Finding Storage Tank Settlement on Granular Soil with Small-Strain Stiffness

Majid Taherzadeh Dorobati[1]; Alborz Hajiannia[2]; Shahrzad Kasaeian[3]; and Ramin Rasekh[4]

[1]Graduate Student, Dept. of Civil Engineering, Islamic Azad Univ., Najafabad Branch, Esfahan, Iran. E-mail: majid_taherzadeh@yahoo.com
[2]Assistance Professor, Dept. of Civil Engineering, Islamic Azad Univ., Najafabad Branch, Esfahan, Iran. E-mail: Alborz@aut.ac.ir
[3]Ph.D. Student, School of Engineering, Univ. of Queensland (UQ), Australia. E-mail: s.kasaeian@uq.edu.au
[4]Graduate Student, Dept. of Civil Engineering, Islamic Azad Univ., Najafabad Branch, Esfahan, Iran. E-mail: rarasekh@yahoo.com

Abstract: Based on the API650 Standard, there are four types of raft foundations for liquefied hydrocarbons storage tanks. Since soil is the only material used in constructing foundation types B.4.1 and B.4.3, we can advance the process through further research to identify their behavior and technical/engineering characteristics. Because settlement is a fundamental problem in foundation design, studying its affecting parameters and estimation approaches are quite important. A study was conducted in South Pars region (Iran) focusing on type B.4.3 foundation, and its settlements were predicted after performing hydro-tests. The general soil profile of the foundation bed was also recorded through some boreholes and Crosshole test and some laboratory tests. For settlement estimations on granular soils, we used Schmertmann equation which is a function of the strain influence factor; the values proposed by some researchers for this factor yield conservative values for the settlement of the above storage tanks. In this research, Schmertmann equation was rewritten with the proposed strain influence factor (a function of shear wave velocity) and the amount of settlements of the storage tanks were estimated. Results indicate that, compared with relations presented by other researchers, the proposed relation estimates settlements with highly acceptable precision.

1- INTRODUCTION

The load bearing capacity and settlement are two main parameters studied in the design of shallow foundations. Usually, when foundations lie on a bed of granular soil, settlement is the more effective and determining factor, the amount of which is limited to an acceptable value of 25 mm. Janbu et al. (1956); Schmertmann et al.

(1978); Taylor and Matyas (1983); Bowles (1987) and Haddad and Amini Ahidashti (2013) are among the researchers who have presented methods for the prediction of settlement based on the elastic solution of the problem. The most widely used among the above mentioned methods is that of Schmertmann et al.'s which is based on the results of the cone penetration test and the definition of the strain influence factor. Since Schmertmann relation is very widely used to predict shallow foundations settlements, and the strain influence factor is an effective parameter in it, the authors' objective in this paper has been to propose a relation for the determination of this factor, based on the shear wave velocity, for the prediction of settlements of storage tanks lying on Type B.4.3 soil foundations according to API650 (2000) Standard. To validate, the Crosshole and Hydro Tests were performed at the site (based on the API653 (2000) Standard, and 1/4, 2/4, 3/4, and 4/4 capacity filled storage tanks), settlements were registered, and then found by the proposed relation. Results have revealed that the latter has predicted the settlements of the aforementioned storage tanks with a highly acceptable precision.

2- THEORETICAL CONCEPTS

Field tests for the determination of the shear wave velocity measure the soil shear modulus under small strains; the stiffness found so, shows that of the soil or rock (known mostly as the dynamic stiffness), and its value varies nonlinearly depending on the resulted level of stress or strain. The maximum stiffness (G_{max}), known also as "small strain", involves all such soil types as clay, silt, sand, gravel, and rock (Tatsouka et al. 2001). For practical purposes and for small and very small strains, the soil stiffness parameters can be considered as constant; when the strain increases, they decrease. Since the strain level is usually low around the geotechnical structures (retaining walls, foundations, tunnels, etc.), two sets of parameters are needed to measure the displacements (Clayton 2011):

a) Parameters related to very small strains (G, E_0, ν_0)

b) Stiffness parameters (with the stress/strain level increase under loading/load removal)

