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Forest soils with low bulk densities are often considered less susceptible to compaction than soils with higher bulk densities. The objective of this study was to determine if soil strength controlled the compression of soils with low bulk density. Four soils were selected for this evaluation. Three of these were andic soils with low bulk density and the fourth soil was a more dense, cohesive soil. Undisturbed samples of saturated and partly saturated soil were compressed in a one-dimensional consolidation test apparatus. Measurements with separate samples were at one of 7 normal stresses between 0.033 and 1.96 MPa. Shear strength of saturated soil was measured in direct shear tests. Primary consolidation of saturated soil was completed in less than one minute at all normal stresses. Shear stress and bulk density increased continuously during shear strain. The compression index of the cohesive soil was significantly larger ($p<0.05$) than that of theandic soils. The shear strength of andic soils (average cohesion intercept of 0.016 MPa and friction angle of 33.3°) was significantly higher ($p<0.05$) than the cohesive soil (cohesion intercept of 0.028 MPa and friction angle of 28.9°). When saturated, the cohesive soil was more compressible than the andic soils because of lower soil strength. A nonlinear model of soil compression was developed that accurately predicted the compressed density of saturated and partly saturated soil as a function of normal stress, initial bulk density of undisturbed samples, and degree of saturation. As degree of saturation decreased, the compressibility of the cohesive soil decreased more rapidly than it did for the andic soils. As a result, bulk density of dry cohesive soil increased less than it did for dry andic soils. Differences in the compressibility of soils were attributed to texture and clay mineralogy. The differences in the compressibility of these soils were much smaller than were the differences in bulk density. Decreasing water content affected the compressibility of the cohesive soil more than it affected the andic soils. Because soil strength controls the compressibility of these forest soils regardless of bulk density, it will also determine the susceptibility of soils to compaction by machines.

Consolidation, Compression, and Shear Strength of Four Western Oregon Forest Soils

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TABLE OF CONTENTS

Chapter I.	INTRODUCTION	1
	Objective	8
	Organization of Chapters	9
	Chapter II	9
	Chapter III	9
	Chapter IV	10
Chapter II.	CONSOLIDATION AND SHEAR STRENGTH OF SATURATED SOIL	11
	Introduction	11
	Measurement of Soil Deformation and Shear Strength	15
	Consolidation and compression	15
	Shear strength	17
	Conclusion	20
	Materials and Methods	21
	Soil materials	21
	Collection of undisturbed soil cores	22
	Consolidation/direct shear machine	25
	Consolidation test	27
	Direct shear test	28
	Soil characterization	29
	Statistical analyses	29
	Results	31
	Soil properties	31
	Consolidation of soils	31
	Compression curves of individual samples	34
	Composite compression curves	39
	Shear strength	45
	Discussion	50
	Conclusions	56
Chapter III.	COMPRESSION OF SATURATED AND PARTLY SATURATED SOIL	58
	Introduction	58
	Materials and methods	65
	Site selection	65
	Soil collection	65
	Compression test	68

Statistical Analyses 69
Results 70
Soil properties 70
Effects of water content on soil compression 74
Development of a nonlinear model of soil compression 85
Discussion 103
Conclusions 111

CHAPTER IV SUMMARY 113

BIBLIOGRAPHY 120

APPENDICES

Definitions 128
List of Symbols 133

LIST OF FIGURES

FIGURE		PAGE
II.1.	Diagrams of cutter head and sample ring, split ring sample holder, and shear box assembly.	24
II.2.	One position in the six position consolidation frame and direct shear test machine.	26
II.3.	Increases in bulk density as a function of time at six normal stresses for an undisturbed, saturated sample of Jory soil.	33
II.4.	Consolidation and rebound curves of three saturated, undisturbed samples of Jory soil.	35
II.5.	Typical consolidation and rebound curve for each soil.	36
II.6.	Bulk density as a function of normal stress of individual samples of each soil.	41
II.7.	Bulk density predicted with Equation 5 as a function of normal stress.	44

List of Figures, Continued.

II.8.	Example of changes in bulk density (A), and shear stress (B) as a function of horizontal strain for each soil.	46
II.9.	Relationship between shear strength and normal stress of individual samples, and the shear strength line (Table II.4) for each soil.	47
III.1.	Relationship between bulk density of a sample of western Oregon forest soil, predicted with Equation 1, and normal stress (normal and logarithmic scale).	62
III.2.	Water release curves from tests using undisturbed core samples from the 9-11 cm layer as a function of soil water content (A) and degree of saturation (B).	72
III.3A.	Bulk density of undisturbed core samples (7- to 12 cm depth) of the Crater Lake soil from the southern Oregon Cascade Mountains compressed in one-dimensional consolidation tests.	75

List of Figures, Continued.

III.3B.	Bulk density of undisturbed core samples (7- to 12 cm depth) of the Hemcross soil from the northern Oregon Coast Range Mountains compressed in one-dimensional consolidation tests (n=109).	76
III.3C.	Bulk density of undisturbed core samples (7- to 12 cm depth) of the Tolovanna soil from the central Oregon Coast compressed in one-dimensional consolidation tests (n=79).	77
III.3D.	Bulk density of undisturbed core samples (7- to 12 cm depth) of the Jory soil from the southern Willamette Valley of western Oregon compressed in one-dimensional consolidation tests (n=140).	78
III.4.	Effects of degree of saturation on the relationship between bulk density and normal stress of the Crater Lake and Tolovanna soils.	96

List of Figures, Continued.

III.5.	Effects of degree of saturation on the relationship between the compressed bulk density and normal stress of four western Oregon forest soils.	97
III.6	Effects of parameters on the relationship between bulk density of saturated soil and normal stress.	102

LIST OF TABLES

TABLE		PAGE
II.1.	Physical properties and organic matter content of four western Oregon forest soils.	32
II.2.	Parameters for nonlinear model (Eqn 2) of all normal stresses, and the compression index, C_c, and rebound index, C_r, of soil at normal stresses greater than 0.1 MPa.	37
II.3.	Parameters for nonlinear models (Eqns 2 and 5) of the consolidation of individual samples.	40
II.4.	Cohesion intercepts, c, and angles of friction, ϕ, of undisturbed, saturated samples of four western Oregon forest soils.	48
III.1.	Classification and physical properties of four western Oregon forest soils.	66
III.2.	Average soil water content and initial bulk density, and number of samples collected at water contents ranging from field capacity to field dry.	71

List of Tables, Continued.

III.3.	Parameters and mean square errors (MSE) for the nonlinear regression model (Eqn 2) of compression of soil collected and tested at different soil water contents.	80
III.4.	Multiple linear regression parameters and regression coefficients for Equation 3.	82
III.5	Regression coefficients between parameters estimated for Equation 2 (Table III.3) and soil water content of each soil.	84
III.6	Parameters and mean square errors (MSE) of the nonlinear regression model of compression using Equation 2.	86
III.7.	Parameters and mean square errors (MSE) For Equations 7, 9, and 10.	89
III.8.	Parameters and mean square errors (MSE) for Equation 11 when values of 0.5, 1.5, and 2 were assigned to the H parameter.	92

List of Tables, Continued.

