Prediction of compaction curves

Nurses Kurucuk
Faculty of Engineering, Monash University, Clayton, VIC, Australia
Jayantha Kodikara
Faculty of Engineering, Monash University, Clayton, VIC, Australia
Delwyn Fredlund
Golder Associates, Saskatoon, Saskatchewan, Canada.

Keywords: soil compaction, soil suction, stiffness, unsaturated soil, pore pressure development

ABSTRACT

Compaction of soil is one of the major activities in geotechnical engineering involving earthworks. Compaction curve generally features an inverted parabolic shape and is used to find the optimum water content that maximises dry density. Since its introduction by Proctor in 1933, several researchers have provided qualitative explanations for the general shape of the compaction curve. Furthermore, there is a vast body of literature covering the behaviour of compacted soil. However, fundamental research on the compaction process and the evolution of compaction characteristics are limited. Therefore, in order to understand the driving mechanisms of compaction, this paper investigates the effect of soil suction and stiffness in the shape of the compaction curve, from unsaturated soil mechanics standpoint. On the basis of unsaturated soil mechanics principles, a theoretical prediction of the parabolic shape of the compaction curve is provided.

1 INTRODUCTION

Soil compaction is widely used in geo-engineering and is important for the construction of roads, dams, landfills, airfields, foundations, hydraulic barriers, and ground improvements. Compaction is applied to the soil, with the purpose of finding optimum water content to maximise its dry density, and therefore, to decrease soil's compressibility, increase its shearing strength, and in some cases, to reduce its permeability. Proper compaction of materials ensures the durability and stability of earthen constructions.

A typical compaction curve presents different densification stages when the soil is compacted with the same apparent energy input but different water contents. The water content at the peak of the curve is called optimum water content (OWC) and represents the water content in which dry density is maximized for a given compaction energy.

Since Proctor`s pioneering work in 1933, many researchers have attempted to explain the leading mechanisms in the densification stages, mainly on the dry side of optimum water content. The compaction curve was explained in terms of capillarity and lubrication (Proctor, 1933), viscous water (Hogentogler, 1936), pore pressure theory in unsaturated soils (Hilf, 1956), physico-chemical interactions (Lambe, 1960), and concepts of effective stress theory (Olson, 1963). Moreover, Barden and Sides (1970) undertook an experimental research on the relation between the engineering performance of compacted unsaturated clay and microscopic observations of clay structure. In addition, Lee and Suedkamp (1972) conducted research on the shape of the compaction curve for different soils. These works provide predominantly qualitative explanations of the shape of the curve.

Despite this research work, and the importance and high demand of the compaction process in engineering practise, it still remains that the compaction of soil is quite complex and not well explained, particularly from a quantitative sense. Therefore, there is need for research to be undertaken at fundamental level to understand the evolution of compaction characteristics of soil and the inverted parabolic shape of the compaction curve.

This paper presents a theoretical prediction of the compaction curve for sand using unsaturated soil mechanics principles. In order to achieve this aim, approaches developed by Hilf (1948) and Fredlund & Rahardjo (1993) for undrained loading process are simulated. Volume changes of a compacted soil are calculated using the volume change theory from unsaturated soil mechanics, and are used o predict theoretical compaction curves.

2 MODELLING OF COMPACTION CURVES

2.1 Theoretical background

Theoretical concepts utilized for the development of soil compaction curves are presented in this section. Initially, Hilf's (1948) and Fredlund & Rahardjo's (1993) approaches for pore pressure development are presented. This is continued with Fredlund & Morgenstern's (1976) volume change theory for a compacted soil and the derivation of the dry density of a soil.

2.1.1 Pore pressure development during static compaction

One of the main simulations for the generation of the compaction curve is that of pore pressure development. Hilf (1948) developed a relationship between pore pressure and applied stress, which is based on one-dimensional soil compression, Boyle's law, and Henry's law, and is expressed as follows:

$$\Delta\bar{u}_a = \left[\frac{1}{1+\dfrac{(1-S_0+hS_0)n_0}{(\bar{u}_{a0}+\Delta\bar{u}_a)m_v}}\right]\Delta\sigma_y \qquad (1)$$

where; $\Delta\bar{u}_a$ = change in absolute pore air pressure, S_0 = initial degree of saturation, h = coefficient of solubility, n_0 = initial porosity, $\bar{u}_{a0}$ = initial absolute air pressure, m_v = coefficient of volume change in saturated soil, and $\Delta\sigma_y$ = change in applied vertical stress.