2-1- STIFFNESS MODIFICATION OF SMALL STRAINS BASED ON SHEAR STRAIN

With an increase in the shear strain, the value of the shear modulus is decreased. To reduce the stiffness, many models have been proposed by different researchers (Seed et al. 1986; Fahey and Carter 1993; Rollins et al. 1998). In addition, an exponential relation has been proposed based on the field and laboratory data to reduce the maximum shear modulus.

$$\frac{G}{G_{MAX}} = \frac{0.06}{\sqrt{\gamma\%}} \tag{1}$$

where G is the shear modulus, and γ is the shear strain; G is found from $G_{MAX} = \rho.V_S^2$, where ρ is the mass density and V_s is the shear wave velocity. In strains above 0.01%, relation (1) yields the maximum shear modulus reduction with acceptable correlation.

Seed et al. (1986) and Rollins et al. (1998) have proposed a range for the maximum shear modulus in gravel in terms of the % shear strain (FIG. 1). In this figure, the graph related to relation 1 (used in this research) can be observed too.

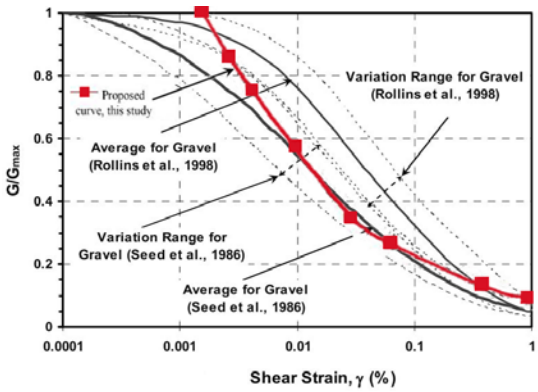
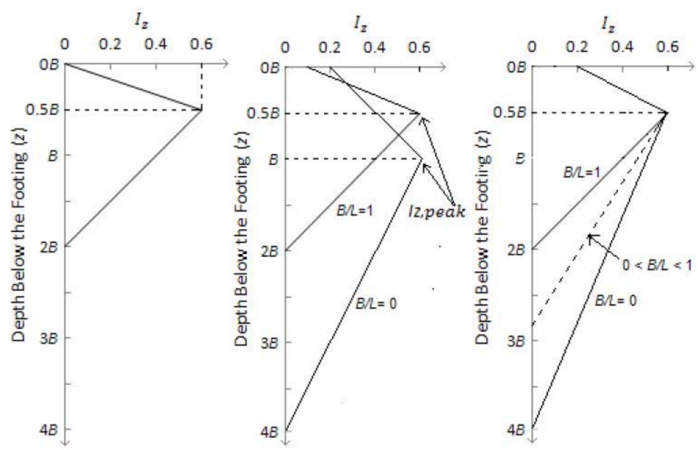


FIG. 1- Range of the stiffness variations with the shear strain level (Rollins et al. 1998; Seed et al. 1986)

2-2- STRAIN INFLUENCE FACTOR AND SCHMERTMANN RELATION

Based on the theory of elasticity, the axial strain ε_Z below the center of a flexible circular loading is as follows:

$$\varepsilon_Z = \frac{q(1+v)}{E_S}\left[(1-2v)\,A' + B'\right] \rightarrow I_Z = \frac{\varepsilon_Z E_S}{q} = (1+v)\left[(1-2v)\,A' + B'\right] \tag{2}$$



(a) (b) (c)