III.9.	Parameters and mean square errors (MSE) for Equation 12 when H was estimated by regression, and when values of 0.5, 1.5, and 2 were assigned to the H parameter.	93
III.10.	Parameters and mean square errors (MSE) for Equation 13.	94
III.11.	Range in measured bulk densities of undisturbed and compressed soils, and the regression coefficients of bulk density predicted with Equation 13 and the bulk density (independent variable) measured on soils compressed at normal stresses between 0.033 and 1.96 MPa.	99
III.12.	Nonlinear regression correlation matrix of soil compression parameters estimated with Equation 13.	100

CONSOLIDATION, COMPRESSION, AND SHEAR STRENGTH OF FOUR WESTERN OREGON FOREST SOILS

Chapter I

INTRODUCTION

The evergreen coniferous forests of the Pacific Northwest are unique among temperate forest ecosystems because of the massive size and longevity of the trees, and the large accumulations of biomass in stands (Waring and Franklin, 1979). Seldom mentioned attributes of these ecosystems are the unique properties of the soil supporting these forests. The most notable properties are a low bulk density and a high macroporosity, soil water content, and organic matter content (Dyrness, 1969; Brown, 1975; McNabb et al., 1986). Bulk densities less than 1.0 Mg/m^3 to a depth of 1 m have been reported for many forest soils in western Oregon (Froehlich and McNabb, 1984). Soils with these unique physical properties are collectively referred to as andic soils (Wada and Harward, 1974; Maeda et al., 1977). Andic soils weather from volcanic ejecta and volcaniclastic sediments (Baldwin, 1964). Fine-textured andic soils are common in mesic forest environments. The largest and most productive forest stands in the Coast Range and Cascade Mountains of western Oregon and

Washington occur on these soils (Steinbrenner, 1979).

The clay minerals responsible for andic properties of fine-textured soils are of noncrystalline and short-range crystalline order rather than of long-range order that is typical of crystalline, layer silicate minerals in other parent materials (Maeda et al., 1977; Kinloch, 1987). Noncrystalline minerals have also been referred to as amorphous, or allophane, minerals but noncrystalline is a more generic term which includes a continuum of noncrystalline and short-range order, crystalline minerals (Wada and Harward, 1974; Maeda et al., 1977; Jackson et al., 1986). Soils containing noncrystalline minerals have also been called allophane soils but will be referred to as andic soils because of the plans to separate these soils into the new soil order of Andisols (Kinloch, 1987).

In addition to a low bulk density, andic soils typically have a high soil water content at saturation, a high soil water content at -1.5 MPa, a high natural water content, a high liquid limit and low plastic index, and irreversible changes in these properties on drying (Maeda et al., 1977). Soil water content is high because of the low bulk density. Volumetric water content of andic soils is often similar to soils containing layer silicate minerals. An important feature distinguishing many andic soils from soils with similar texture, containing crystalline layer silicate minerals, is that void characteristics dominate their behavior rather than physical-chemical properties. The unique physical properties that noncrystalline minerals impart to andic soils are the basis for considering andic soils as a separate group from cohesionless and cohesive soils (Maeda et al., 1977), and for separating andic soils into the proposed new soil order of Andisols (Kinloch, 1987).

More than 70 Oregon soil series have medial, ashy, or cindery family classifications indicative of andic soils; over two-thirds of these are in the medial classification occurring west of the Cascade Mountains (Huddleston, 1979). The andic properties of these soils are often not as distinctive as those in other Pacific Rim countries. The andic soils in Oregon often contain noncrystalline minerals, layer silicate minerals, and integrates of the two minerals (Paeth et al., 1971; Taskey et al., 1979). A survey of a few western Oregon soils containing noncrystalline clays showed that the liquid limits were lower, the plasticity index higher, and that less irreversible change occurred in these properties than with andic soils in other countries (McNabb, 1979).

The unusual physical properties of andic soils have raised many questions about forest management practices that have the potential to depreciate soil quality irreversibly. Practices should be devised which insure the long-term productivity of these soils. Considerable information exists on the management of non-andic soils but comparatively little information exists regarding management of andic soils in forest ecosystems. Their unusual physical properties hinder the extrapolation of information from non-andic soils. Of greatest concern is the uncertainty of how deformation affects these soils.

Andic soils are generally compacted by tracked machines and rubber-tired skidders (Froehlich et al., 1980; Froehlich et al., 1985; Allbrook, 1986; Geist et al., 1989). But compacted bulk densities of andic soils seldom exceed the natural bulk densities of non-andic soils and are always less than the compacted bulk density of these soils. For this reason, andic soils are often considered less susceptible to compaction than non-andic soils (Howard et al., 1981).

Bulk density is the most common measure of soil degradation by compaction and is generally associated with reduced tree growth on compacted soil (Greacen and Sands, 1980). In the Pacific Northwest, compaction often reduces the height growth of young conifers in proportion to the relative increase in bulk density (Froehlich and McNabb, 1984). The compacted bulk densities of andic soils and other forest soils with low bulk density which reduce tree growth, however, are much lower than the bulk densities considered necessary to limit root growth. Limiting bulk densities are generally between 1.5 and 1.9 Mg/m^3 (Minore et al., 1969; Heilman, 1981; Daddow and Warrington, 1983; Jones, 1983). Growth of a 31-year-old Pseudotsuga menziesii (Mirb.) Franco stand in western Oregon was reduced 11.8 percent as a result of compaction which occurred during harvesting of the original forest with crawler tractors (Wert and Thomas, 1981). The average bulk density of soil in skid roads was 1.22 Mg/m^3 after 31 years. Compacted forest soils are slow to recover unless tilled (Froehlich and McNabb, 1984). Compacted soils are estimated to take 4- to 7-decades to return to their original bulk densities and andic soils may take longer to recover than other soil materials (Froehlich et al., 1985).

Few data exist on the deformation of andic soils from an applied stress. Maeda et al. (1977) reviewed the literature on the physical properties of andic soils, including their engineering properties. The data base was dominated by studies of Japanese soils which had natural soil water contents higher than those measured in the Pacific Northwest. Andic soils have a poorly defined optimum water content for compaction because the broad peak of the compaction curve makes identifying a maximum bulk density difficult (Maeda et al., 1977; Froehlich et al., 1980; Howard et al., 1981). The bulk density of andic soils achieved in impact compaction tests at similar energies are lower than the compacted bulk densities of non-andic soils (Howard et al., 1981). Furthermore, these bulk densities are generally not attained when soils are compacted by harvesting machines (Froehlich et al., 1980).

Larson et al. (1980) conducted one-dimensional consolidation tests on two disturbed, unsaturated andic soils from Hawaii. The compression indexes were smaller than about 80 percent of the other soil materials tested. The soils with the smallest compression indexes were generally sands and sandy loams which are less easily compacted in this type of test (Lambe and Whitman, 1979). These data suggest that andic soils are less compressible than fine-textured cohesive soils. Impact compaction tests of two andic soils from California, however, resulted in increases in bulk density that were similar to the increase in more dense soils of non-andic parent materials (Howard et al, 1981).

