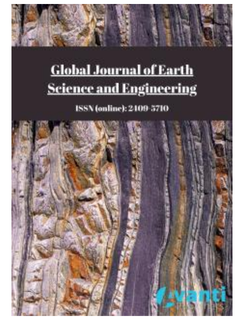
Global Journal of Earth Science and Engineering, 2025, 57-65

Published by Avanti Publishers

Global Journal of Earth Science and Engineering

ISSN (online): 2409-5710



Insight into Employability of Mohr Coulomb and Hardening Soil Model in the Undrained Analysis

Dhanaji S. Chavan*

Department of Civil Engineering, Walchand College of Engineering Sangli, Maharashtra, India

ARTICLE INFO

Article Type: Research Article

Academic Editor: Muhamad Yusa

Keywords:
Undrained analysis
Pore water pressure
Hardening soil model
Mohr coulomb model

Timeline:
Received: July 05, 2025
Accepted: August 28, 2025
Published: September 27, 2025

Citation: Chavan DS. Insight into employability of mohr coulomb and hardening soil model in the undrained analysis. Glob J Earth Sci Eng. 2025; 12: 57-65.

DOI: https://doi.org/10.15377/2409-5710.2025.12.4

ABSTRACT

Quite often soil at the site is saturated with water. Any addition of the load onto such soil with low permeability results in generation of excess pore water pressure. Real life problems such as construction of footing, construction of embankment on saturated clayey soil and deep excavation falls under this category. The numerical modelling of such problems must be capable of simulating generation of excess pore water pressure accurately. Mohr Coulomb model is the most widely used constitutive model in the numerical analysis of soil problems. However, being elastic-perfectly plastic, application of this model in simulation of short term undrained simulations needs to be investigated. In the present study, efficacy of the Mohr Coulomb model and Hardening Soil model present in the PLAXIS have been investigated for coupled analysis. It is found that Mohr Coulomb model overestimates the undrained shear strength of the soil significantly and fails to produce the realistic stress path. Hardening soil model captured the realistic pore pressure response whereas the Mohr Coulomb model underestimated the pore pressure and resulted in unrealistic pore pressure evolution. It is recommended that advanced constitutive models such as Hardening Soil model should be employed in short term undrained numerical analysis.

*Corresponding Author
Email: dhanaji.chavan@walchandsangli.ac.in

1. Introduction

Many times soil at the site is saturated with water. If permeability of such soil is low, as in case of clay, silt and sandy silt; construction of footing or any other surcharge over such soils results in undrained monotonic loading of the subsoil [1-3]. Stability of the footing/surcharge immediately after construction is critical due to generation of excess pore water pressure [2, 4-6]. Therefore, undrained shear strength and evolution of excess pore pressure needs to be given due consideration in the analysis of such problems.

Nowadays, numerical modelling is widely used to assess stability and settlement of the structures resting on the soils [7-11]. The reliability of the numerical results depends upon the constitutive model used to simulate the soil stress strain response [12-16]. Mohr Coulomb (MC) model is the most widely used constitutive model in the numerical analysis due to its simplicity and less number of material parameters [17-20]. Being linear elastic perfectly plastic, this model fails to capture most typical hardening response of the soil observed at the site [12, 20-23]. Therefore, use of Mohr Coulomb model in the undrained analysis needs to be investigated. In the present study, MC model and Hardening Soil (HS) model available in the PLAXIS have been investigated in respect of pore water pressure evolution, undrained shear strength and effective stress path. Light is also shed on the advantages and limitations of these models. Further, the care that has to be exercised while using these models is also highlighted. The present study sheds light on the importance of choosing appropriate constitutive model in numerical analysis so that real life problems such as construction of footing on saturated soil, construction of embankment on saturated clayey soil and deep excavations in clayey/silty clay/clayey silt, can be modelled realistically. The realistic modelling results in realistic deformations/settlements which helps in the design of such projects.

2. Superiority of Hardening Soil Model over Mohr Coulomb Model

Mohr Coulomb model is a widely used constitutive model in the field of soil mechanics. It is a linear elastic perfectly plastic material model. In this model, the elastic response is governed by Young's modulus and Poisson's ratio and the plastic response is governed by the angle of internal friction and cohesion. Further, yield criteria and failure criteria are same in this model [24]. That is in another words, material fails when it reaches yielding.