FIG. 2- Diagrams for strain influence factor proposed by different researchers: a) Schmertmann (1970), b) Schmertmann et al. (1978), c) Terzaghi et al. (1996) (adapted after Sivakugan and Das 2010)

where $A', B' = f(Z/B)$, E_S is the Young modulus of elasticity, I_z is the strain influence factor, q is load/unit area, and ψ is the Poisson's ratio. The most important point

about the value of this parameter is to check its variations with an increase in depth. The values suggested for I_Z by other researchers (Terzaghi et al. 1996; Sivakugan and Das 2010) and found by Schmertmann relation are shown in FIG. 2.
Thus, having I_Z, we can predict the settlement as follows:

$$S_e = C_1 C_2 q \sum_{0}^{2B} \frac{I_z}{E_s} \Delta z \tag{3}$$

3- METHOD PROPOSED FOR THE PREDICTION OF THE SETTLEMENT OF RAFT FOUNDATIONS (API650 STANDARD, TYPE B.4.3)

Based on the shear stiffness found for small strains, relations found from the theory of elasticity, and relations presented by Schmertmann, a new method (with the following proposed steps) has been presented to predict the settlement of the raft foundations implemented according to API650 Standard.

Step 1- The shear wave velocity is registered through Crosshole tests in the boreholes under the desired foundations and G_{max} is found from $G_{MAX} = \rho . V_S^2$ based on the shear velocity in every layer.

Step 2- The maximum Young modulus is found from $E_{MAX} = 2(1+\nu)G_{MAX}$.

Step 3- The vertical strain in the middle of every soil layer is found using the theory of elasticity and the following relation:

$$\varepsilon_z = \frac{\sigma_z}{E} - \nu\left(\frac{\sigma_X}{E} + \frac{\sigma_Y}{E}\right) \tag{4}$$

where ν is the Poisson's ratio (equal to 0.3), σ_X, σ_Y, and σ_Z are the vertical and horizontal stresses, and E is the Young modulus. When loading is axisymmetric, $\sigma_X = \sigma_Y = k_0 . \sigma_Z$, where k_0 is the coefficient of the soil lateral pressure at rest; when ν = 0.3 and k_0 = 0.4, we will have:

$$\varepsilon_z = 0.76 \frac{\sigma_z}{E} \tag{5}$$

Step 4- The relation between the shear and axial strains is as follows:

$$\gamma\% = (1+\nu)\varepsilon_z\% \tag{6}$$

where ε_Z is the axial strain (%). Substituting this in relation (1), and taking ν = 0.3, we can have:

$$\frac{G}{G_{MAX}} = \frac{E_z}{E_{MAX}} = \frac{0.06}{\sqrt{\varepsilon_z\%(1+\nu)}} = \frac{0.0526}{\sqrt{\varepsilon_z\%}} \tag{7}$$

Step 5- The axial strain is found from relation (5) and substituted in relation (7); we will have:

$$\varepsilon_z\% = \left(\frac{1444.22\sigma_z}{E_{MAX}}\right)^2 \tag{8}$$

Step 6- Using relation (2) and the axial strain found from relation (8), we can find the strain influence factor as follows:

$$I_z = \frac{\varepsilon_z . E_z}{q} = 3085.46\left(\frac{\sigma_z}{\rho . V_s^2}\right)^2 . \frac{E_z}{q} \tag{9}$$

where E_Z is the elasticity modulus found from the triaxial test in the middle of each layer.To find the excess vertical stress (σ_Z) created at depth Z, use can be made of Boussinesq relation as follows:

$$\sigma_z = q\left[1 - \frac{1}{\left(1 + \left(\frac{R}{Z}\right)^2\right)^{\frac{3}{2}}}\right] \tag{10}$$

where R is the radius of the circular loading area and Z is the desired depth under the center of the loading area.

As I_z highly affects the settlement predictions of Schmertmann relation, we studied, like other researchers, the variations of the proposed I_z with depth (Z/B) and drew the related graph (FIG. 3) based on relation (9) and the shear wave velocity found from the Cross hole Test.