The failure of andic soils to increase in bulk density more than soils of higher bulk density suggests that soil strength of andic soils is high despite the low bulk density. Allbrook (1986) reported that an increase in vane shear strength and cone index strength of a volcanic ash soil in central Oregon was significantly related to an increase in bulk density. Froehlich and McNabb (1984) also reported that the increase in vane shear strength of a volcanic ash soil from northeastern Oregon was similar to the increase in soils with nearly twice the bulk density of the andic soil. Coarse-textured volcanic ash soils would be expected to have high vane shear strength because of the method of measurement (Wroth, 1984) and the bridging of pumice particles (Cochran, 1971). The shear strength of fine-textured andic soils is typically low. Soils have low cohesion intercepts and angles of friction (Maeda et al., 1977). The accurate measurement of the shear strength of many andic soils is hindered by a large variability in engineering properties of these soils over short distances (Pope and Anderson, 1961).

In conclusion, andic soils and soils with andic properties occur in many forests of the Pacific Northwest. The noncrystalline minerals of andic soils impart unique physical properties to andic soils that contribute to their productivity. Information on the physical properties of andic soils in western Oregon is limited to the measurement of bulk density and water retention. Andic soils are often assumed to be less susceptible to compaction because of their low natural and compacted bulk densities. The growth of trees on compacted soil, however, is often reduced for decades. Measurement of deformation and soil strength of andic soils is needed to understand and predict their behavior from mechanical stresses.

OBJECTIVE

The objective of this study was to determine if the compressibility of soils with low bulk density was controlled by soil strength. Consolidation and shear strength were measured on undisturbed core samples of four western Oregon forest soils. The compressibility of partly saturated, undisturbed core samples collected at several in situ water contents was also measured in a one dimensional consolidation test.

Three andic soils with low bulk density were contrasted with a fine-textured, cohesive soil which contained layer silicate minerals. The four soils were chosen to represent specific types of engineering materials. Sites for obtaining soil samples were chosen on the basis of a criteria of soil and site properties. Only one site was sampled for each soil.

ORGANIZATION OF CHAPTERS

Chapter II

Chapter II includes the criteria and procedures for selecting sites and soils, and a description of the consolidation and direct shear tests. All soils were tested when saturated. Three replications of each soil were consolidated at seven normal stresses between 0.033 and 1.96 MPa and the bulk density measured after each stress was applied. Three samples were also consolidated at each of six normal stresses between 0.033 and 0.98 MPa. Results include: the compression and rebound index of the individual soils; a nonlinear model of the compression curve of each soil; modification of a nonlinear compression model to describe the relationship between bulk density and normal stress from measurements of individual samples; and the shear strength of saturated soil.

Chapter III

Chapter III describes the collection and compression of partly saturated soil. Data for consolidation of saturated soil reported in Chapter II were included as an additional soil water content. The nonlinear model of soil compression developed in Chapter II was used to predict the compression of each soil at each collected water content. This model occasionally failed at some water contents and the parameters that were obtained at other water contents were not related to soil water content. A nonlinear compression model was developed that included variables for soil water content expressed as applied normal stress, degree of saturation, and variation in initial bulk density. This model accurately predicted the bulk density of each soil for a range of normal stresses and degrees of saturation. Finally, the effects of changing soil water content on soil compression and compressive strength are discussed.

Chapter IV

Chapter VI is a summary of the relationship between soil compression and strength, and the general model of soil compression.

Chapter II

CONSOLIDATION AND SHEAR STRENGTH OF SATURATED SOILS

INTRODUCTION

Uncertain knowledge of soil deformation in response to mechanical stress hinders development of techniques which minimize the adverse effects of soil compaction on soil productivity. Deformation results from using crawler tractors and rubber-tired skidders to harvest forests and prepare sites for reforestation. Increases in bulk density from compaction are commonly associated with reduced tree growth (Greacen and Sands, 1980). Forest soils in the Pacific Northwest, however, have often been considered less susceptible to compaction because the undisturbed and the compacted bulk densities are usually quite low (Howard et al., 1981). The bulk density of these soils are much lower than those considered to limit root growth (Minore et al., 1969; Heilman, 1981; Daddow and Warrington, 1983). But significant reductions in forest growth are common (Froehlich and McNabb, 1984), and the reduced growth rates may persist for decades (Wert and Thomas, 1981).

Many of the forest soils in western Oregon are derived from volcanic ejecta and volcaniclastic rocks (Baldwin, 1964). Soils weathered from these rocks have several atypical properties (Maeda et al., 1977). Most notable is a low bulk density, often between 0.50 and 1.0 Mg/m^3 in mesic forest ecosystems (Froehlich and McNabb, 1984;

McNabb et al., 1986). The fine-textured soils also have a high liquid limit, a high soil water content, and a low plastic index (McNabb, 1979). Soils with these properties are called andic soils (Maeda et al., 1977; Kinloch, 1987). Andic soils contain varying amounts of amorphous, halloysitic, and other unnamed noncrystalline minerals of nonrepetitive mineral structure rather than the rigid repetitive structure of layered silicate minerals (Maeda et al., 1977). More than 70 soil series in Oregon have medial, ashy, or cindery family classifications indicative of andic properties. Over two-thirds of these series are medial soils occurring west of the Cascade Mountains (Huddleston, 1979); most were or are forested. Recognition of the unique physical properties of andic soils is the basis for the proposed new order of Andisols (Kinloch, 1987) that will result in the reclassification of many forest soils in western Oregon. The consolidation and shear strength of andic soils have seldom been measured (Maeda et al., 1977), even though these soils have unique properties.

Although shear strength of most subsoil horizons and deeper layers of forest soils in the Pacific Northwest is high (Yee and Harr, 1977; Schroeder and Alto, 1983), noncrystalline minerals are thought to reduce the shear strength of forest soils in western Oregon. The presence of noncrystalline minerals in certain types of pyroclastic rocks are also thought to contribute to mass failure in the Western Oregon Cascades (Paeth et al., 1971; Youngberg et al., 1975; Taskey et al., 1978). Soils dominated by these minerals generally have lower angles of friction than soils containing layer silicate minerals (Pope and Anderson, 1961). Measuring the shear strength of soils with noncrystalline mineralogy, however, is hindered by large sample to sample variability which makes locating the soil strength line, or Mohr envelope, difficult. In other Pacific Rim countries, soils of noncrystalline mineralogy also have a low shear strength. Angles of friction are typically less than 10° (Maeda et al., 1977).

The shear strength of andic soils in western Oregon may differ from the more widely studied andic soils of other Pacific Rim countries because of differences in other soil properties. For example, many of the latter soils are reported to become nonplastic when air-dried (Warkentin and Maeda, 1974). A limited survey of fine-textured andic soils in western Oregon, however, failed to find any soils that became nonplastic when air-dried (McNabb, 1979). The soils in western Oregon also contained layer silicate minerals and other minerals of the continuum in crystallinity existing between andic materials and layer silicate minerals (Maeda et al., 1977; Jackson et al., 1986), which contributed to the plasticity of the soils.

Consolidation of andic soils from western Oregon has not been reported and few data are available from elsewhere. Two of the 36 agricultural soils studied by Larson et al. (1980) had andic properties. The compression index of the two andic soils was less than for soils containing layer silicate minerals. Maeda et al. (1977) also reported low compression indexes for andic soils. Impact compaction tests of andic soils result in a low maximum bulk density and a poorly defined optimum soil water content because of the broad peak in the compaction curve (Froehlich et al., 1980). The maximum compacted bulk density of andic soils is lower than those of other soils (Howard et al., 1981).