Experimentally, it is observed that the soil shows nonlinearity and plasticity right from the very small deformation [25-28]. Further, the stiffness of the soil is stress dependent [29-34]. That is, the secant modulus/tangent modulus of the stress strain curve conducted at different confining pressure is different. This is called as stress (pressure) dependency of the stiffness. In HS model, this effect is incorporated in the formulation of the model whereas in Mohr coulomb model there is no provision to incorporate stress dependency of the stiffness. In other words, in case of Mohr Coulomb model, for each confining pressure we have to enter different value of stiffness and run the simulation separately. In case of Hardening Soil model, only one value of secant modulus is enough. Using the stress dependency parameter m, stiffness at all other confining pressure is automatically calculated. In Mohr Coulomb model, yield surface is fixed whereas in Hardening soil model it expands with straining.

3. Modelling of Triaxial Test

In the present study, the Isotropically Consolidated Undrained Triaxial (ICUT) test is simulated using the MC and HS models available in PLAXIS. The triaxial specimen is modelled as axisymmetric model as shown in Fig. (**1**). The dimensions of the domain are kept to be 1 m × 1 m. The boundary conditions are also shown in the Fig. (**1**).

To begin with, the triaxial specimen is modelled with HS model and the constitutive parameters for this purpose are taken from Surarak *et al.* [35]. These parameters are given in Table **1**. It should be noted that the unit weight of the soil is defined as zero in the analysis. This helps to choose the domain dimensions of our choice since the self-weight components does not enter into the analysis. The initial effective confining pressure considered is 138 kPa. The excess pore water pressure evolution obtained from the present simulation for the HS model is compared with the experimental response from Surarak *et al.* [35] in the Fig. (**2**). It is observed that there

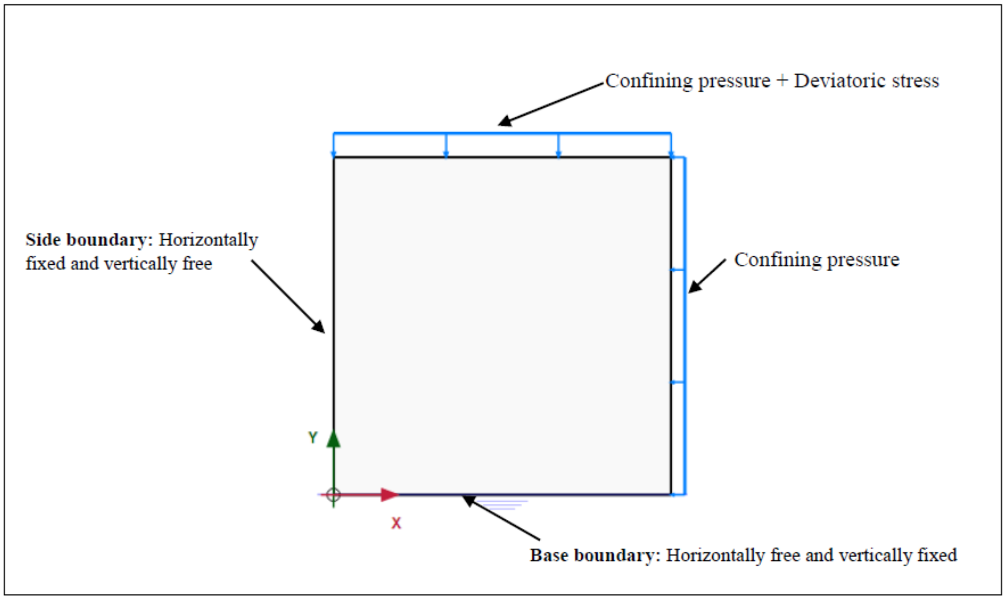


Figure 1: The axisymmetric model of the triaxial specimen.

Table 1: The constitutive parameters used in the analysis [35].

ϕ' (deg)	ψ' (deg)	C' (kPa)	E^{ref}_{50} (kPa)	E^{ref}_{oed} (kPa)	E^{ref}_{ur} (kPa)	R_F	m	E^{nr}_{0}	V_{ur}
27	0	1	800	850	8000	0.9	1	0.74	0.2