Hilf (1948) developed this equation assuming that air and water phases are undrained, and volume reduction is due to air dissolving in the water and compression of free air. Both liquid and solid parts were considered to be incompressible. He also assumed that the change in air pressure is equal to the change in pore water pressure, and therefore, matric suction change was insignificant. These assumptions will be reviewed further in Section 2.2. On the other hand, Fredlund & Rahardjo (1993) developed an equation for undrained loading for pore pressure changes by using pore pressure parameters:

$$\Delta u_a = B_a\Delta\sigma_y \qquad (2) \qquad\qquad \Delta u_w = B_w\Delta\sigma_y \qquad (3)$$

where; B_a = pore air pressure parameter, B_w = pore water pressure parameter, and $\Delta\sigma_y$ = change in applied vertical stress.

This approach assumes that soil particles are incompressible, but the fluid phases are compressible. In contrast to the approach by Hilf (1948), matric suction changes were taken into account. It was accepted that air and water pressures become equal at saturation. Therefore, matric suction will decrease from the initial value to zero at saturation. Pore air (B_a) and pore water (B_w) pressure parameters are defined as follows:

$$B_a = \frac{R_2R_3-R_4}{1-R_1R_3} \qquad (4); \qquad\qquad B_w = \frac{R_2-R_1R_4}{1-R_1R_3}. \qquad (5)$$

In Equations 4 and 5, R_1, R_2, R_3 and R_4 are defined as:

$$R_1 = \frac{m_2^s/m_1^s-1-(1-S+hS)n/(\bar{u}_a m_1^s)}{m_2^s/m_1^s+SnC_w/m_1^s} \qquad (6); \qquad R_2 = \frac{1}{m_2^s/m_1^s+SnC_w/m_1^s} \qquad (7)$$

$$R_3 = \frac{m_2^a/m_1^a}{m_2^a/m_1^a-1-(1-S+hS)n/(\bar{u}_a m_1^s)} \qquad (8); \qquad R_4 = \frac{1}{m_2^a/m_1^a-1-(1-S+hS)n/(\bar{u}_a m_1^s)} \qquad (9)$$

where; m_1^s = compressibility of soil particles with respect to net stress $(\sigma_y - u_a)$, m_2^s = compressibility of soil particles referenced to suction $(u_a - u_w)$, m_1^a = compressibility of air phase with respect to net stress $(\sigma_y - u_a)$, m_2^a = compressibility of air phase with respect to suction $(u_a - u_w)$, $\bar{u}_a$ = absolute pore air pressure S = degree of saturation, h = coefficient of solubility, n = porosity, and C_w = compressibility of water.

2.1.2 Computation of volume change and dry density

The volume change constitutive relationship proposed by Fredlund & Morgenstern (1976) for unsaturated soils is used for the calculations of compaction curves, and is expressed as follows:

$$\frac{\Delta V_v}{V_0} = m_1^s \Delta(\sigma_y - u_a) + m_2^s \Delta(u_a - u_w) \quad (10)$$

where; ΔV_v = overall volume change of a soil element, V_0 = initial total volume of the soil element, m_1^s = compressibility of soil particles with respect to net stress $(\sigma_y - u_a)$, m_2^s = compressibility of soil particles referenced to suction $(u_a - u_w)$, $\Delta(\sigma_y - u_a)$ = change in net stress, and $\Delta(u_a - u_w)$ = change in soil suction.

Since soil particles are incompressible, it is accepted that deformation is primarily due to compression of the pore fluid (i.e., the air and water mixture). The independent stress state variable concept is utilized in the derivation; namely, net stress $(\sigma_y - u_a)$ (causes a reduction in volume with compression), and matric suction stress $(u_a - u_w)$ (generally results in volume increase with compression). Consequently, overall volume change of the soil sample is calculated by multiplying right hand side of Equation 10 by the initial volume of the soil sample. Furthermore, the final total volume can be obtained using the volume change of the soil, and this gives the corresponding dry density.

