Constitutive modeling of cemented gravely sands including the effects of cement type

*A. Hamidi, P. Yarbakhti:
School of Engineering, KharazmiUniversity

Received: 26 Aprill 2011 Revised 25 Oct 2011

Abstract

In this paper, a constitutive model is proposed for prediction of the shear behavior of a gravely sand cemented with different cement types. The model is based on combining stress-strain behavior of uncemented soil and cemented bonds using deformation consistency and energy equilibrium equations. Cement content and cement type are considered in a model as two main parameters. Based on the proposed method, the behavior of cemented soil with different cement types is predicted for conventional triaxial test condition. Porepressure developed during undrained loading besides volumetric strains in drained condition are also modeled according to this framework. Comparison of model results with experimental data indicates its reasonable accuracy.

KeyWords: Constitutive model, Cemented soil, Gravely sand, Stress-strain behavior, Cement Content, Cement type.

*Corresponding author hamidi@tmu.ac.ir

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

Introduction

Slopes and vertical cuts are usually observed to be stable for a long time in coarse-grained alluviums. Stability is often attributed to the cementation effects producing increased shear strength in these deposits. Due to the problems associated with preparation of undisturbed samples from these soils, cementation effects are usually studied using artificial cementation. Experimental studies on behavior of cemented sands have been reported by several researchers like Clough et al. (1981), Leroueil and Vaughan (1990), Coop and Atkinson (1993), Ismail et al. (2002) and Consoli et al. (2006).

Many researchers have worked on constitutive modeling of the behavior of cemented sands, e.g. Pekau and Gocevski (1989), Reddy and Saxena (1992), Lagioia and Nova (1995), Vatsala et al. (2001), Vaunat and Gens (2004). In the present study, a constitutive model is developed for cemented gravely sand of Tehran alluvium. Figure 1 shows gradation curve and Table 1 indicates physical properties of the representative soil. Model results are compared with three different sets of experimental data to investigate its ability. Data include triaxial tests conducted by Asghari et al. (2003), Hamidi et al. (2004) and Haeri et al. (2005). Stress-strain behavior in drained and undrained conditions besides pore pressure and volumetric strains are modeled and results showed good consistency for propose model.

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

Fundamentals of model

Vaunat and Gens (2004) and Gens et al. (2007) separated the shear behavior of an argillaceous rock into individual behaviors of uncemented soil and cemented bonds. They used deformation consistency between two parts to model the mechanical behavior of an argillaceous rock by following equations:

$$p = (1+\chi)p_m + \chi(p_b) \tag{1}$$

$$q = (1+\chi)q_m + \chi(q_b) \tag{2}$$

where p_m and q_m are the mean and deviatoric effective stresses for uncemented soil matrix. Also p_b and q_b are the corresponding values for cemented bonds. Parameter χ is a coefficient which controls the contribution of each component in mean effective stress (p) and deviatoric stress (q) of cemented soil. It can be determined using the following equation:

$$\chi = \chi_0 e^{-L} \tag{3}$$

where χ_0 is the initial value of χ when there is no damage to the bonds and can be determined by model calibration. Value of damage parameter, L, can be determined using the following equation:

$$L = Ln\left(\frac{k_{b0}}{k_b}\right) \tag{4}$$

Where k_{b0} is the stiffness of cemented bonds in zero confining stress and k_b is its value in other confinements.

This framework is used as the basis of model in present study. The stress-strain behavior of the uncemented gravely sand is predicted using generalized plasticity model proposed by Pastor et al. (1985).

Also a new model is suggested for the behavior of cemented bonds. These two parts are combined to determine stress-strain behavior of cemented soil.

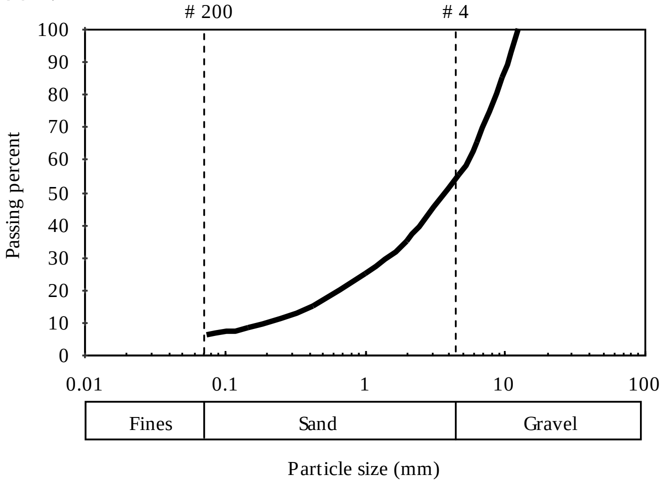

Fig1. Gradation curve of sandy gravel representative of Tehran alluvium (Haeri et al. 2002)

