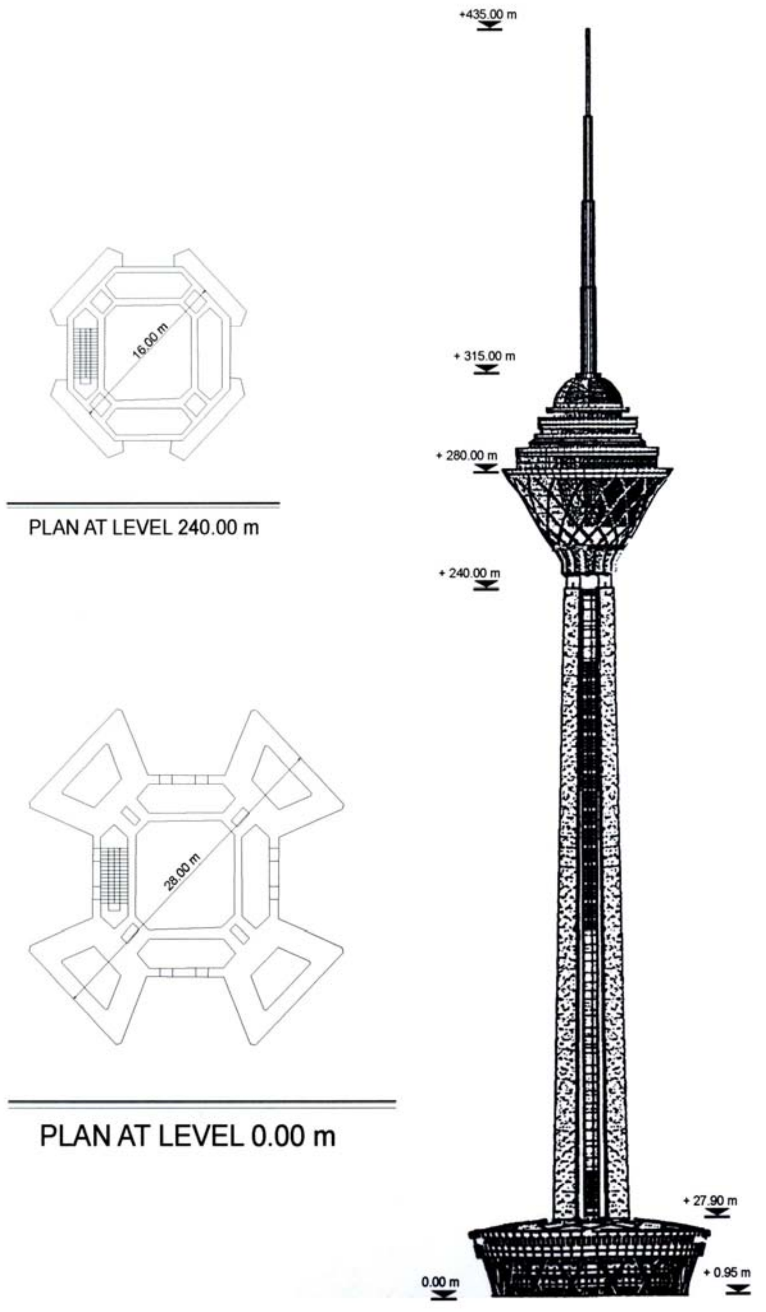

Figure1. Milad Tower of Tehran

The past attempts for performing pressuremeter test in coarse gravel of Tehran were unsuccessful, mainly due to drilling problems. The test pockets were usually too tight or too wide because of large size particles. The use of various trial methods for the creation of test pockets was discussed by Pahlavan et al (2004). In presented study, a modified single core barrel of 59mm external diameter was used. Fixing a reamer of 64mm external diameter on the top of core barrel just below the head and another one behind the core bit were proved to be successful. Another problem at the early stages of the research was the frequent bursting of membranes even using the slotted tube. After visual inspection of busted membranes, Pahlavan et al. (2004) judged that the failure is due to laying sharp coarse cuttings of soils over a fulcrum created during the expansion of membrane between the probe and slotted tube. To overcome the problem, various methods are discussed Pahlavan et al. (2004). The pressuremeter tests were carried out according to the stress control method, described as "Method A" of ASTM D4719-1994. To study the unloading-reloading stiffness, unloading-reloading loops were performed according to Baguelin et al. (1978). Considering the creep of unloading point, the loops were formed according to Fahey (1991) so the pressure was fixed until deformation stops. A total of 37 pressuremeter tests, in 7 boreholes, have been performed at depths of 1 to 25m. The results have been corrected for the pressure and volume losses, according to Baguelin et al. (1978).