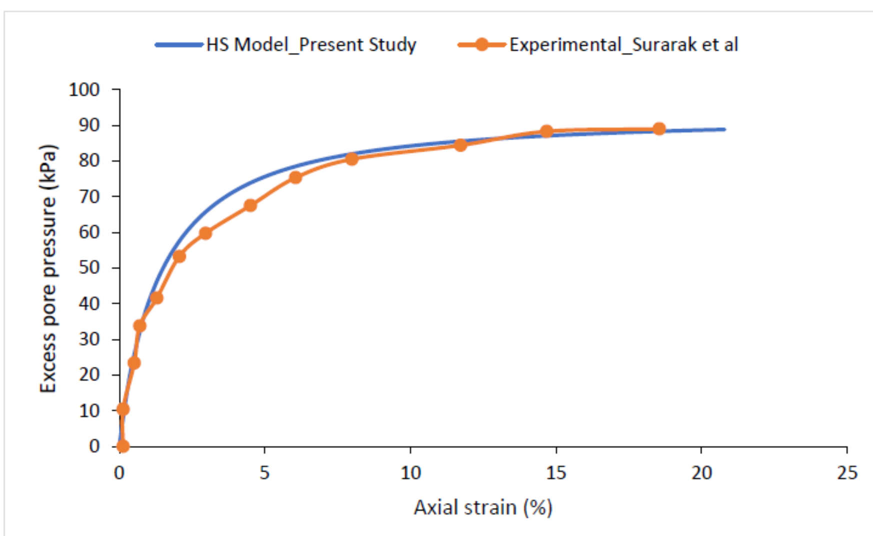


Figure 2: Comparison of the excess pore water pressure evolution from this study with that from Surarak *et al*. [35].

is an excellent agreement between the two responses. The objective of this comparison was to ensure that the triaxial simulation performed in the present study was modelled correctly. Fig. (**3**) shows the comparison between the pore water pressure obtained at the top and bottom of the domain. The pore pressure responses are identical only. This further ensures accuracy of the simulation performed in present study.

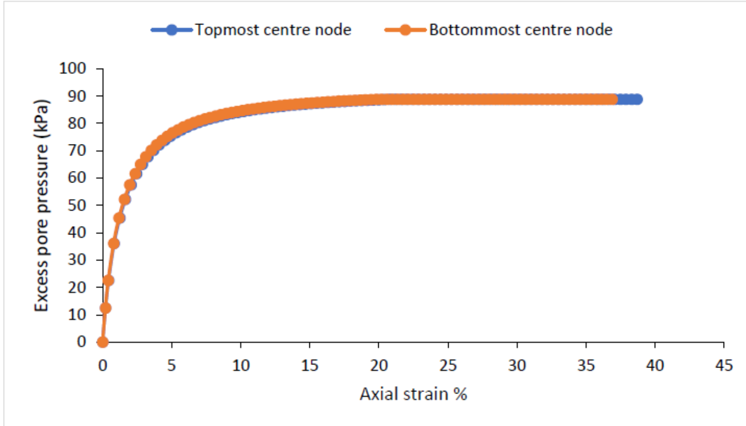


Figure 3: Comparison of excess pore water pressure recorded at the top and bottom of the axisymmetric model (HS model).

4. Results and Discussion

4.1. Findings

The triaxial test was simulated at the confining pressure of the 138 kPa using HS and MC model. The constitutive parameters for the HS model were taken from Table **1**. The elastic modulus (800 kPa), friction angle (27°) and cohesion (1 kPa) for Mohr Coulomb model were also taken from same table. This was done because here objective was to know what happens to the response when elastic modulus and shear strength parameters are same but the constitutive models are distinct. The Poisson's ratio was chosen to be 0.3. The effective stress paths are shown in Fig. (**4**). It is clear that the effective stress path for MC model is vertical and that from HS model is curved one and moves leftward. Both paths ultimately reach to the failure envelope. However, the deviatoric stress at failure for MC model is 151 kPa and that for HS model is 80 kPa. In other words, the strength predicted by MC model is 1.89 times that from HS Model. The pore pressure evolution is also compared in the Fig. (**5**). The pore pressure evolution is nonlinear for HS model whereas that for MC model is almost bilinear. The rate of pore pressure evolution is very high in HS model. The maximum pore pressure developed in HS case is 89 kPa. Out of this 89 kPa, 75 kPa is developed at strain less than 5%. In case of the MC model, the maximum pore pressure developed is around 50 kPa and is attained at strain of 18%.