Table1. Physical characteristics of gravely sand representative of Tehran alluvium

Parameter	Value
G_s	2.58
D_{50} (mm)	4.0
D_{10} (mm)	0.2
Fine content (%)	6
Sand content (%)	49
Gravel content (%)	45
PL (%)	12
LL (%)	25
□$_{min}$ (kN/m^3)	16.0
□$_{max}$ (kN/m^3)	18.74

Modeling the behavior of cemented bonds

The isotropic yield strength of bonds can be determined by extension of triaxial test results to high confinements. Figure 2 indicates variation

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

of the isotropic yield strength of bonds with cement content for different cement types as follows:

$$p_{bf} = r \times cc^{s} \tag{5}$$

where p_{bf} is the isotropic yield strength of bonds and cc indicates the cement content in percents. Model parameters r and s are dependent to cement type and can be determined by model calibration.

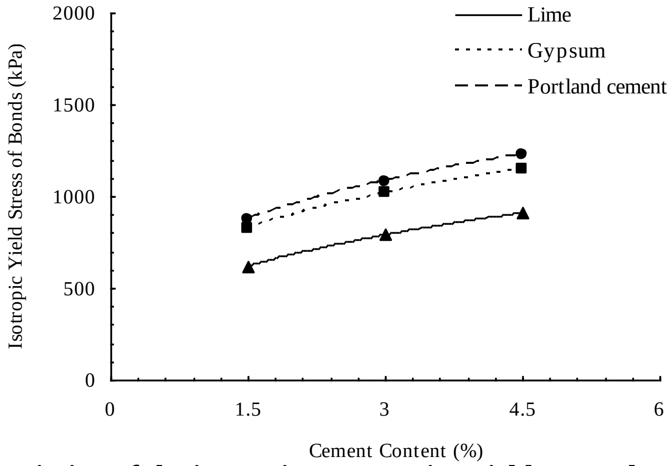

Fig 2. Variation of the isotropic compression yield strength of cemented bonds with cement content.

The following equation is considered as yield envelope of cemented bonds in present study:

$$q_b = q_{bi}(1 - \frac{p_0}{p_{bf}})^{a} \tag{6}$$

where q_b is bond strength in any confinement, p_0. Model parameter "a" controls shape and curvature of yield surface. A linear yield envelope results in unit value for "a". However, it can be considered as a function of cement content.

q_{bi} is the bond strength in zero confinement and is considered as a linear function of cement content. Figure 3 shows variation of bond

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

strength in zero confinement with cement content for different cement types. According to the figure, bond strength in zero confinement can be estimated as a function of cement content as follows:

$$q_{bi} = z \times cc \tag{7}$$

Model parameter z is determined using model training procedure.

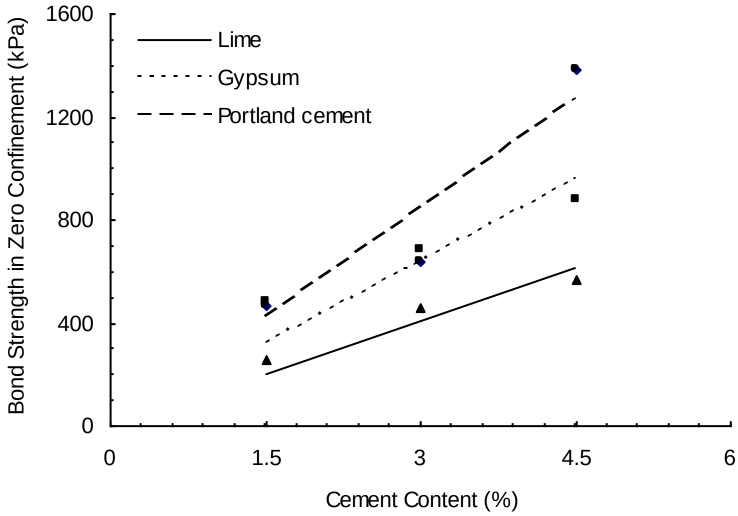

Fig3. Change of bond strength in zero confinement with cement content and cement type