The objective of this study was to determine if soil strength controlled the compression of low bulk density soils. Consolidation tests of saturated soil was used to measure soil compression. Shear strength was measured in a direct shear test to confirm that increases in bulk density were controlled by differences in soil strength. Four soils were selected to represent specific types of engineering material. Three soils were andic soils or had several properties characteristic of andic soils. The results for these soils were contrasted with a cohesive soil containing layer silicate minerals.

MEASUREMENT OF SOIL DEFORMATION AND SHEAR STRENGTH

The variables and parameters describing the deformation of soil, whether by compaction, consolidation, or shear stress, vary according to the methods used (Bell, 1977; Soane et al., 1981a; Wroth, 1984). Selection of a testing procedure depends on the objective of the measurements, the properties of soil being investigated, and the applicable soil mechanics theory. Differences in the costs and number of tests required also affects the selection of methods. Tests that measure soil behavior at both saturated and partly saturated conditions are seldom made because the measurement of partly saturated soil strength is complicated by changes in air pressure in some tests and by the difficulty of measuring negative pore water pressure (Bishop and Blight, 1963).

Consolidation and Compression

Consolidation refers to the compression of saturated soil over time following the application of a static stress. Consolidation is based on Terzaghi's consolidation theory of dissipation of positive pore water pressure in saturated soil following an increase in applied stress. Consolidation is a time dependent measure of soil deformation which has two components: the dissipation of positive pore water pressure that initially controls compression and the increase in effective stress; and the compression of the soil as a function of the applied stress after the pore pressure has dissipated. Compression is defined as the decrease in soil volume, or increase in bulk density, resulting from the application of a static external stress (USDI, Bureau of Reclamation, 1974). Compression is also used to describe the increase in bulk density of partly saturated soil from the application of a static external stress (Larson et al., 1980). Terzaghi's consolidation theory does not describe the compression of partly saturated soil.

Larson et al. (1980) have described the increase in bulk density of soil using the compression index obtained with one-dimensional consolidation tests of partly saturated, disturbed samples. This model of soil compression is:

$$\rho_c = \rho_k + C_c \log (\sigma_a/\sigma_k), \qquad [1]$$

where ρ_c is the compressed bulk density (Mg/m^3) following the application of a normal stress, σ_a (MPa), ρ_k is the bulk density at a known normal stress, σ_k (MPa), and C_c is the compression index. The compression index is the slope of the linear part of the relationship between bulk density and the logarithm of normal stress (Terzaghi and Peck, 1967).

The relationship between compressed bulk density and normal stress is defined as the compression curve. The compression curve of a soil differs with soil condition. Compacted, dried, or undisturbed soils are less compressible at lower stresses. Soils which compress more slowly at lower normal stresses than at higher normal stresses are considered to be overconsolidated (Terzaghi and Peck, 1967). This part of the compression curve is not described by the compression index. Disturbance, aggregation, and degree of saturation may also affect the compression index of soils (Terzaghi and Peck, 1967; Larson et al., 1980).

The known normal stress, σ_k, is the minimum normal stress where the relationship between bulk density and the logarithm of normal stress, i.e., the compression index, is linear. The value of σ_k in Equation 1 must often be increased if Equation 1 is to remain valid for an overconsolidated soil. An increase in the value of σ_k also increases the known bulk density, ρ_k. This increases the range of bulk density at lower normal stresses which cannot be predicted by Equation 1.

More recently, Bailey et al. (1986), fit a 3-variable, nonlinear model to the entire compression curve:

$$\ln(\rho_c) = \ln(\rho_o) - (A + B\cdot\sigma)\cdot(1 - (EXP(-C\cdot\sigma)), \qquad [2]$$

where ρ_c is the compressed bulk density (Mg/m^3), ρ_o is the estimated bulk density at zero stress (Mg/m^3) and is estimated by regression, and σ is the normal stress (MPa). A, B, and C are parameters describing the compression curve. Parameters have been related to soil water content, texture, overconsolidation void ratio, organic matter content, and plasticity (McBride, 1989). The nonlinear model describes the compression curve at all stresses, which overcomes an important limitation of predicting bulk density using Equation 1.

Shear Strength

Soil strength is affected by numerous soil properties and by the method used to measure it. Soil properties affecting soil strength include bulk density, texture, gradation, normal stress, structure, temperature and stress history (Mitchell, 1976). Methods include type of test, rate of loading, range of confining stresses, drainage during compression and shear, stress history, type of stress, and criteria for failure (Lambe and Whitman, 1979).

Most measures of soil strength are based on the Mohr-Coulomb failure theory which states:

$$\tau = c + \sigma \tan \phi, \qquad [3]$$

where τ is the shear stress (MPa) on the failure plane at time of failure, c is the cohesion intercept (MPa), σ is the normal stress (MPa) on the failure plane, and ϕ is the angle of friction (degree). The cohesion intercept and angle of friction are affected by pore water pressure. Measurement of the normal stress and pore water pressure allows the calculation of effective normal stress,

$$\sigma' = \sigma - \mu, \qquad [4]$$

where σ' is the effective stress (MPa) and μ is the pore water pressure (MPa). The effective stress is the stress between soil particles. The μ is pore water pressure and is normally positive during consolidation and undrained shear testing of many saturated soils but, occasionally, may become negative during undrained shear tests of overconsolidated clay soils or dense sands at low normal stresses. The effective normal stress allows the effective shear strength, and effective cohesion intercept and angle of friction to be calculated using Equation 3.

A method of measuring soil strength cannot be selected solely on the basis of theory (Soane et al., 1981). The most accurate measure of soil strength is the method that best describes soil behavior in a specific situation. Triaxial tests are the standard to which most methods of measuring soil strength are compared (Wroth, 1984). Depending on the procedures used, a triaxial test can measure states-of-stress for constructing Mohr circles, stress paths, effective stresses, and the theoretical location of the failure plane. Although a measure of the shear strength parameters can also be obtained with a direct shear test, this test only provides a measure of the normal and shear stresses and assumes that the horizontal plane through the shear box is the failure plane. If conducted at a low rate of strain, and drainage is allowed, a direct shear test measures the effective stresses. When adjusted for differences in methodology, the angle of friction measured in triaxial and direct shear tests generally differs by less than 2°, especially for dense sand (Lambe and Whitman, 1979). Differences between angles of friction can be larger for fine-textured soils because errors are more likely from assuming that pore pressures are zero in direct shear tests.

Index tests, such as vane shear tests and cone penetrometers, were developed for in situ measurement of shear strength of normally consolidated, fine-textured soils (Wroth, 1984). As a group, index tests are more sensitive to differences in soil type, bulk density, and stress history than other methods. Using index tests to compare the shear strength of different soils with a wide array of soil textures and stress histories is uncertain because of test dependent effects on soil strength.

Conclusion

In the absence of a preferred method for measuring soil compaction, methods should be selected which accurately reflect differences in and among soils. For forest soils which are generally compacted in situ, one-dimensional consolidation tests of undisturbed soil is a reasonable choice that allows a wider range and greater control of stresses than impact compaction tests. Furthermore, bulk densities obtained with different impact tests have not been related to the compaction of forest soil by machines in the field (Froehlich et al., 1980).