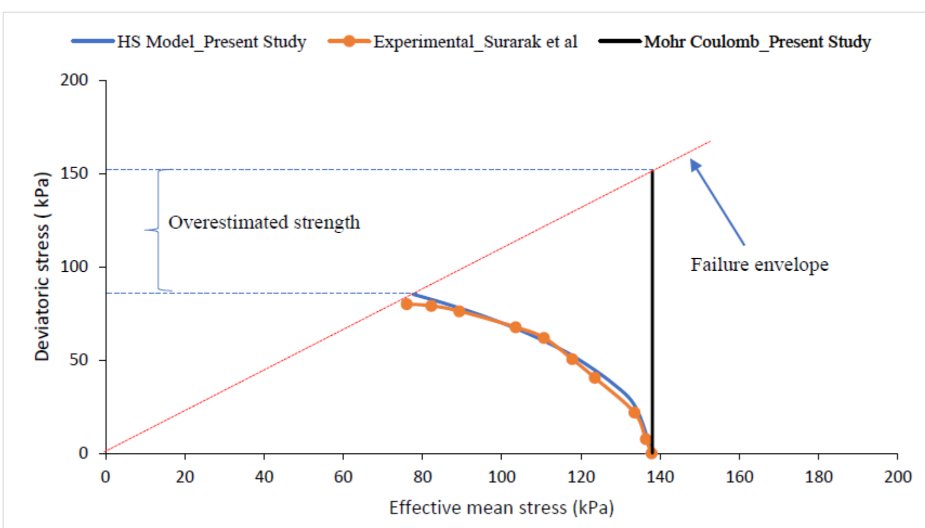


Figure 4: Overestimation of undrained shear strength by Mohr Coulomb model.

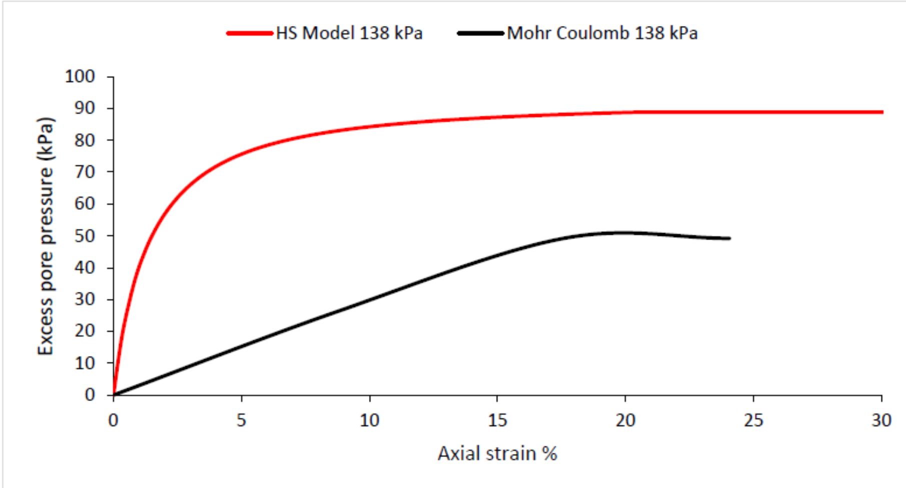


Figure 5: Comparison of the pore pressure evolution.

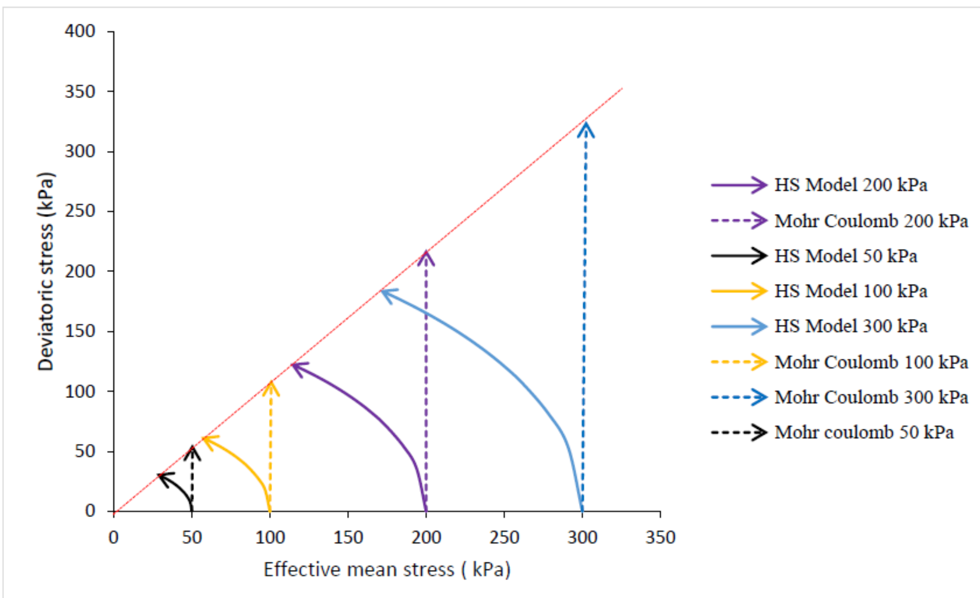


Figure 6: Effect of confining pressure on effective stress path and undrained shear strength.