Also the following equation is considered for bond stiffness:

$$k_b = k_{bi}(1 - \frac{p_0}{p_{bf}})^b \tag{8}$$

Figure 4 shows variation of bond stiffness in zero confining stress with cement content. As the figure shows, bond stiffness increases after a cement content of about 0.75% for different cement types. This is the threshold value for zero stiffness of cemented bonds. The trend shown in this figure can be mentioned using the following equation:

$$k_{bi} = \eta(cc - \beta) \tag{9}$$

Parameters η and β can be determined using a regression procedure during model calibration. Model parameter "b" controls the rate of change in bond stiffness and can be considered as a constant.

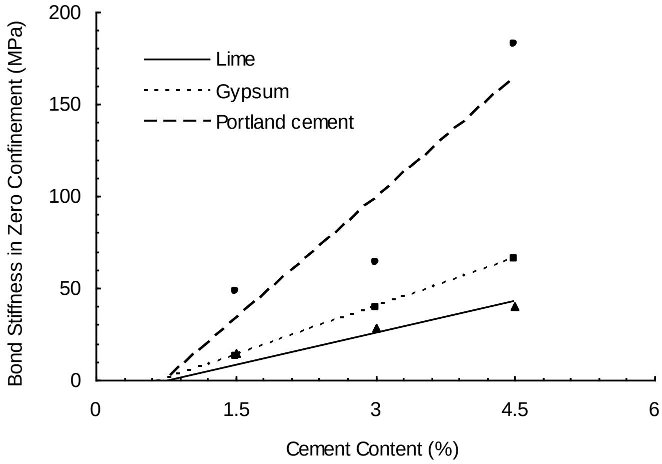

Fig4. Variation of bond stiffness with cement content

A linear elastic stress-strain behavior is considered for cemented bonds before failure. As a result, the strain associated to failure point, ε_f, can be calculated by the following equation:

$$\varepsilon_f = \frac{q_b}{k_b} \tag{10}$$

After failure the strength of bonds decreases. Rate of reduction in bond strength is considered as follows:

$$q_b = q_{bi} \exp[-\alpha(\varepsilon - \varepsilon_f)] \tag{11}$$

where ε is the axial strain and α is a model parameter. Damage parameter L can be determined as follows:

$$L = \ln\{\frac{k_{b0}}{\alpha q_{bi} \exp[-\alpha(\varepsilon - \varepsilon_f)]}\} \tag{12}$$

Modeling the behavior of uncemented soil

Experimental studies on the uncemented gravely sand have been reported by Asghari et al. (2003). Generalized plasticity model of Pastor et al. (1985) is used for modeling the behavior of uncemented part of soil. Details and results of modeling are reported in Haeri and Hamidi (2009) which shows fairly good predictions of the model. It uses the flow rule initially suggested by Frossard (1983) as follows:

$$d_g = \frac{d\varepsilon_v^p}{d\varepsilon_s^p} = (1+\theta)(M_g - R) \tag{13}$$

In this equation d_g is the rate of dilation, $d\varepsilon_v^p$ and $d\varepsilon_s^p$ are the increments of plastic volumetric strain and plastic shear strains respectively, θ is a constant which can be determined by model calibration, M_g is the slope of critical state line, and R is the stress ratio, q/p. The model uses a non associated flow rule. Equations of yield and plastic potential surfaces are as follows:

$$f = \{q - M_f\, p(1+\frac{1}{\theta})[1-(\frac{p}{p_c})^{\theta}]\} \tag{14}$$

$$g = \{q - M_g\, p(1+\frac{1}{\theta})[1-(\frac{p}{p_g})^{\theta}]\} \tag{15}$$

In these equations p_c and p_g are the isotropic yield stresses associated to each surface. The value of M_f can be related to M_g using the soil density, D_r by the following equation:

$$M_f = D_r M_g \tag{16}$$

Model results for cemented soil

Model results for stress-strain behavior of cemented soil are presented for drained and undrained conditions separately.