Consolidation increases soil strength because of an increase in the bulk density (Hvorslev, 1961; Lambe and Whitman, 1979). Measuring soil strength provides a basis for comparing bulk densities and increases in bulk densities of different types of soil. This is possible if the method used to measure soil strength is not biased by differences in soil material. Soil strength measured in direct or triaxial shear tests provides the most reliable basis for making comparisons among soils. Direct shear tests can be performed in much less time than triaxial tests, which is an important factor when many samples must be tested. Samples are also consolidated in a one-dimensional consolidation test prior to the measure of shear strength in a direct shear test.

MATERIALS AND METHODS

Soil Materials

Measurement of soil compression and shear strength in one-dimensional consolidation and direct shear tests limited the number of soils which were tested. Four types of soil material were identified that included a wide range of soil properties: 1) a coarse-textured volcanic ash; 2) a low plasticity soil with an organic matter content >15 percent; 3) a medium- to fine-textured soil with low plasticity; and 4) a fine-textured soil with moderate to high plasticity. These criteria insured that the first three soils had andic properties and the fourth soil had a mineralogy dominated by layer silicate minerals. Only one site was sampled for each type of soil.

Soil series and other soil mapping units with the required properties were identified by checking Soil Surveys, USDA Forest Service Soil Resource Inventories and soil series data sheets for soil materials with the required properties. The criteria used to select series or mapping units included soil texture, clay mineralogy (taken from family name or inferred from the Atterberg limit measures of plasticity (McNabb, 1979)), and organic matter content taken from series descriptions or inferred from soil color. This procedure identified one or two soil series or mapping units per type of material.

Possible sample locations of each soil series were identified and checked in the field to verify the series, presence or absence of soil disturbance, and the possibility of excavating a large number of undisturbed soil cores. Sites with a large number of downed logs, dense shrub understory, and numerous large roots at the 0- to 20 cm depth, were avoided. Atterberg tests were performed on samples from locations with fine-textured soils that had a low plasticity, or a moderate to high plasticity. The two samples from the sites with the greatest difference in soil plasticity became the sites where the fine-textured andic soil and cohesive non-andic soil were collected.

The four soil series, their taxonomic classification, and approximate sample location were:

1) Crater Lake soil - medial, frigid, Typic Vitrandept
(42° 40' N. Lat., 122° 19' W. Long.);

2) Tolovanna soil - medial, mesic Typic Dystrandept
(45° 1.5' N. Lat., 123° 58' W. Long.);

3) Hemcross soil - medial, mesic Andic Haplumbrept
(45° 16' N. Lat., 123° 39' W. Long.); and

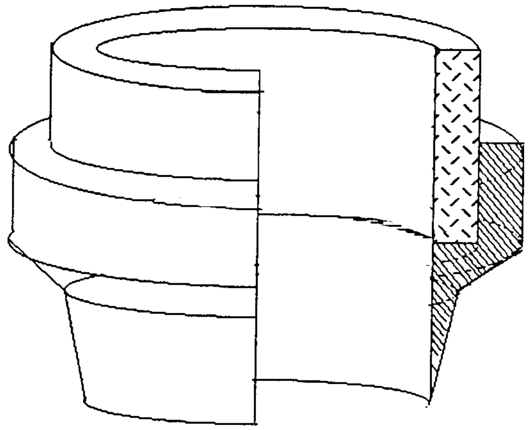
4) Jory soil - clayey, mixed, mesic Xeric Haplohumult
(43° 57' N. Lat., 123° 21' W. Long.).

All sample sites were in stands of mature timber and were protected from earlier partial harvests by adjacent trees. The Hemcross site had been significantly disturbed by the burrowing of mountain beaver (Aplodontia rufa (Raf.)). This activity had mixed horizons over large areas. Areas with obvious disturbance were avoided.

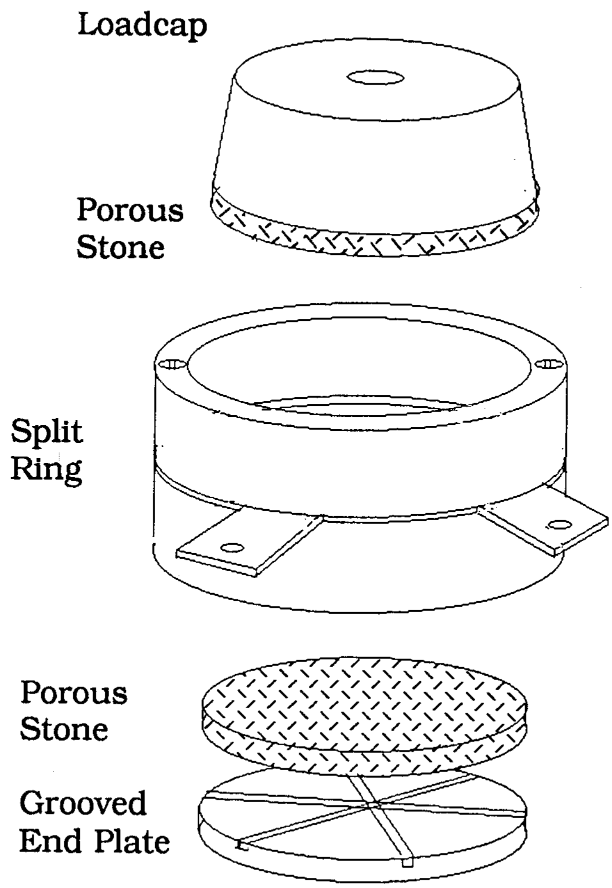
Collection of Undisturbed Soil Cores

Core samples were collected from three locations across an area of approximately 100 m^2. Areas with stumps, downed logs, dense understory vegetation, and large roots were avoided to reduce the difficulty of obtaining undisturbed samples. Samples were collected a few days after cessation of a heavy rain, when the soil water content was near field capacity. The forest floor, debris, and mineral soil to a depth of 7 cm was removed from an area of about 2 m^2. Surface soil was removed because its variability was assumed to be higher and compaction of soil by machines often displaces surface soil (Froehlich et al., 1980). A trench, about 50 cm deep, was excavated along one side of the cleared soil to expose roots and aid in excavation of cores. Undisturbed soil cores were collected from the 7- to 12 cm depth.

Core samples were collected in rings of polyvinyl-chloride (PVC) pipe, with a height of 3.5 cm. Inside diameter of rings was 7.45 cm and the wall thickness was 0.85 cm. For sampling, a ring was placed in a cutting head with a beveled cutting edge (5°) which guided the cutter-ring assembly into the soil (Figure II.1). The cutter-ring assembly was placed on the soil surface near the edge of the shallow trench. A light pressure was applied to the top of the cutter-ring assembly by hand while the soil was cut away from the edge of the cutter by hand. The procedure is similar to one described by the USDI, Bureau of Reclamation (1974). Samples were discarded if rock fragments >5 mm, or roots >2 mm, in diameter were encountered. Smaller roots were clipped at the edge of the cutter to minimize disturbance of the core. Cores occasionally contained rock fragments up to about 1 cm in diameter.