Triaxial test simulations were also performed at the following initial effective confining pressures: 50 kPa, 100 kPa, 200 kPa and 300 kPa. The effective stress path and excess pore pressure evolutions are shown in Fig. (**6-10**). From Fig. (**6**), it is observed that stress path for Mohr Coulomb model are vertical irrespective of the initial effective confining pressure whereas the stress paths for HS model are curved one and moves leftward. Further, irrespective of the initial effective confining pressure, the Mohr Coulomb model always estimates significantly higher strength than the one predicted by HS model (Fig. **6**). From the Fig. (**7**), it is observed that with increase in the initial effective confining pressure, there is significant increase in the excess pore water pressure for HS case. In case of MC model, however, the rise in the pore pressure with rise in initial confining pressure is very small as seen in Fig. (**7**). To have further insight into the pore pressure evolution, the pore pressures are normalized with the initial effective confining pressure and are plotted in the Fig. (**8** and **10**). From Fig. (**8**), it is found that irrespective of the initial effective confining pressure, the normalized pore pressure is almost identical in HS case. The normalized pore pressures plotted in Fig. (**10**), however, are found to be distinct. It should be noted that in case of MC model, for initial effective confining pressure of 50 kPa, 100 kPa and 138 kPa, normalized pore pressure become identical at an axial strain beyond 17%.

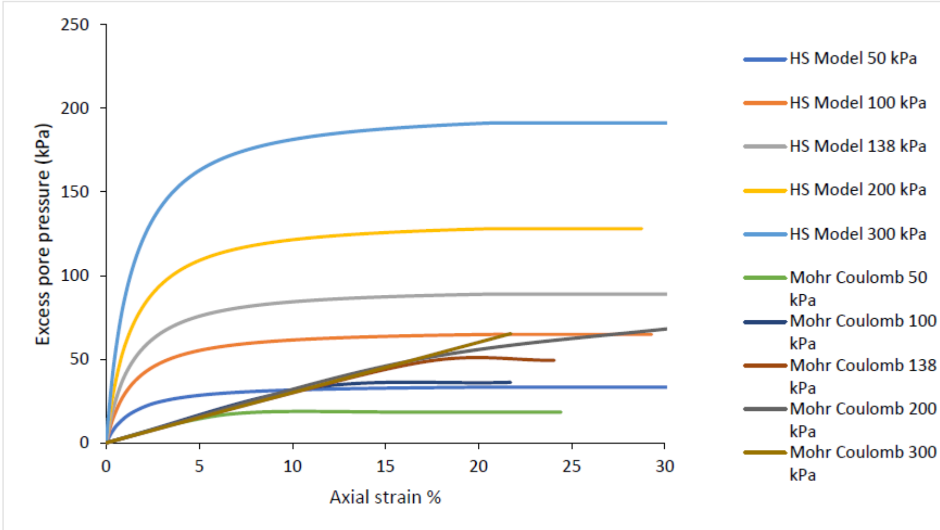


Figure 7: Excess pore pressure evolution at different confining pressure.

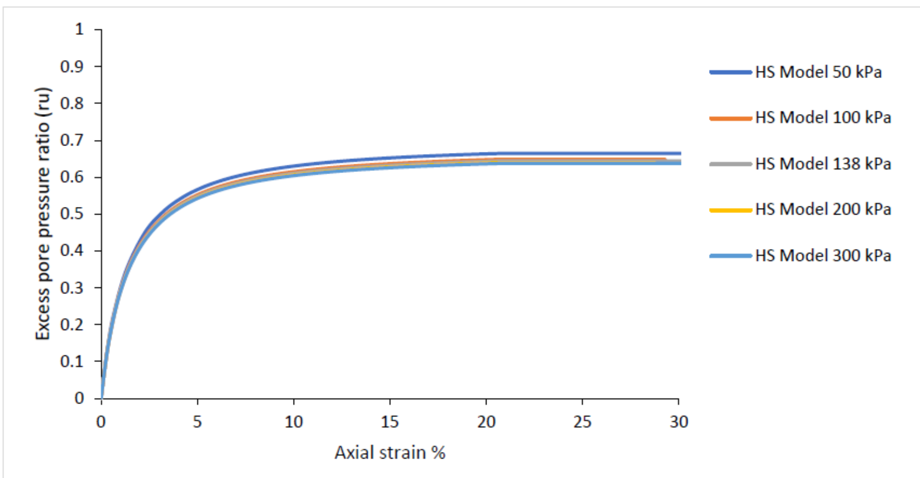


Figure 8: Normalized excess pore pressure at various confining pressure (HS model).