2.2 Estimation of soil compaction curves

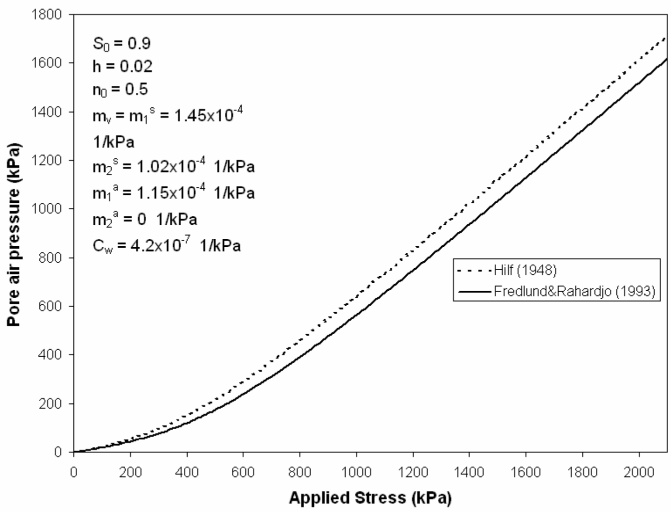

Figure 1: Pore air pressure development with applied stress

In order to produce compaction curve for a given set of parameters, the pore air pressure development for Hilf`s (1948) and Fredlund & Rahardjo's (1993) approaches are plotted in Figure 1. The parametric values used in this calculation are given in this figure. It should be noted that Fredlund & Rahardjo`s (1993) approach takes into consideration the changes in suction with compression, whereas Hilf's (1948) analysis assumes that the suction change is negligible during compression and suction is equal to the initial matric suction. Therefore, Hilf`s (1948) approach produces larger pore air pressure values than Fredlund & Rahardjo's (1993) approach for a given applied stress.

Using the volume change equation (Equation 10) and air pressure computed with applied stress, (Figure 1), compaction curves are produced for both approaches as shown in Figure 2. In Figure 2 compressibility coefficients are assumed to be constant and their values are given on the figure. It should be noted that compaction curve which is plotted using Hilf's (1948) approach does not include the suction component of volume change theory (or assumed m_2^s is assumed equal to zero in Equation 10).

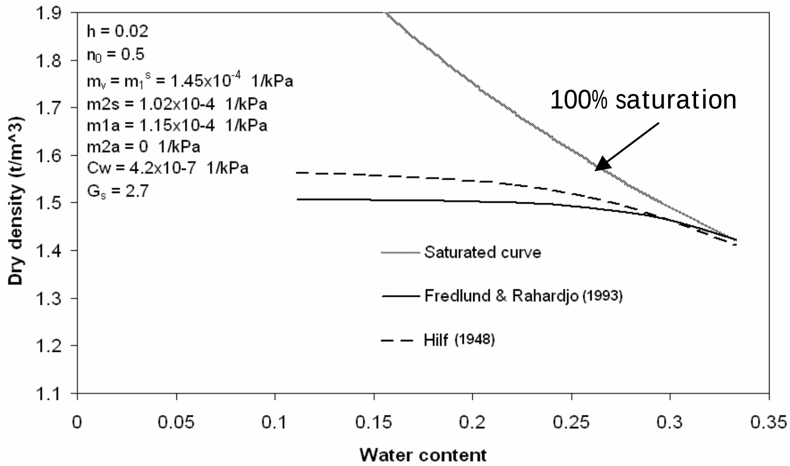

Figure 2: Compaction curves for constant coefficient of compressibility

It is clear that this method (using constant compressibility coefficients (m_v, m_1^s, m_2^s, in fact it can be shown that m_v=m_1^s) can produce a reasonable shape for the compaction curve on the wet side of OWC, but not on the dry side of OWC. Moreover, suction change that is included in Fredlund & Rahardjo's (1993) approach results in some reduction in dry density, but does not change the shape of the compaction curve at lower water contents. In order to examine the likely suction change during compaction, consideration is given to the experimental results produced by Montanez (2002), as shown in Figure 3. Montanez (2002) produced suction contours for compacted sand-bentonite mixtures associated with domains defined by compaction curves at various apparent energy levels. He found that matric suction only decreases marginally with a density increase and in many cases can be considered constant. Therefore, Hilf's analysis of assuming constant suction during compaction appears to be close to the real situation, but direct data measured during compaction process may be useful to validate this assumption.

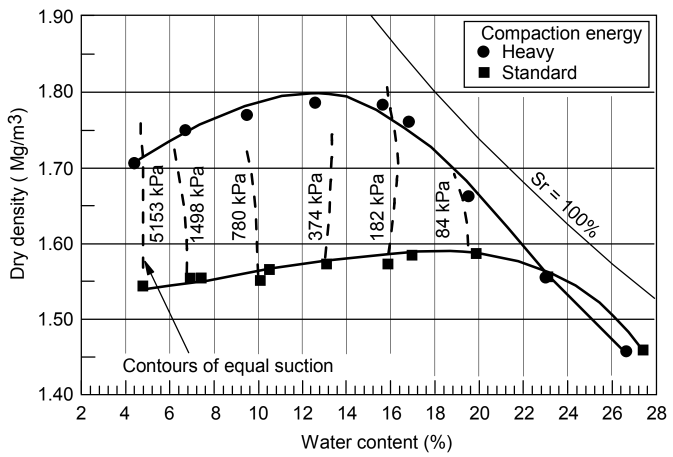

Figure 3: Data for uniform sand-bentonite (10%) mixture (adopted from Montanez, 2002)