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

1. Drained condition

Stress-strain behavior of cemented bonds and uncemented soil are combined using Equations (1) and (2) to determine stress-strain behavior of cemented soil. Comparison of model results with experimental data is shown in Fig 5 for different cement types. Results show satisfactory predictions of model for stress-strain behavior of cemented soil. Peak shear stress and failure strain are predicted in a good manner. After peak and ultimate shear strengths are simulated well. However there are some differences between predicted initial stiffness with experimental results. As it can be observed, initial stiffness is usually overestimated by model.

χ_0 and α are two parameters in stress-strain modeling which should be determined by model calibration. As the figure shows, experimental data for three confining pressures of 25, 100 and 500 kPa are used for training of model and determination of parameters.

χ_0 can be considered as a function of cement content and confining stress by the following equations:

$$\chi_0^{l,d} = (0.4cc + 3) \times \exp[(0.0003cc - 0.0003)p_0] \qquad (17)$$

$$\chi_0^{g,d} = (35) \times \exp[(-0.002)p_0] \qquad (18)$$

$$\chi_0^{p,d} = (0.9cc + 8) \times \exp[(0.0006cc - 0.004)p_0] \qquad (19)$$

$\chi_0^{l,d}$, $\chi_0^{g,d}$ and $\chi_0^{p,d}$ are χ_0 values for lime, gypsum and Portland

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

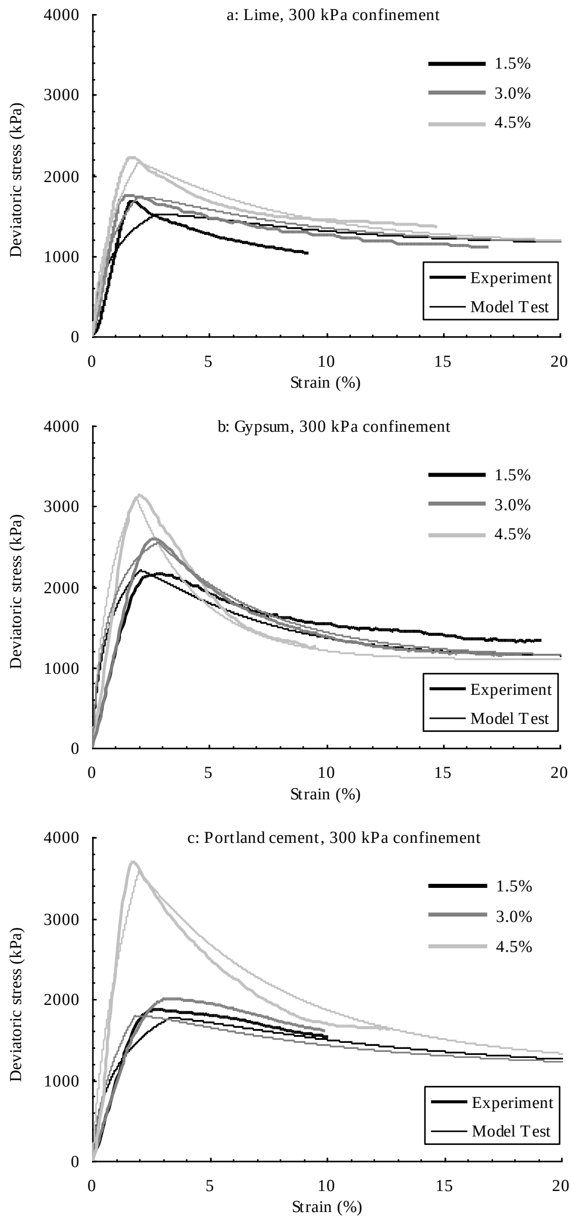

Fig 5. Model results for stress-strain behavior of cemented soil in drained condition

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

cement agents respectively in drained condition. Cement content, cc, should be used in percents and confining stress, p_0, should be applied in kPa. As it can be seen, χ_0 decreases with reduction of cement content or increase in confining stress which indicates less contribution of bonds in shear strength of cemented soil.

Parameter α controls the rate of reduction in shear strength after yield point. It can also be interpreted as a function of cement content and confining stress as follows:

$$\alpha^{l,d} = 22 - 0.02 p_0 \tag{20}$$

$$\alpha^{g,d} = 6 + 0.01 p_0 \tag{21}$$

$$\alpha^{p,d} = 16 - 0.01 p_0 \tag{22}$$

$\alpha^{l,d}$, $\alpha^{g,d}$ and $\alpha^{p,d}$ are α values for lime, gypsum and Portland cement agents respectively in drained condition. The model is tested for a confinement of 100 kPa which reveals accuracy of suggested expressions for other confinements.