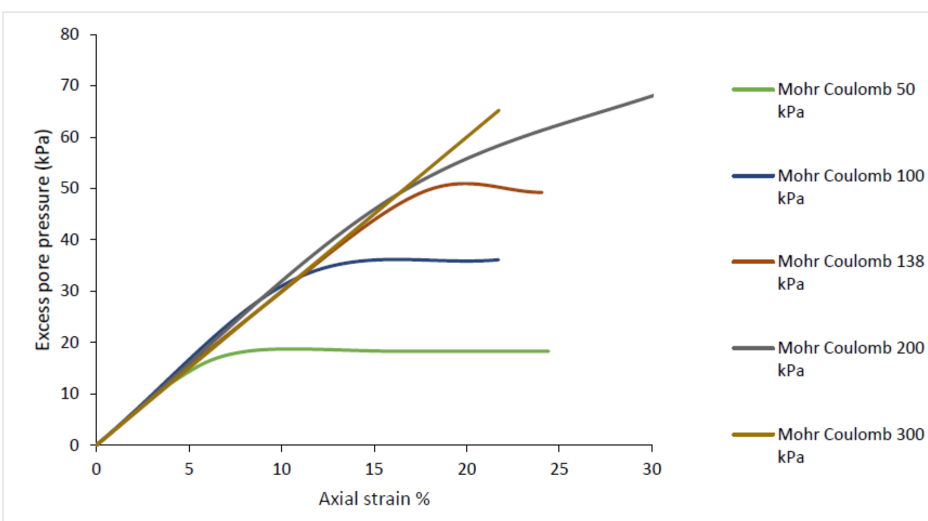


Figure 9: Excess pore pressure evolution at different confining pressure (MC model).

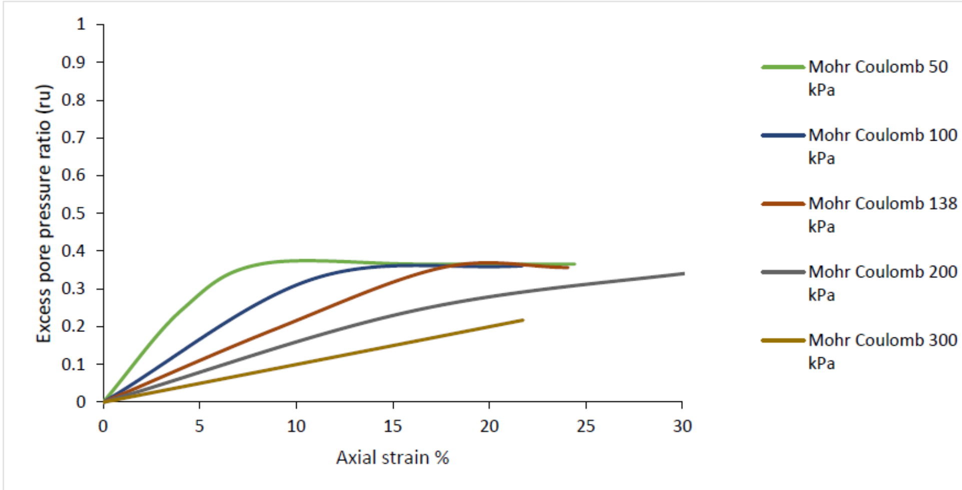


Figure 10: Normalized excess pore pressure at various confining pressures (Mohr Coulomb model).

4.2. Implications

For the same elastic moduli and shear strength parameters the Mohr Coulomb model always overestimates the shear strength of the soil. This may endanger the stability of the real life projects if the numerical analysis relies just on the Mohr Coulomb model. Therefore, in conjunction with Mohr Coulomb model other advanced material models should always be used in the stability analysis of the important (critical) real life projects. The excess pore pressure predicted by the MC model is always lower than its HS counterpart and the rate of evolution is also low. While dealing with the real life problems this may result into small and misleading pore pressure development. In all cases (i.e., confining pressures) the pore pressure predicted by MC model are significantly smaller than those from HS model. This implies that MC model fails to capture the contractive tendency of the soil, in undrained analysis, effectively.

5. Conclusions

The following major conclusions are drawn from the present study:

1. The Mohr Coulomb model though widely used and simple one, significantly overestimates the undrained shear strength. For initial effective confining pressure of 138 kPa, the undrained shear strength given by Mohr Coulomb model is 89% higher than one given by Hardening Soil model.
2. the Hardening Soil model effectively captures the nonlinear effective stress path in the undrained analysis. The effective stress path obtained from Mohr Coulomb model is vertical whereas that obtained from Hardening Soil model is curved and moves leftward. In case of Hardening Soil model, increase in the initial effective confining pressure increases the excess pore water pressure; however, the normalized excess pore pressure remains identical.
3. In case of Mohr Coulomb model, increase in the initial effective confining pressure increases the excess pore water pressure but this increment is significantly low when compared with one noticed for Hardening Soil case. Further, the normalized excess pore pressure is found to be distinct. It is worth to note that for the initial effective confining pressure of 50 kPa, 100 kPa and 138 kPa, the normalized pore pressure becomes identical beyond axial strain of 17%.