PVC Sample Ring and
Cutter Head



Split Ring Sample Holder

Shear Box Assembly

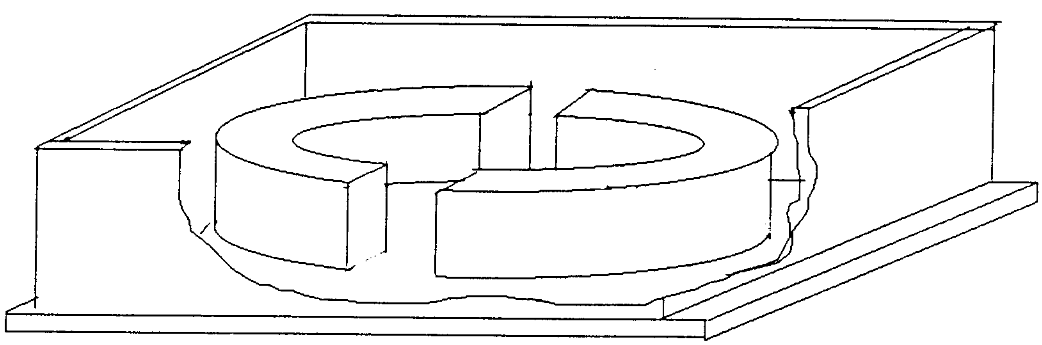


Figure II.1. Diagrams of cutter head and sample ring, split ring sample holder, and shear box assembly.

Filled assemblies were removed from the profile, the ring was removed from the cutter head and the ends were trimmed flush with the ring. Filled rings were separated by polycarbonate discs (0.08 cm thick and approximately 7 cm in diameter) and stacked in columns of six rings. The stacks of six cores were taped together, and the exposed ends were covered with a polycarbonate disc, parafilm, and plastic wrap, which were then sealed with tape.

Consolidation/Direct Shear Machine

A machine was built which performed the functions of both a level-action consolidation frame (one-dimensional consolidation) and a direct shear machine (Figure II.2). The machine had six positions for conducting simultaneous consolidation tests. Direct shear tests were performed individually using a single motor, drive, and load cell that traveled on a track among positions.

Core samples were transferred from the PVC ring to a split-ring, which was then clamped into a removable, consolidation/shear box assembly during testing (Figure II.1). The split-ring consisted of two sections of PVC pipe with the same dimensions as those used for sample rings. The sections were separated with four spacers, 0.08 mm thick. The split ring and spacers were held together during transfer and consolidation by two screws. The screws were retracted and spacers were removed before the direct shear test. The purpose of separating the rings was to eliminate friction between rings during the direct shear test. A porous stone was placed against each end of the

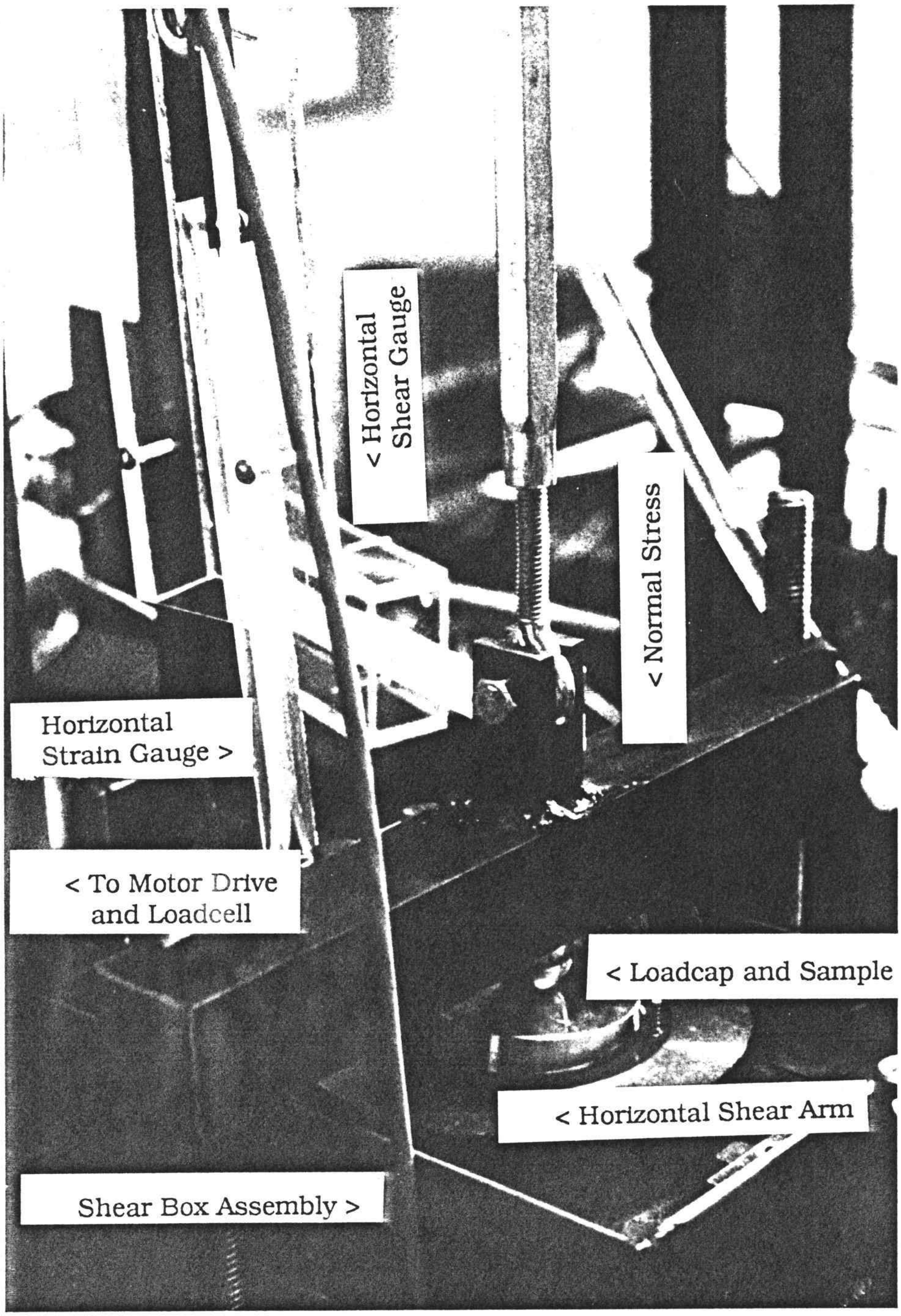


Figure II.2. One position in the six position consolidation frame and direct shear test machine.

soil core, with a grooved drain plate added to the bottom, and a load cap added to the top before placing the sample in the consolidation/shear box assembly (Figure II.1).

Consolidation Test

Consolidation data were obtained with two types of tests. The first type of test measured consolidation at several normal stresses before the shear strength was measured at the highest normal stress. In the second type of test, samples were consolidated at one of several normal stresses and the shear strength was measured.

For both types of tests, a core sample was placed in the consolidation/shear assembly, saturated from below with distilled water, and allowed to equilibrate for 12 hours. The height of the soil core was measured while in the split-ring which, in turn, was in the consolidation/shear box assembly. The normal stress on the sample was the load cap and top stone. This stress was 0.002 MPa. The measured height was used to calculate the initial bulk density of the sample. Change in the height of the core sample was measured with a linear voltage displacement transducer with an accuracy of 0.002 mm. The output of the strain gauges was monitored with a Campbell 21X™ data logger.