2. Undrained condition

Results of modeling for uncemented soil and cemented bonds in undrained state are combined using Equations (1) and (2) to determine stress-strain behavior of cemented soil in undrained condition. χ_0 and α are calculated using model training as follows:

$$\chi_0^{l,u} = 6.2 - 0.009 p_0 + 0.00001 {p_0}^2 \tag{23}$$

$$\chi_0^{g,u} = 7.8 + 0.01 p_0 - 0.00002 {p_0}^2 \tag{24}$$

$$\chi_0^{p,u} = 7.4 + 0.02 p_0 - 0.00008 {p_0}^2 \tag{25}$$

$$\alpha^{l,u} = 1.3 \tag{26}$$

$$\alpha^{g,u} = 5.0 \tag{27}$$

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

$$\alpha^{p,u} = 1.6 \tag{28}$$

$\chi_0^{l,u}$, $\chi_0^{g,u}$ and $\chi_0^{p,u}$ are χ_0 values for lime, gypsum and Portland cement agents respectively in undrained condition. Also $\alpha^{l,u}$, $\alpha^{g,u}$ and $\alpha^{p,u}$ are associated α values. Figure 6 shows results of modeling for soil cemented with 3% of lime, gypsum and Portland cement. Comparison with experimental data shows its acceptable predictions.

Pore pressure and volume change behavior

The following expression is used in present study to evaluate pore pressure in cemented bonds:

$$u_b = C(1 - \exp(-\mu\varepsilon)) \tag{29}$$

As indicated in this equation, pore pressure in cemented bonds, u_b, can be approximated using two additional parameters C and μ. Parameter C indicates pore pressure at large axial strains. Pore pressure in cemented bonds cannot be measured directly. As a result, these parameters should be determined by model calibration for test results on cemented soil.

Pore pressure values in cemented bonds are combined with associated values in uncemented matrix using the following equation:

$$u = (1 + \chi)u_m + \chi u_b \tag{30}$$

In this equation u and u_m are pore pressure values in cemented and uncemented soil respectively. C and μ are determined by model training for three confinements of 25, 100 and 500 kPa as follows:

$$C^l = 270 + 0.1p_0 + 0.0007p_0^{\ 2} \tag{31}$$

$$C^g = 440 + 0.1p_0 + 0.0002p_0^{\ 2} \tag{32}$$

$$C^p = 190 + p_0 - 0.002p_0^{\ 2} \tag{33}$$

$$\mu^{l} = 0.2 + 0.0006 p_0 \tag{34}$$

$$\mu^{g} = 0.2 + 0.0003 p_0 \tag{35}$$

$$\mu^{p} = 0.4 \tag{36}$$

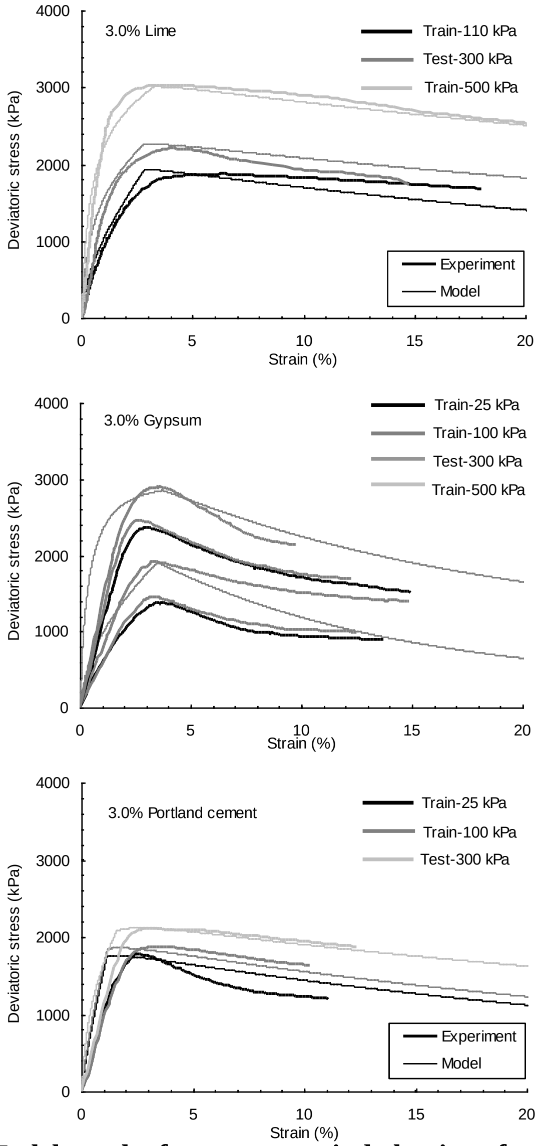

Fig6. Model results for stress-strain behavior of cemented soil in undrained condition