3. Shear modulus

Menard shear moduli (G_M) from pseudo-elastic part of pressuremeter curves and also reloading moduli (Gr) from reloading part of unloading–reloading loops were determined. The volumetric strains were converted into circumferential strains at the cavity wall according to Briaud et al. (1983). Then pressuremeter moduli were calculated using Equation (1):

$$G = \frac{1}{2}\frac{\Delta P}{\Delta \varepsilon_\theta} \tag{1}$$

Where ΔP and $\Delta\varepsilon_\theta$ represent the change of pressure and cavity strain respectively. In order to calculate stiffness for large and small strains, secant shear moduli were determined. This was carried out for the reloading section of unloading-reloading loops (such as points A, B & C in Figure 2). Hereby, secant shear modulus (Gs) is the stiffness, measured between the start point of reloading section and any other point on the curve of reloading.

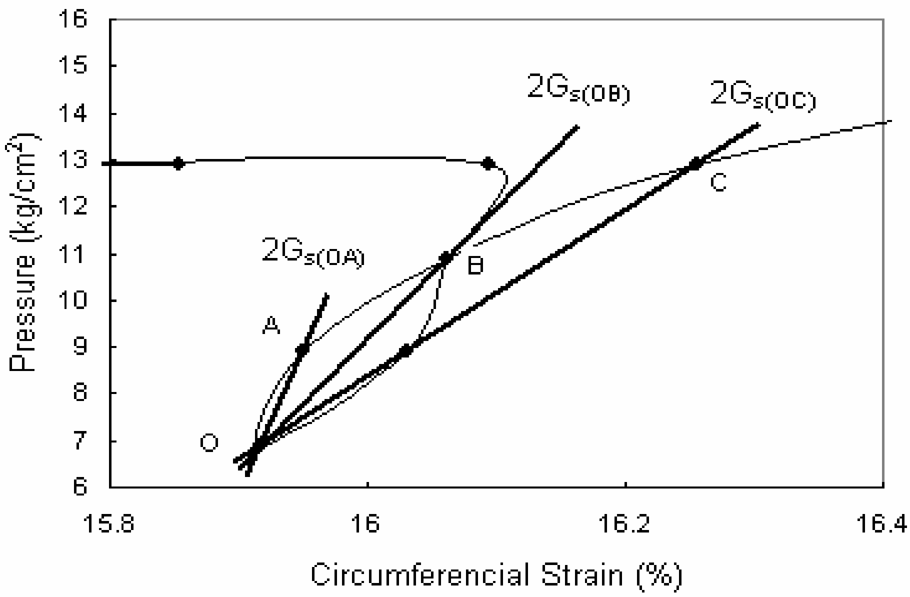

Figure 2. Determination of shear stiffness from unloading-reloading loops

Shear wave tests were also carried out to obtain the shear modulus of soil for small strain (G_o). The comparison of results is discussed by Pahlavan et al, (2004) and the relationship of $G_o=60G_M$ is proposed between pressuremeter moduli (G_M) and (G_o) for the studied sites.