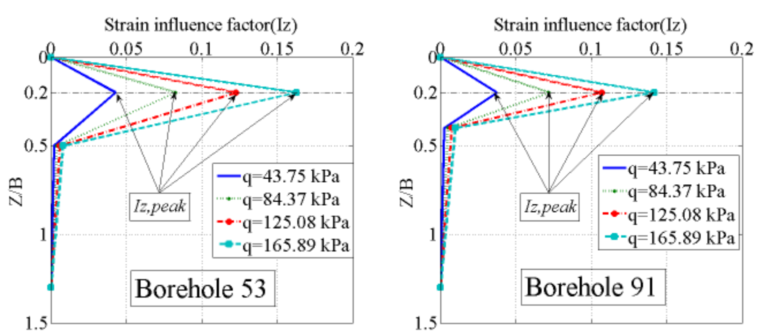


FIG. 3- Proposed strain influence factor diagram under different loadings

As shown in FIG. 3, the proposed I_z variations with depth has a trend similar to the results found by other researchers with the only difference that I_{zpeak} occurs at a depth equal to 0.2B, and reaches its minimum (almost zero) at the depth 0.5B. The authors believe the reason is the presence of a compressible layer up to the depth 0.5B in the site being studied. Therefore, it can be stated that the depth of the compressible layer has a controlling effect on the value of I_z.

Since I_z is very small after depth 0.5B, it can be taken equal to zero and present a graph for full storage tanks (like those presented by Schmertmann 1970; Terzaghi et al. 1996; Sivakugan and Das 2010) as in FIG. 4.

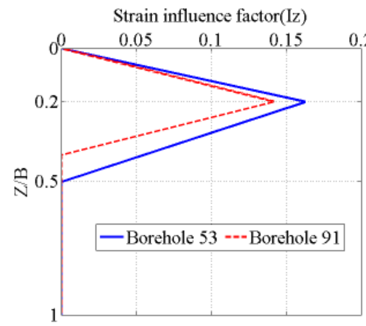


FIG. 4- Proposed strain influence factor diagram

Therefore, having V_s, ρ, E_s for every layer (up to depth 0.5B), I_z can be found from relation 9 for each layer. in the site under investigation use has been made of the Crosshole Test to find V_s. Substituting it in Schmertmann relation and considering C_1=C_2=1, for the storage tanks being studied the settlement can be predicted as follows:

$$S_e = q \sum \frac{I_z}{E}.\Delta z \tag{11}$$

4- CASE HISTORIES

To validate the proposed method, the settlements of 4 oil condensate storage tanks set up on the raft foundations (constructed according to API650/Appendix B, 2000 Standard) in South Pars region, Iran (FIGs. 6 show the details of the raft foundation).were estimated with the proposed relations and compared with those found from the hydrotests (according to API653/Section 10, 2000). FIGs. 5 shows the plan of the position of the boreholes and the storage tanks, and the soil profile of the the site of the raft foundations being studied.