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

In these equations C^l, C^g and C^p are C values for lime, gypsum and Portland cement respectively and μ^l, μ^g and μ^p indicate associated μ values. Result of pore pressure modeling is shown in Fig 7 including model train and model test data for soil cemented with different cement types. Figures show fairly good agreement between experimentttal data and modeling results. Initial positive pore pressure due to the soil compression and afterward suction besides ultimate pore pressure are also modeled satisfactory.

Also deformation consistency equation for cemented soil can be written as follows:

$$d\varepsilon_v = d\varepsilon_{vm} + d\varepsilon_{vb} \tag{37}$$

In this equation, $d\varepsilon_v$ is volumetric strain in cemented soil. Corresponding values for uncemented soil and cemented bonds are $d\varepsilon_{vm}$ and $d\varepsilon_{vb}$, respectively.

Volumetric strains in uncemented soil can be related to the induced pore pressures in undrained state using rebound modulus by the following equation:

$$\varepsilon_{vm} = \frac{u_m}{E_{rm}} \tag{38}$$

In this equation E_{rm} is the rebound modulus for uncemented soil. Average value of rebound modulus for uncemented soil is estimated about 5500 kPa using results of triaxial tests in different confining stresses.

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

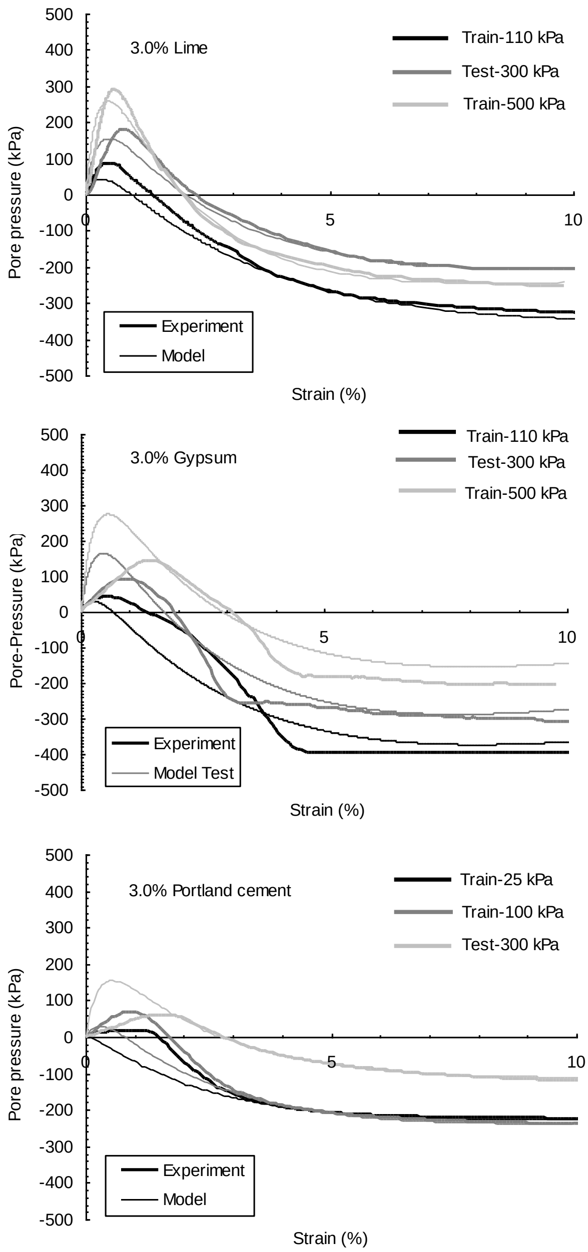

Fig7. Model results for pore pressure changes of cemented soil in undrained condition

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

Pore pressure and volumetric strains in cemented bonds are also related in a similar manner using rebound modulus of cemented bonds, E_{rb} which is determined by model training and is shown in Fig 8 for three different cement types. Model results for volumetric strains are shown in Fig 9. The figure shows its ability for prediction of volumetric strains especially in large strains.