4- The use of different stiffness values in settlement predictions

The Milad Tower of Tehran, with the final height of 435m, has a circular footing with a diameter of 66m and embedment depth of 14m. The tower was monitored at the time of construction. Calculations of tower settlement have been carried out a comparison between calculated and measured settlements is presented. Elasticity modulus (E) has been determined from shear modulus of pressuremeter tests and shear waves. Poisson's ratio of ν=0.28 was used and Equation 2 applied.

$$E = 2\,G\,(1+\nu) \tag{2}$$

Table 2 indicates shear moduli and elasticity moduli.

Table 2. Modules used to calculate the settlement of Milad Tower

Depth (m)	Shear Modulus (MPa)	Elasticity Modulus (MPa)
Pressuremeter Tests		
	G_r	E_M
4-0	102.9	55.6
6-4	233	82.2
8-6	101	66.6
12-8	137	96.5
14-12	137	64.8
19-14	172	96.8
20-19	339.4	119.6
25-20	172.8	186.6
>25	603.8	267.5
Shear Wave Tests		
2-0	445.5	1140.5
4-2	685	1753.6
6-4	1318	3374
9-6	1497.6	3834
13-9	2010.7	5147.3
16-13	3522.8	9018.3
19-16	4600	11776
22-19	5096	13045.8
26-22	5217	13356.8
>26	5354	13706.2

4.1- Calculation of settlement by closed-form solution for elastic half-space

The equation, proposed by (Timoshenkov & Goodier, 1951), and routine calculation procedure indicated in textbooks was followed. The coefficient of depth effect is used and influence factors was reduced 7% for rigid foundations (Bowles, 1996). The stratum depth, causing settlement, was assumed to be equal to 5B (Bowles, 1996), B is foundation diameter. Also, the weighted average values of (Es) were used. Hereby, applying the elasticity moduli of pressuremeter test (E_r, E_M) and shear wave (Eo), the settlement of Milad Tower Foundation was calculated.

4.2- Calculation of settlement by method of Menard & Rousseau

Semi-empirical equation by Menard and Rousseau, was used to calculate the settlement of the footing Baguelin et al. (1978), assuming a homogenous ground:

$$S = \frac{2}{9E_d} q^* .B_0 (\lambda_d \frac{B}{B_0})^\alpha + \frac{\alpha}{9Ec} .q^* .\lambda_c .B \tag{3}$$

Where,
q*= The net average bearing stress q-qo
q = The average bearing stress
qo = The vertical stress of adjacent overburden to the foundation, in terms of the total stress, at the foundation level.
Bo = Reference width (Normally equals 60 cm)
D = Depth of foundation, which is equal to 14m for the studies tower.
B = Diameter or width of foundation, assumed to be more tan Bo, B≥Bo
α = Rheological coefficient, suggested by Baguelin et al. (1978). It is assumed equal to 0.33 for the studied case.
λ_c, λ_d = Shape factors, which are function of ratio of length to width of footing. (Baguelin et al., 1978). These factors are equal to 1 for the studied case.
Foundation Depth Influence Coefficient is assumed to be equal 1.16.

Table 4 indicates the input parameters and the settlements of layered ground in the site of Milad Tower.

Table 3. The used parameters and calculated settlements by semi-empirical method

Ec * (Mpa)	Ed ** (Mpa)	surcharge due to lateral earth pressure (kPa)	net pressure (kPa)on foundation q-q$_o$=q*	Settlement (mm)
221.3	308	143.9	112	2.6
* Ec=E1*** ** 1/Ed = ¼ [1/E1 + 1/0.85 E2 + 1/E3, 4,5 + 1/ 2.5E6,7,8 + 1/ 2.5E9,10] *** Subscribe 1 denotes Layer 1				

4.3 - Finite element analysis of settlement

AFENA (Carter & Balaam, 2000) finite element computer program was applied for settlement analysis. AFENA could consider the non-linear behavior (e.g. - Fahey and Carter's model, 1993). It also, incorporates Junbu (1963) Model to consider the effects of confining pressure. The details of the analysis are described by Pahlavan (2003). Both linear elastic and Moher-Colomb elastoplastic models was used but no difference between the results was observed in the range of applied loads of Milad Tower. This proves that the cemented coarse gravel of the site is so hard that, even the load of Milad tower can not lead to the increase of the strains behind linear range.