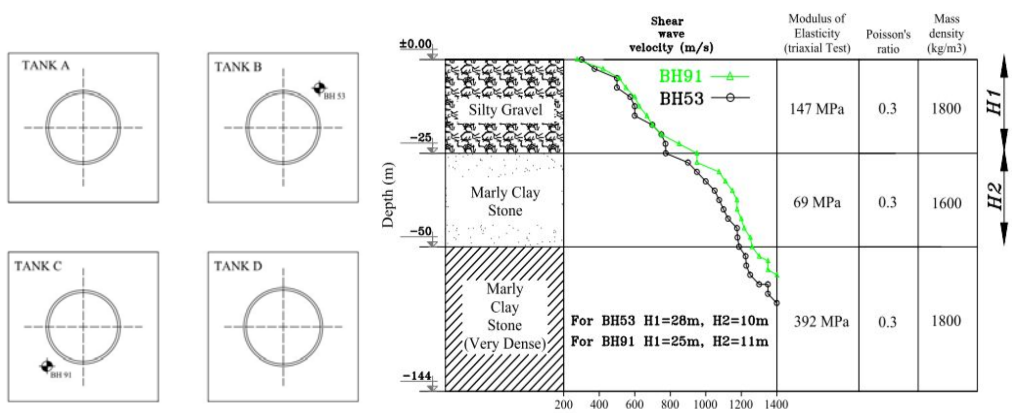
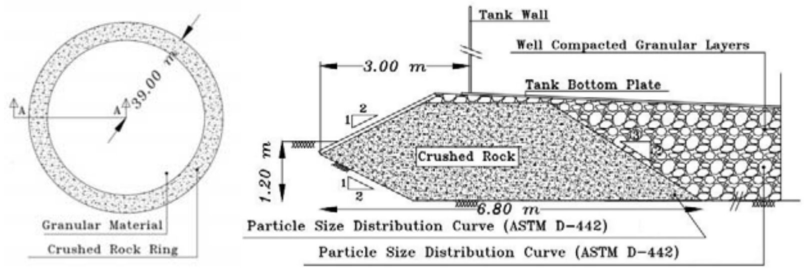


FIG. 5- Position plan and the soil profile of the site of the storasge tanks



(a) Raft foundation plan (b) Cross section A-A

FIG. 6- Plan and transverse section of the raft foundation under investigation

5- RESULTS AND DISCUSSION

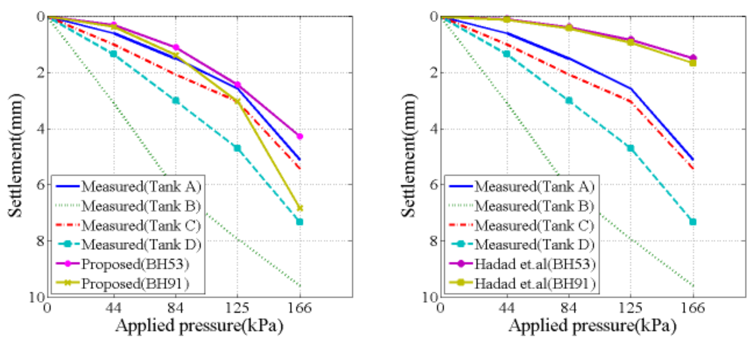
To study the precision of the proposed method, the settlements of the storage tanks under investigation were also estimated by the relations presented by Haddad and Amini Ahidashti (2013); Bowles (1987); Taylor and Matyas (1983) and Janbu et al. (1956) and found also based on the I_z values presented by Schmertmann (1970); Schmertmann et al. (1978) and Terzaghi et al. (1996) and compared with those found from this research and those from monitoring. Relations proposed by Janbu et al. and Taylor et al. and Bowles are to predict the same settlements found from the relations derived from the theory of elasticity with the only difference that the influence factor (I) in them has different values shown in Table 1.

Table 1- Relations used to predict settlements

Method	Expression for settlement	Explanations
Janbu et. al (1956)	$S = \frac{qB}{E_s} I$ q:net applied pressure on the foundation B:diameter of foundation E_s:modulus of elasticity I:influence factor	$I = \mu_0 \mu_1$ $\mu_0, \mu_1 \rightarrow$ in graphs
Taylor et. al (1983)		$I = (1+\nu)\left[(1-\nu)\alpha_0 - \alpha_1\right]$ $\alpha_0, \alpha_1 \rightarrow$ in graphs ν :poisso's ratio
Bowles (1987)		$I = I_s I_f$ I_s = shape factor I_f = depth factor Used: $\frac{H}{B} = 0.5$
Haddad and Amini Ahidashti (2013)	$S = \left(\frac{313.75q}{E_{max}}\right)\left(\frac{B}{50}\right)$ q:net applied pressure on the foundation B:diameter of foundation E_{max}:maximum stiffness	$E_{max} = 2(1+\nu)G_{max}$ ν :poisson's ratio $G_{max} = \rho v_s^2$