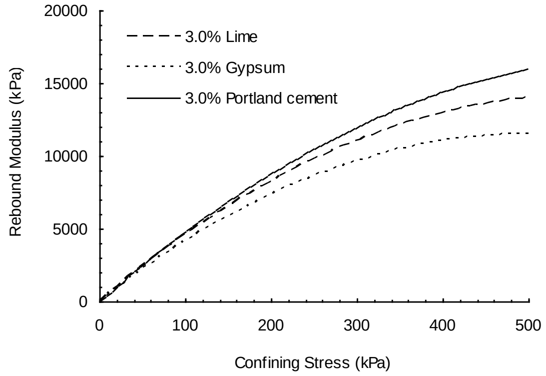

Fig8. Rebound modulus of cemented bonds for soil cemented with 3.0% of different cement types

Model parameters

Expressions for determination of five parameters χ_0, α, C, μ and E_{rb} based on cement content and confining pressure are presented in text. Values of other parameters are summarized in Table2 for different cement types. These parameters are determined based on calibration process. The model is based on 13 parameters with specified mechanical description. Model calibration for parameters needs experimental data which are mainly derived from triaxial test or other conventional soil mechanics experiments.

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

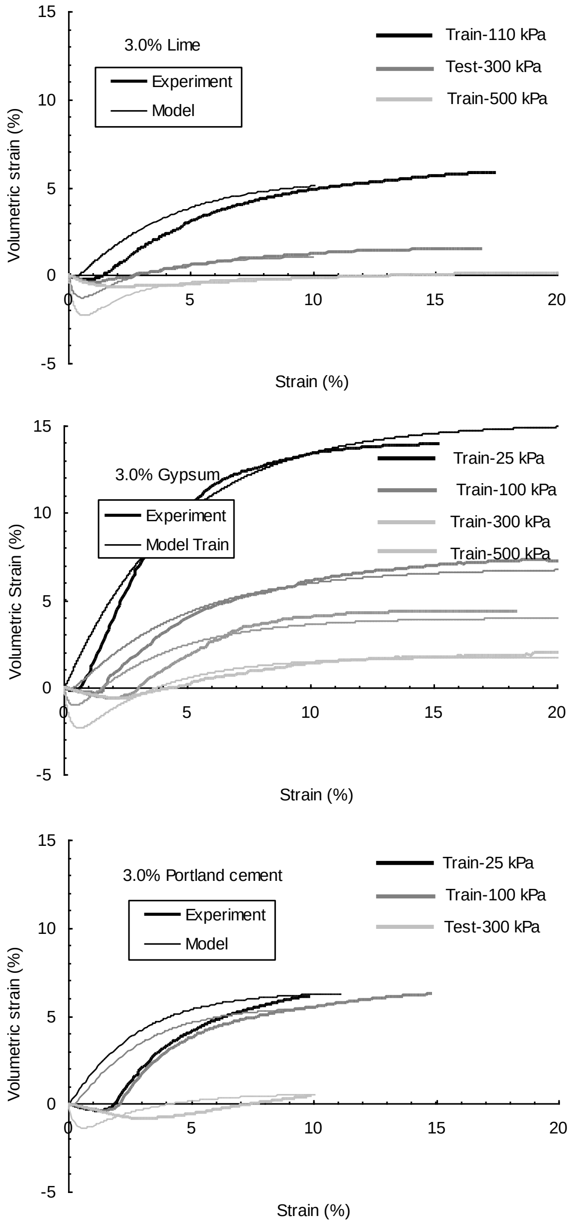

Fig9. Model results for volumetric strains of cemented soil in drained condition

[DOR: 20.1001.1.22286837.1391.6.2.4.8]

Table2. Values of Parameters used in constitutive model

Parameters	Lime	Gypsum	Portland cement
r	540	730	770
s	0.35	0.30	0.31
z	137	213	281
□	11.5	17.5	45.0
□	0.76	0.75	0.78
a^*	0.15+0.01cc	1.5+0.10cc	0.6+0.05cc
b^*	0.8+0.01cc	2.1+0.30cc	2.2+0.30cc
E_{rm}^{**}	5.5	5.5	5.5

Summary and conclusion

A constitutive model developed for interpretation of the mechanical behavior of a gravely sand cemented and is tested for the soil cemented with different cement types. Comparison of modeling results with experimental data showed that the model predicts the mechanical behavior of cemented soil with an acceptable accuracy. As the cement content increases in cemented soil, it acts as the filler of voids rather than effective bonding between soil grains. The presented model is able to predict the behavior of cemented soil in cement contents less than this threshold value.