Furthermore, even if there is a small suction decrease with compaction, m_2^s applicable to the unloading process can be considered smaller than that applicable to loading process. Therefore, the effect of the suction component (as in the second component of Equation 10) can be considered not very significant. With this insight, the focus on the compaction process falls on the coefficient of soil compressibility due to net stress, m_{1s}. In the following part of the paper, the influence of coefficient of compressibility due to net stress (m_{1s}) on the shape of the compaction curve is presented. Direct data related to the variation of m_1^s with saturation are not available. However, some insight can be gained from the shear-wave velocity measured by Cho and Santamarina (2001) in granite powder at various saturations, shown in Figure 4, to explain the relationship between m_1^s and degree of saturation.

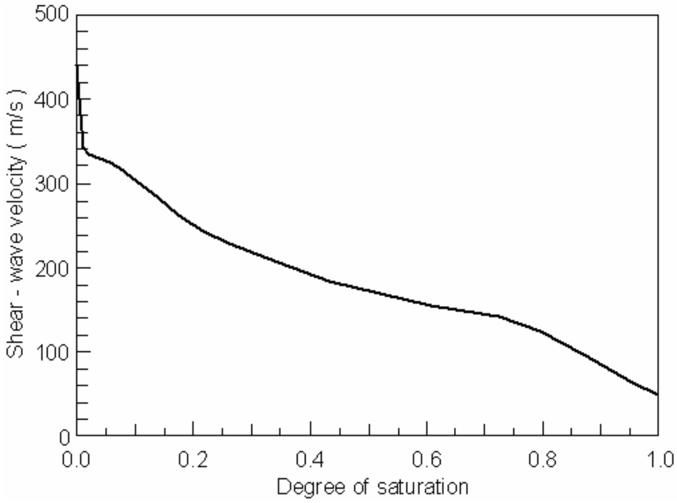

Figure 4: Shear-wave velocity data for granite powder (adopted from Cho & Santamarina, 2001)

It is clear from Figure 4 that the shear-wave velocity (v_s), hence shear modulus ($v_s=\sqrt{G/\rho}$) of the soil increases with decreasing saturation (or conversely soil compressibility decreases). Therefore, it can be argued that m_1^s will decrease with decreasing saturation in some form. On this basis, a relationship between m_1^s and degree of saturation is plotted as shown in Figure 5.