5-1- RESULTS OF PREDICTIONS AND COMPARISON

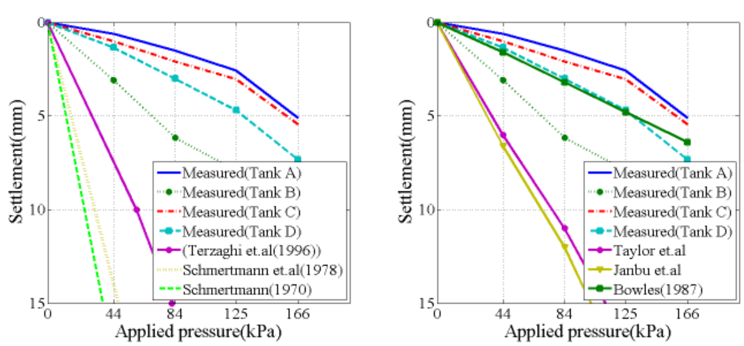
FIG. 7 shows the registered settlements under the 4 storage tanks, and also the results found by the proposed relation based on the data gathered from boreholes 53 and 91. Since the proposed relation and that presented by Haddad and Amini Ahidashti (2013) are both based on Geophysical and seismic tests results, a comparison has been made between them based on the above mentioned gathered data.



(a) (b)

FIG. 7- Settlement comparison monitoring and a) proposed b)Haddad's results

FIG. 8a compares the settlements found from the relations presented by Schmertmann (1970); Schmertmann et al. (1978) and Terzaghi et al. (1996) with those of monitoring, and FIG. 8b compares those found from the theory of elasticity-based relations proposed by Janbu et al. (1956); Taylor and Matyas (1983) and Bowles (1987) with those of monitoring.



(a) (b)

FIG. 8- Settlement comparison: a) Schmertmann/Terzaghi and monitoring results b) Bowles's/Janbu et al.'s/Taylor et al.'s and monitoring results

As shown in FIG. 8b, for H/B = 0.5, Bowles (1987) relation yields quite acceptable results which conform well to those found in the present study $I_{Z_{z=0.5B}} = 0$ In general, it can be stated that the results found from Bowles (1987) and the proposed relations present quite acceptable (compared to other methods studied); this is clearly shown in FIG. 9.

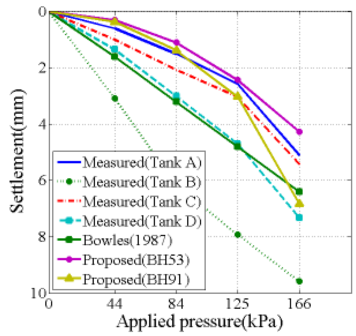


FIG. 9- Settlement comparison (results of the proposed/Bowles relations and monitoring)

6- CONCLUSIONS

In this paper, effort has been made to study the settlement of raft foundations of storage tanks (of liquefied hydrocarbons) constructed over granular soils based on the shear wave velocity and the soil maximum stiffness (stiffness of small strains); results are as follows:

1- The use of real soil characteristics (E_{max}, G_{max}) in the prediction of settlement yields more precise and realistic results in undisturbed versus disturbed soils (digging boreholes, etc.).

2- Geophysical and seismic tests (e.g. Crosshole) are more advantageous compared to drilling boreholes because:

- They are mostly non destructive and do not require drilling or banning/confining an area for a long period of time.
- They provide extensive and consistent data regarding the area under investigation.
- They are more economical compared to other exploration methods.

3- Considering the soil nonlinear behavior, its stiffness is reduced with an increase in the level of the applied stress; effects of this reduction can be taken into account through the related presented equations and models. It is possible to rewrite Schmertmann relation with an assumed strain influence factor and present a relation for the prediction of the settlement of foundations constructed on granular soil using Elasticity Theory-based formulae and stiffness relations for small strains (maximum stiffness), and considering the existing stress level.