In linear, elastic model, the calculation of settlement was carried out for two cases of uniform and layered ground, using elasticity moduli of pressuremeter tests (Er & Em), and shear wave (Eo). Table 4 indicates the calculated settlements.

Fahey and Carter's non-linear model (1993) was also used for foundation settlement estimation. Through this method, the following equation, which is similar to Junbu (1963) formula, is applied for the changes of soil stiffness by the confining stress level:

$$\frac{G_o}{P_a} = C\left(\frac{P'}{P_a}\right)^n \tag{4}$$

In this formula; P' is the average effective confining pressure; Pa denotes atmosphere pressure to make the parameter of C dimensionless. Go presents shear modulus resulted from shear wave test or maximum shear modulus; and n is modulus power. The non-linear stiffness of soils could be assumed as a modified hyperbolic (Fahey & Carter, 1993):

$$\frac{G}{G_o} = \left[1 - f\left(\frac{\tau}{\tau_{\max}}\right)^g\right] \tag{8}$$

Where; G is the shear stiffness resulted from unloading-reloading loops or secant shear modulus. Go is maximum shear stiffness or the shear stiffness resulted from shear wave. τ represents shear stress for the measured G and $\tau_{\max}$ is the maximum shear stress (shear strength) of soil. *f & g* are two empirical parameters. The modified hyperbolic model of Fahey & Carter (1993) is applied for modeling the stress-strain equation. The calculations of settlement were carried out for the values of (*g*=3.3 and *f*=1), (*g*=2 & *f*=1), (*g*=3.3 & *f*=0.5), and (*g*=10 & *f*=1) derived from curve fitting of actual stiffness of soil for various strain by Pahlavan (2003). The other input parameters of AFENA software are as follows:

C=16500, Pa=100 kPa, and n=0.6 based upon fitting Junbu Model for shear moduli Go

ϕ' =37° and c =293 kPa (Soil Engineering Services Co., 1996)

ψ =20° and ko =1 are assumed. Regarding the fact that the studied soil is in the range of linear elastic deformations -under the pressure of tower foundation. Since no plastic deformation occurs, the assumed value of ψ has not influenced the result.

ν =0.1 is taken. The value of ν, for Fahey & Carter's Model has been discussed comprehensively (Fahey & Carter, 1993). If ν is considered as a constant, the volumetric modulus of soil (K), directly related to shear modulus (G), will reduce by increase of shear stresses. Therefore, Fahey & Carter (1993) proposed that the value of ν must increase with the increase of shear stress, to keep K constant. The value of 0.1 is suggested for the minimum ν, which will reach 0.5 by reduction of G. Table 4 indicates the calculated values of settlement of Milad Tower foundation, applying Fahey & Carter's Model, for different values of *f & g*.

5- Comparison of measured and calculated settlement

Any likely motions of Milad tower were measured by means of micro-geodesy system through the monitoring of any movement of 28 points on the body of the tower. No reading was undertaken for points located on the footing of the tower at the early stages of constructions. The first reading of points, located on the body of the tower, was taken after the completion of the tower up to height of 49.2m. The second reading was taken after the tower reached to the height of 308m. An extra weight of 144040 Tons, equal to an extra stress of 420kPa, was exerted to the ground between two readings. The difference of two readings shows the settlement of the soil and also deformation of concrete under the extra weight. Based upon simple calculation, the deformation of concrete proved to be neglectable. In accordance with the measurements, the soil settlement has been determined as 24.3 mm, resulted by the pressure of 420 kPa on ground (Mahab-Ghods Consultant Engineers, 2002).