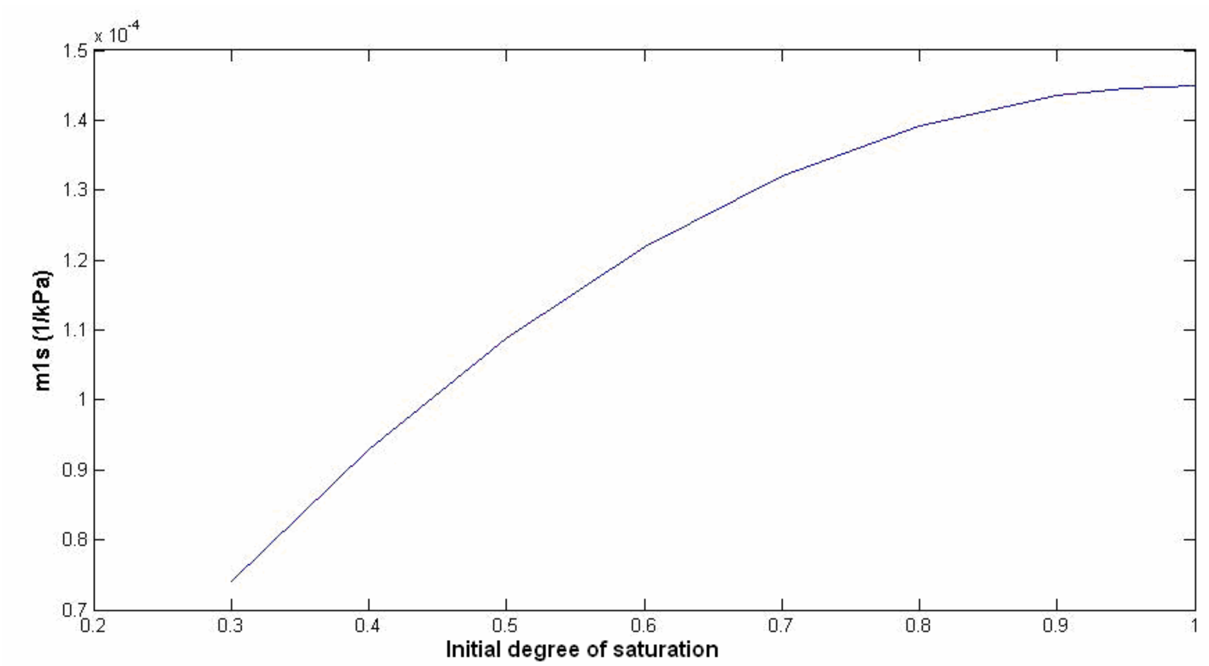

Figure 5: Change in m_1^s values with initial degree of saturation

When the variation of m_1^s with degree of saturation is taken into account, the resulting compaction curves are shown in Figure 6. Clearly, now the proper shape of the compaction curve can be produced. Moreover, it is clear that the dependency of compaction curves at different stress levels (or energy input) is reasonably well captured, where optimum water content shifts to the right with higher stress levels.

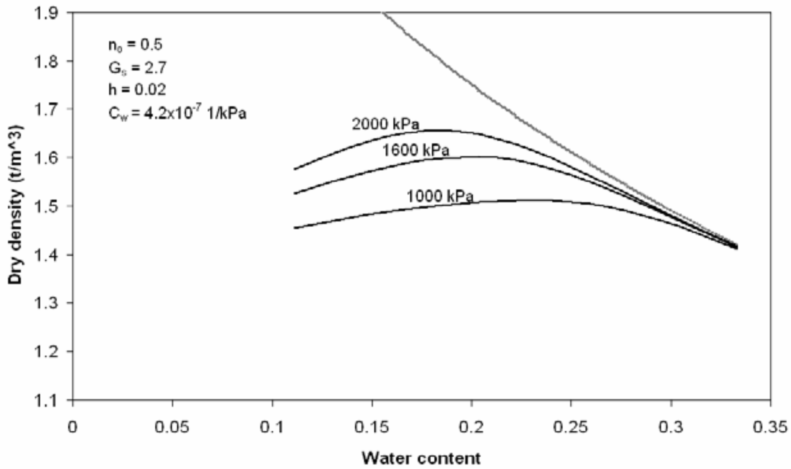

Figure 6: Compaction curves for different energy levels

3 CONCLUSIONS

This paper presents the theoretical concepts related to the prediction of the compaction curve. For the first time, it highlights the fact that shape of the compaction curve can be predicted using unsaturated soil mechanics principles. The main insight gained was that the changes in matric suction is not important for the evolution of the compaction states, but the influence of matric suction on the material compressibility with respect to net stress is the governing factor determining the compaction density. Therefore, it can be reasoned that the inverted parabolic shape of the compaction curves is a direct function of the variation of the material compressibility with degree of saturation. More research, however, is necessary to reinforce these concepts.

REFERENCES

Barden, L., and Sides, G. R. (1970), Engineering behaviour and structure of compacted clay, *Journal Soil Mechanics and Foundations Division, ASCE*, *96*, No. SM4, 1171.

Cho, G. C., and Santamarina, J. C. (2001), Unsaturated particulated materials – Particle level studies, *Journal of Geotechnical and Geoenvironmental Engineering,* 127(1), 84-96.

Fredlund, D. G., and Morgenstern, N. R. (1976), Constitutive relations for volume change in unsaturated soils, *Canadian Geotechnical Journal,* 14, 3, 261-276.

Fredlund, D. G., and Rahardjo, H. (1993), Soil mechanics for unsaturated soils, p. 191-194, John Wiley & Sons.

Hilf, J. W. (1956), *An Investigation of Pore Water Pressures in Compacted Cohesive Soils*, Technical Memorandum 654, U. S. Department of the Interior, Bureau of Reclamation, Denver, Colorado.

Hogentogler, C. A. (1936), Essentials of soil compaction, *Proceedings Highway Research Board, National Research Council, Washington, D.C.*, 309-316.

Lambe, T. W. (1960), Structure of compacted clay, *Transactions, ASCE*, *125*, 682-705.

Lee, D. Y., and Suedkamp, R. J. (1972), Characteristics of irregularly shaped compaction curves of soil, *Highway Research Board*, *381*, 1-9.

Montanez, J. E. C. (2002), Suction and volume changes of compacted sand-bentonite mixtures, PhD thesis, University of London, Imperial College of Science, London, England.

Olson, R. E. (1963), Effective stress theory of soil compaction, *Journal Soil Mechanics and Foundations Division, ASCE*, *89*, No. SM2, 27-45.

Proctor, R. R. (1933), Fundamental Principles of Soil Compaction, *Engineering News-Record*, *111*, 286.