4- To validate, the results of the proposed relation were compared with those found by Hydro Tests performed under 4 storage tanks (in South Pars Site). The acceptable conformity of the results of the proposed relation to the real (registered) ones shows that the proposed relation is quite acceptably precise (compared to other methods).

5- Results have shown that using H/B = 0.5 in Bowles (1987) relation, yields quite acceptable results regarding the settlement of the storage tanks in question; these results are in good conformity with those of the present research ($I_z = 0$ for $Z = 0.5B$); this is another point that verifies the value of I_z proposed in this research.

6- Based on the relation proposed for the determination of I_z, it can be stated that I_z variations with depth are not constant meaning that they depend on the applied stress and the shear wave velocity at the desired depth; therefore, it can be concluded that the depth of the compressible layer has a controlling effect on the value of I_z.
7- Compared with the methods proposed by Janbu et al. (1956), Schmertmann (1970,1978) Taylor et al. (1983), and Terzaghi et al. (1996), the one presented by Haddad et al. (2013) yields acceptable results.
8- Geophysical and seismic methods are quite efficient in predicting the soil in-situ parameters for the determination of settlements and yield more realistic results.

REFERENCES

API 650 (American Petroleum Institute).(2000). Welded Tanks for Oil Storage,Appendix-B.Tenth Edition, November 1998,Addendum 1, January 2000 Addendum 2. API,Washington, D.C.

API 653 (American Petroleum Institute).(2000). Tank Inspection, Repair, Alteration, and Reconstruction ,Appendix-B.Tenth Edition, November 1998,Addendum 1, January 2000 Addendum 2. API,Washington, D.C.

Bowles, J. E. (1987). "Elastic foundation settlement on sand deposits." J. Geotech. Eng, Vol. 113 (8): 846-860.

Clayton, C. R. I. (2011). "Stiffness at small strain." research and practice. Geotechnique, Vol. 61 (1): 5-37.

Hadad, A., and Ahidashti, R. A. (2013). "Prediction of immediate settlement of shallow foundation over granular soils using small-strain stiffness." Journal of American Science Vol. 9 (6): 480-489.

Janbu, N., Bjerrum, L., and Kjaernsli, B. (1956). "Vetledning ved Losning au Fundamentering Soppgauer."NGI,Norwegian Geotechnical Institute Publication No.16.

M.Fahey, and Carter, J. P. (1993). "A finite element study of the pressuremeter test in sand using a nonlinear elastic plastic model." Can.Geotech. J, Vol. 30 (2): 348–362.

Rollins, K. M., Evans, M. D., Diehl, N. B., and III, W. D. D. (1998). "Shear Modulus and Damping Relationships for Gravels."J. geotech. geoenviron. eng. City: 10.1061/(ASCE)1090-0241(1998)124, pp. 396-405.

Schmertmann, J. H. (1970). "Static cone to compute static settlement over sand." J Soil Mech Found Div, Vol. 96 (3): 1011–1043.

Schmertmann, J. H., Brown, P. R., and Hartman, J. P. (1978). "Improved strain influence factor diagrams." J Geotech Eng Div,ASCE, Vol. 104 (8): 1131–1135.

Seed, H. B., Wong, R. T., Idriss, I. M., and Tokimatsu, K. (1986). "Moduli and damping factors for dynamic analyses of cohesionless soils." J. geotech. geoenviron. eng., Vol. 112 (11): 1016-1032.

Sivakugan, N., and Das, B. M. (2010). Geotechnical Engineering: a practical problem solving approach.: J. Ross.

Taylor, B. B., and Matyas, E. L. (1983). "Influence Factors for Settlement Estimates of Footings on Finite Layers." Can Geotech J, Vol. 20: 832-835.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). Soil mechanics in engineering practice, John Wiley & Sons Inc,New York.