In the first type of test, a succession of 6 normal stresses was used to consolidate each sample: namely, 0.033; 0.063; 0.125; 0.25; 0.49; and 0.98 MPa. The lowest stress was applied to a sample and the height of the sample was measured at 0.25, 0.5, 1, 2, 4, 8, 16, 32, 64, 126, 256, and 512 min. By the last measurement, secondary compression was low. The increase in bulk density between 256 and 512 min was seldom more than 0.005 Mg/m^3. After the last measurement, the next highest stress was applied and the height of the sample measured for the same time intervals. Following the last measurement at a normal stress of 0.98 MPa, the stress was released in four increments to 0.033 MPa, then reapplied to determine the recompression curve. The normal stresses for rebound and recompression were changed at 32 min intervals. After the second application of the 0.98 MPa stress, the normal stress was increased to 1.96 MPa and allowed to remain at this level for 512 min. This sequence of measurements completed the first type of consolidation test. Three samples of each soil were tested.

In the second type of test, the sample was placed in the machine, initial data were recorded, and one normal stress was applied. The normal stress was 0.033, 0.063, 0.094, 0.125, 0.25, 0.49, or 0.98 MPa. Compression of the sample was recorded at the times described earlier. A direct shear test was performed after the last measurement. Three samples were tested at each stress.

Direct Shear Test

After the consolidation test, the shear arm was connected to the motor and load cell (Figure II.2). The screws holding the split-ring together were retracted, the spacers between the rings were removed, and a second linear voltage displacement transducer was placed against the shear arm to measure horizontal strain. Each core was sheared at the rate of 0.5 mm/min for a total horizontal displacement of 1 cm. The failure criterion was the shear strength measured at a horizontal strain of 10 percent. A peak strength at a strain <10 percent was not measured on any sample.

Soil Characterization

Bulk soil samples from the 7- to 12 cm depth were collected from 3 to 4 soil pits at each location. The liquid limit and plastic index were measured on soils at water contents as collected in the field according to ASTM test D4318-84 (ASTM, 1985). The remaining soil was air dried for the following determinations: organic matter content by Walkley-Black (Kauffman and Gardner, 1976); particle density by the pycnometer method (Blake and Hartge, 1986); and particle size analysis by the hydrometer method (Gee and Bauder, 1986).

Statistical Analyses

The Statistical Analysis System (SAS, 1986) was used for all analyses. The Marquardt nonlinear curve fitting technique was used to fit curves to: 1) samples consolidated at several normal stresses; and 2) samples consolidated at one normal stress. Linear regression was used to estimate the compression index, C_c, in Equation 1 for normal stresses > 0.10 MPa of samples consolidated at several stresses. For normal stresses > 0.10 MPa, the relationship between bulk density and the logarithm of the normal stress was linear. Linear regression was also used to analyze the relationship between bulk density and the logarithm of normal stress when the normal stress was reduced; the slope of this relationship was the rebound index, C_r.

Analysis of variance was used to determine significant differences among parameters of Equations 1 and 2 for each soil from the first phase of tests.

Linear regression was used to determine the shear strength variables for the Coulomb failure criterion for normal stresses between 0.032 and 1.96 MPa. A second regression was carried out to determine shear strength parameters for normal stresses between 0.032 and 0.10 MPa to check for curvature of the shear strength line and the possibility of obtaining a cohesion intercept of zero (Mitchell, 1976).

RESULTS

Soil Properties

The four soils met most of the criteria listed for selection (Table II.1). The organic matter contents of the Hemcross and Tolovanna soils were higher than anticipated, but their bulk densities, high liquid limits, and low plastic indexes resulted in a plasticity angle similar to other andic soils in western Oregon (McNabb, 1979). The low particle densities of Hemcross and Tolovanna soils are, in part, the result of the high organic matter content, but they may also have resulted from incomplete saturation of porous silt- and sand-sized particles. These particles may be pseudomorphs of clay minerals or other porous particles (Paeth et al., 1971; Flint and Childs, 1984). The variation in bulk density is similar to that reported for surface soils from the western Oregon Cascades (McNabb et al., 1986).

Consolidation of Soils

Although the height/diameter ratio of the samples was near the upper limit recommended for one-dimensional consolidation tests (Lambe, 1951), consolidation of all samples was rapid (Figure II.3). Increasing the applied stress resulted in an immediate, large increase in bulk density. Primary consolidation was generally completed within about 0.25 min. Large pore water pressures were unlikely to develop with these rapid rates of consolidation and low bulk densities (Terzaghi and Peck, 1967). Secondary consolidation, which occurred after 1 min, accounted for about 50 percent of the increase in bulk

Table II.1. Physical properties and organic matter content of four western Oregon forest soils. Data are for a composite sample of soil from the 7- to 12 cm depth.

Soil	Particle Size Distribution Sand	Silt	Clay	Texture	Unified Class.	Bulk Density	Particle Density	Atterberg Limits θ_l	I_p	Organic Matter
	----- kg/kg -----					-- Mg/m^3 --		---- kg/kg ---		
Crater Lake	0.635	0.308	0.057	SL	SM	0.707 (0.023)††	2.50	NP†	--	0.026
Hemcross	0.153	0.493	0.354	SiCL	MH	0.577 (0.036)	2.28	0.911	0.051	0.180
Tolovanna	0.350	0.392	0.258	L	MH-OH	0.482 (0.034)	2.24	1.170	0.090	0.254
Jory	0.348	0.355	0.297	CL	MH	0.992 (0.059)	2.44	0.555	0.187	0.060

† Atterberg limits: liquid limit, θ_l, plastic index, I_p, and nonplastic, NP.
†† Standard error of bulk density (d.f. = 17 to 22).