Table 4 indicates the measured and calculated settlements for Milad Tower by different methods. The largest values of calculated settlements are resulted from applying elasticity moduli of pressuremeter tests (Er & Em). Through the performed tests of this investigation, the circumferential strain level of the resulted moduli from pseudo-elastic section is between 0.5% & 7% for (Em), and between 0.1% & 0.9% for (Er). The strain levels in hard soils are normally

0.1% or less, and 0.5% in most states (Tatsuoka & Kohata, 1995). The least calculated settlement is related to shear wave moduli (Eo), considered in Junbu Model. The effect of confining stress level has been considered, so soil stiffness has increased according to depth. The calculated settlements, by shear wave moduli (Eo), are related to shear wave moduli of shallow depths (0.16 B to 5B) and the calculated settlements are much less than the values from pressuremeter test, but more than the results of Junbu Model. In Fahey & Carter's model, the role of strain level is also considered. However, it should be noted that, in the studied cases, *f & g* do not influence the results. It seems that, in stiff soils with high values of Go (like the gravel of Tehran) and strain levels of about 0.1%, the non-linear behavior does not influence the results so linear analysis of the immediate settlement of buildings in such an alluviums is reasonable for buildings.

Table 4. Comparison of settlement (mm) from different methods

<table>
<tr><th>Method</th><th>Model</th><th>Input variables</th><th colspan="2">Center Settlement</th></tr>
<tr><td rowspan="3">Closed-Form Solution</td><td rowspan="3">Linear elastic</td><td>E_M (Menard Module)</td><td colspan="2">139.8</td></tr>
<tr><td>E_r (Module of loading in unload-reload loops)</td><td colspan="2">24.4</td></tr>
<tr><td>E_o (Shear Wave Tests)</td><td colspan="2">2.7</td></tr>
<tr><td>Menard & Rousseau (1962)</td><td>——</td><td>E_M (Menard Modules)</td><td colspan="2">2.6</td></tr>
<tr><td rowspan="11">Finite Element Analysis using AFENA</td><td rowspan="6">Linear Elastic & Elastoplastic (the same)</td><td rowspan="2">E_m (Menard Modules of Pressuremeter Tests)</td><td>Layered Ground</td><td>95.9</td></tr>
<tr><td>Uniform Ground</td><td>135</td></tr>
<tr><td rowspan="2">E_r (Modules of loading in unload-reload loops)</td><td>Layered Ground</td><td>19.2</td></tr>
<tr><td>Uniform Ground</td><td>27.5</td></tr>
<tr><td rowspan="2">E_o (Shear Wave Tests)</td><td>Layered Ground</td><td>1.7</td></tr>
<tr><td>Uniform Ground</td><td>2.1</td></tr>
<tr><td>Junbu (1963)</td><td>fitting E_o (of Shear Wave Tests) to Junbu Model</td><td colspan="2">1.33</td></tr>
<tr><td rowspan="4">Fahey & Carter (1993)</td><td rowspan="4">- fitting Go (of Shear Wave Test) to Junbu model
- G (Modules of loading in unload-reload loops)</td><td>f=0.5 g=3.3</td><td>1.715</td></tr>
<tr><td>f=1, g=3.3</td><td>1.720</td></tr>
<tr><td>f=1, g=2</td><td>1.760</td></tr>
<tr><td>f=1, g=10</td><td>1.710</td></tr>
<tr><td>Micro-Geodesy</td><td colspan="2">according to the measurements</td><td colspan="2">24.3</td></tr>
</table>

Assessing the calculated settlements from different methods, it is proved that only the results of elasticity moduli of reloading section (Er) of pressuremeter tests (among all the methods) present more conformity with the results of measurements.

6. Conclusion

For coarse cemented gravel of Tehran, the shear modulus –resulted by shear wave- is approximately 60 times the same modulus, out of pressuremeter test. In comparison with the measured and calculated settlements of Milad Tower in Tehran, it seems that taking the reloading section of pressuremeter curves –to determine elasticity modulus (Er)- for estimating the settlement of coarse-cemented gravel of Tehran will be more accurate than other methods. Numerical elastoplastic analysis proves that the settlement of high-rise structures foundation can be assumed to be linear in Tehran. Therefore, a linear analysis is of a proper accuracy.

7. References

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