The present research investigated two constitutive models only. In near future, a rigorous study involving recently developed advanced constitutive models may be carried out. Further, this study has focused on monotonic loading only. The efficacy of MS model and HS model under cyclic/dynamic loading may also be investigated in near future.

5.1. Care that Needs to be Exercised

Even while using the HS model the constitutive parameters should be first calibrated carefully and then only it shall be used in the analysis of the real life problem. One of the major steps in the use of constitutive models is evaluation of the constitutive parameters as accurately as possible. Arbitrary use of the material parameters may result into numerically correct but physically unrealistic responses.

Conflicts of Interest

The author declares that there is no conflicts of interest.

Funding

This study did not receive any project-specific external funding.

Acknowledgments

The work was partly carried out during the Author's stint as a post-doctoral fellow with Prof. G L Sivakumar Babu at IISc Bangalore. The financial and technical assistance received during this tenure is gratefully acknowledged.

Data availability

All data supporting the findings of this manuscript are included within the manuscript itself.

References

[1] Bowels JE. Foundation analysis and design. New York: McGraw-Hill International Editions; 2001.

[2] Lambe TW, Whitman RV. Soil mechanics SI version. New Delhi: Wiley India Pvt. Ltd.; 2008.

[3] Hansen JB, Gibson RE. Undrained shear strengths of anisotropically consolidated clay. Geotechnique. 1949; 1: 189-200. https://doi.org/10.1680/geot.1949.1.3.189

[4] D'Ignazio M, Phoon KK, Tan SA, Länsivaara TT. Correlations for undrained shear strength of Finnish soft clays. Can Geotech J. 2016; 53: 1628-45. https://doi.org/10.1139/cgj-2016-0037

[5] Ladd CC. Stability evaluation during staged construction. J Geotech Eng. 1991; 117: 540-615. https://doi.org/10.1061/(ASCE)0733-9410(1991)117:4(540)

[6] Terzaghi K, Peck RB, Mesri G. Soil mechanics in engineering practice. New York: John Wiley & Sons Inc.; 1996.

[7] Augarde CE, Lee SJ, Loukidis D. Numerical modelling of large deformation problems in geotechnical engineering: a state-of-the-art review. Soils Found. 2021; 61: 1718-35. https://doi.org/10.1016/j.sandf.2021.08.007

[8] Barbosa RE, Ghaboussi J. Discrete finite element method for multiple deformable bodies. Finite Elem Anal Des. 1990; 7: 145-58. https://doi.org/10.1016/0168-874X(90)90006-Z

[9] Zhang W, Cassidy MJ, Tian Y. Practical large-deformation finite-element method for 3D geotechnical problems involving free surface deformations. Int J Geomech. 2020; 20: 1-13. https://doi.org/10.1061/(asce)gm.1943-5622.0001648

[10] Borja RI, Tamagnini C, Alarcón E. Elastoplastic consolidation at finite strain part 2: finite element implementation and numerical examples. Comput Methods Appl Mech Eng. 1998; 159: 103-22. https://doi.org/10.1016/S0045-7825(98)80105-9

[11] Carter JP, Booker JR, Small JC. Finite elasto-plastic consolidation. Int J Numer Anal Methods Geomech. 1979; 3: 107-29. https://doi.org/10.1002/nag.1610030202

[12] Lade PV. Overview of constitutive models for soils. In: Soil constitutive models: evaluation, selection, and calibration. Geo-Frontiers Congress 2005. Reston: ASCE; 2005. p. 1-34. https://doi.org/10.1061/40786(165)1

[13] Mohd Najmu Saquib WK, Showkat R. Soil constitutive models and their application in geotechnical engineering: a review. Int J Eng Res Technol. 2018; 7: 137-45. Available from: www.ijert.org

[14] Prevost JH, Popescu R, Obispo SL. Constitutive relations for soil materials. Electron J Geotech Eng. 1996.