Acknowledgements

The authors would like to thank Prof. S. Mohsen Haeri from Sharif University of Technology for his kind guidelins and providing the experimental data.

References

1. Asghari E., Toll D. G. and S. M. Haeri, "Triaxial behavior of a cemented gravely sand", Tehran alluvium, Geotechnical and Geological Engineering Journal, Vol. 21, No. 1 (2003) 1-28.

2. Clough G. W., Sitar N., Bachus R. C. and N. S. Rad, "Cemented sands under static loading", Journal of Geotechnical Engineering ASCE, Vol. 107, No. 6 (1981) 799-817.
3. Consoli N. C., Rotta G. V. and P. D. M. Prietto, "Yieldingcompressi-bility-strength relationship for an artificially cemented soil under stress Géotechnique", Vol. 56, No. 1 (2006) 69-72.
4. Coop M. R. and J.H. Atkinson, "The mechanics of cemented carbonate sands Géotechnique", Vol. 43, No. 1 (1993) 53-67.
5. Frossard E., "Une equation d'ecoulement simple pour les materiaux granulaires Géotechnique", Vol. 33, No. 1 (1983) 21-29.
6. Gens A., Vaunat J., Garitte B. and Y. Wileveau, "In situ behaviour of a stiff layered clay subject to thermal loading: observations and interpretation Géotechnique", Vol. 57, No. 2 (2007) 207-228.
7. Haeri, S. M., Yasrebi, S. and E. Asghari, "Effects of Cementation on the Shear Strength Parameters of Tehran Alluvium Using Large Direct Shear Test, In Proceedings of 9th IAEG Congress, Durban", South Africa, (2002) 519-525.
8. Haeri S. M., Hosseini S. M., Toll D. G. and S. S. Yasrebi, "The behavior of an artificially cemented sandy gravel Geotechnical and Geological Engineering Journal", Vol. 23, No. 5 (2005) 537-560.
9. Haeri S. M. and A. Hamidi, "Constitutive modeling of cemented gravely sands Geomechanics and Geoengineering", An International Journal, Vol. 4, No. 2 (2009) 123-139.
10. Hamidi, A., Haeri, S. M. and N. Tabatabaee, "Influence of Gypsum Cementation on the Shear Behavior of Cemented Sands. In 1st NCCE", Sharif University of Technology, Iran (2004).

11. Ismail M. A., Joer H. A., Sim W. H. and M. F. Randolph, "Effect of cement type on shear behavior of cemented calcareous soil Journal of Geotechnical and Geoenvironmental Engineering ASCE", Vol. 128, No. 6 (2002) 520-529.

12. Lagioia R. and R. Nova, "An experimental and theoretical study of the behavior of calcarenite in triaxial compression Géotechnique", Vol. 45, No. 4 (1995) 633-648.

13. Leroueil S. and P. R. Vaughan, "The general and congruent effects of structure in natural soils and weak rocks. Géotechnique", Vol. 40, No. 30 (1990) 467-488.

14. Pastor M., Zienkiewicz O. C. and K. H. Leung, "Single model for transient soil loading in earthquake analysis", II: Non associative models for sands International Journal of Numerical and Analytical Methods in Geomechanics, Vol. 9 (1985) 477-498.

15. Pekau O. A. and V. Gocevski, "Elasto-plastic model for cemented and pure sand deposits Computers and Geotechnics", Vol. 7 (1989) 155-187.

16. Reddy K. R. and K. S. Saxena, "Constitutive modeling of cemented sand Mechanics of Materials", Vol. 14 (1992) 155-178.

16. Vaunat J. and A. Gens, "Aspects of modelling geotechnical problems in hard soil and soft argillaceous rocks", In: Pande G, Pietruszczak S (eds) In NUMOG IX (2004) 37-43.

17. Vatsala A., Nova R. and B. R. Srinivasa Murthy, "Elastoplastic model for cemented soils", Journal of Geotechnical and Geoenvironmental Engineering ASCE, Vol. 127, No. 8 (2001) 679-687.