[15] Hejazi Y, Dias D, Kastner R. Impact of constitutive models on the numerical analysis of underground constructions. Acta Geotech. 2008; 3: 251-8. https://doi.org/10.1007/s11440-008-0056-1

[16] Onyelowe KC, Ebid AM, Ramani Sujatha E, Fazel-Mojtahedi F, Golaghaei-Darzi A, Kontoni DPN, *et al*. Extensive overview of soil constitutive relations and applications for geotechnical engineering problems. Heliyon. 2023; 9: e14465. https://doi.org/10.1016/j.heliyon.2023.e14465

[17] Griffiths DV. Failure criteria interpretation based on Mohr-Coulomb friction. J Geotech Eng. 1990; 116: 986-99. https://doi.org/10.1061/(ASCE)0733-9410(1992)118:1(189)

[18] Ti KS, Huat BB, Noorzaei J, Jaafar S, Sew GC. A review of basic soil constitutive models for geotechnical application. Electron J Geotech Eng. 2009; 14: 1-18. Available from: https://www.researchgate.net/publication/228565882

[19] Robert DJ. A modified Mohr-Coulomb model to simulate the behavior of pipelines in unsaturated soils. Comput Geotech. 2017; 91: 146-60. https://doi.org/10.1016/j.compgeo.2017.07.004

[20] Kahlström M. Plaxis 2D comparison of Mohr-Coulomb and soft soil material models. Luleå: Luleå University of Technology; 2013. Available from: https://urn.kb.se/resolve?urn=urn:nbn:se:ltu:diva-49929

[21] Lee J, Jin H, An J, Baek Y, Yoon H. A study on the suitability of the Mohr-Coulomb model for numerical analysis of ground excavation. J Eng Geol. 2020; 30: 1-15.

[22] Teshager DK, Chwała M, Puła W. Probabilistic analysis of foundation settlement considering spatial variability in soil parameters: a comparison of hardening soil model and Mohr-Coulomb. Georisk Assess Manag Risk Eng Syst Geohazards. 2025; 2025: 1-21. https://doi.org/10.1080/17499518.2025.2515586

[23] Shi J, Yu Z, Jiang W, Zhan Z, Xia P. Sensitivity analysis and parameter selection of the modified Mohr–Coulomb model in foundation pit excavation in granite residual soil areas. Discov Appl Sci. 2024; 6: 653. https://doi.org/10.1007/s42452-024-06392-1

[24] Yin Z, Hicher P, Jin Y. Elastoplastic modeling of soils: from Mohr-Coulomb to SIMSAND. In: Practical constitutive modelling of saturated soils. Singapore: Springer; 2020. https://doi.org/10.1007/978-981-15-6307-2_5

[25] Puzrin AM. Constitutive modelling in geomechanics: introduction. Berlin: Springer; 2012. https://doi.org/10.1007/978-3-642-27395-7

[26] Atkinson JH. Non-linear soil stiffness in routine design. Geotechnique. 2000; 50: 487-508. https://doi.org/10.1680/geot.2000.50.5.487

[27] Mašín D. Modelling of soil behaviour with hypoplasticity. Cham: Springer; 2019. https://doi.org/10.1007/978-3-030-03976-9

[28] Puzrin AM, Bažant ZP, Bischof P. Non-linear model of small-strain behaviour of soils. Geotechnique. 1998; 48: 217-33. https://doi.org/10.1680/geot.1998.48.2.217

[29] Prashant A, Bhattacharya D, Gundlapalli S. Stress-state dependency of small-strain shear modulus in silty sand and sandy silt of Ganga. Geotechnique. 2019; 69: 42-56. https://doi.org/10.1680/jgeot.17.P.100

[30] Clayton CRI. Stiffness at small strain: research and practice. Geotechnique. 2011; 61: 5-37. https://doi.org/10.1680/geot.2011.61.1.5

[31] Cha M, Santamarina JC, Kim HS, Cho GC. Small-strain stiffness, shear-wave velocity, and soil compressibility. J Geotech Geoenviron Eng. 2014; 140(10): 1-4. https://doi.org/10.1061/(ASCE)GT.1943-5606.0001157

[32] Salgado R, Bandini P, Karim A. Shear strength and stiffness of silty sand. J Geotech Geoenviron Eng. 2000; 126: 451-62. https://doi.org/10.1061/(ASCE)1090-0241(2000)126:5(451)

[33] Mooney MA, Rinehart RV. In situ soil response to vibratory loading and its relationship to roller-measured soil stiffness. J Geotech Geoenviron Eng. 2009; 135: 1022-31. https://doi.org/10.1061/(asce)gt.1943-5606.0000046

[34] Gu X, Yang J, Huang M. DEM simulations of the small strain stiffness of granular soils: effect of stress ratio. Granul Matter. 2013; 15: 287-98. https://doi.org/10.1007/s10035-013-0407-y

[35] Surarak C, Likitlersuang S, Wanatowski D, Balasubramaniam A, Oh E, Guan H. Stiffness and strength parameters for hardening soil model of soft and stiff Bangkok clays. Soils Found. 2012; 52: 682-97. https://doi.org/10.1016/j.sandf.2012.07.009