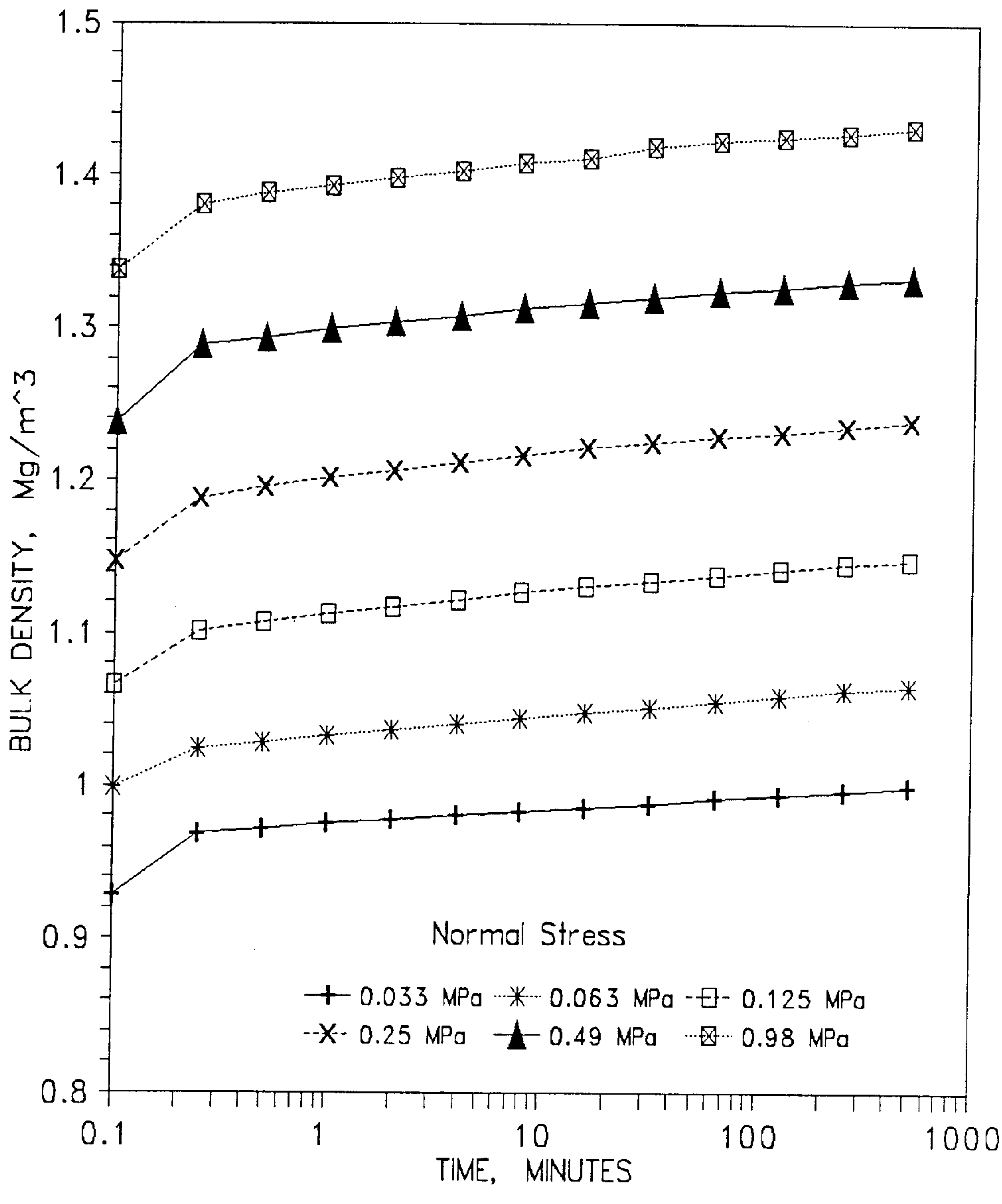


Figure II.3. Increases in bulk density as a function of time at six normal stresses for an undisturbed, saturated sample of Jory soil.

density at each normal stress.

Differences in the initial bulk density of samples affected the compression curves of each soil at lower normal stresses (Figure II.4). Differences in compressed bulk density decreased at higher applied stresses. The failure of the curves to merge into one compression curve suggests that the differences in fabric and structure contributing to variation in initial bulk density were not totally destroyed by consolidation.

Figure II.5 shows one compression and rebound curve for each of the four soils evaluated. Only the compression curve of the Jory soil suggested the occurrence of overconsolidation. However, the overconsolidation stress was estimated to be less than 0.06 MPa (Figure II.4; Terzaghi and Peck, 1967).

The relationship between bulk density and the logarithm of normal stress of the andic soils were curved at higher stresses (Figure II.5), rather than linear, which is more typical (Terzaghi and Peck, 1967; Larson et al., 1980). The fact that the compression curve of the Crater Lake soil was not linear at higher stresses may have resulted from crushing of porous ash particles (Youngberg and Dyrness, 1964).

Compression Curves of Individual Samples

The compression index, C_c, of the Jory soil was significantly higher than those of the andic soils (Table II.2). The compression index of the andic soils was not significantly different, although the

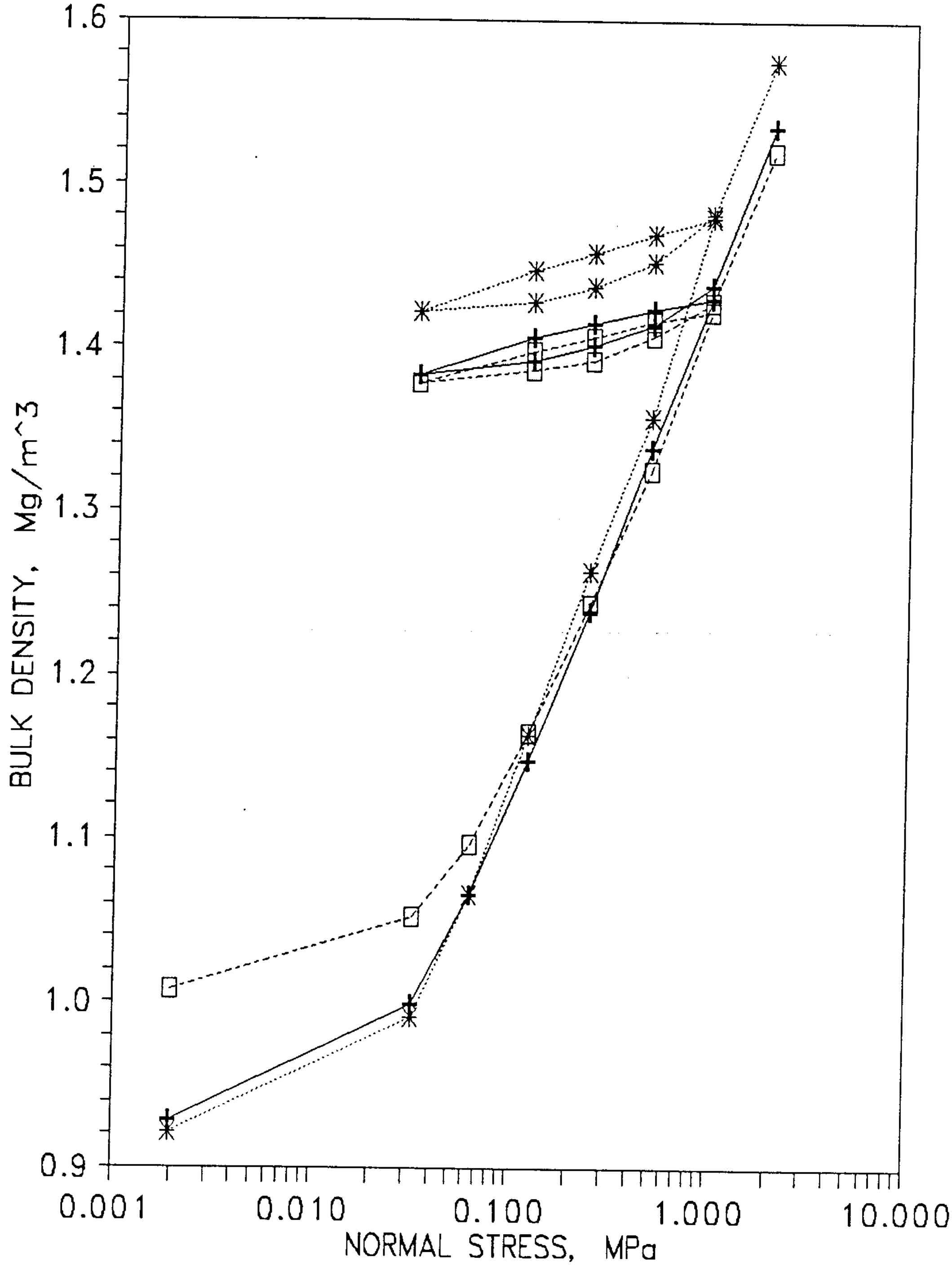


Figure II.4. Compression and rebound curves of three saturated, undisturbed samples of Jory soil.

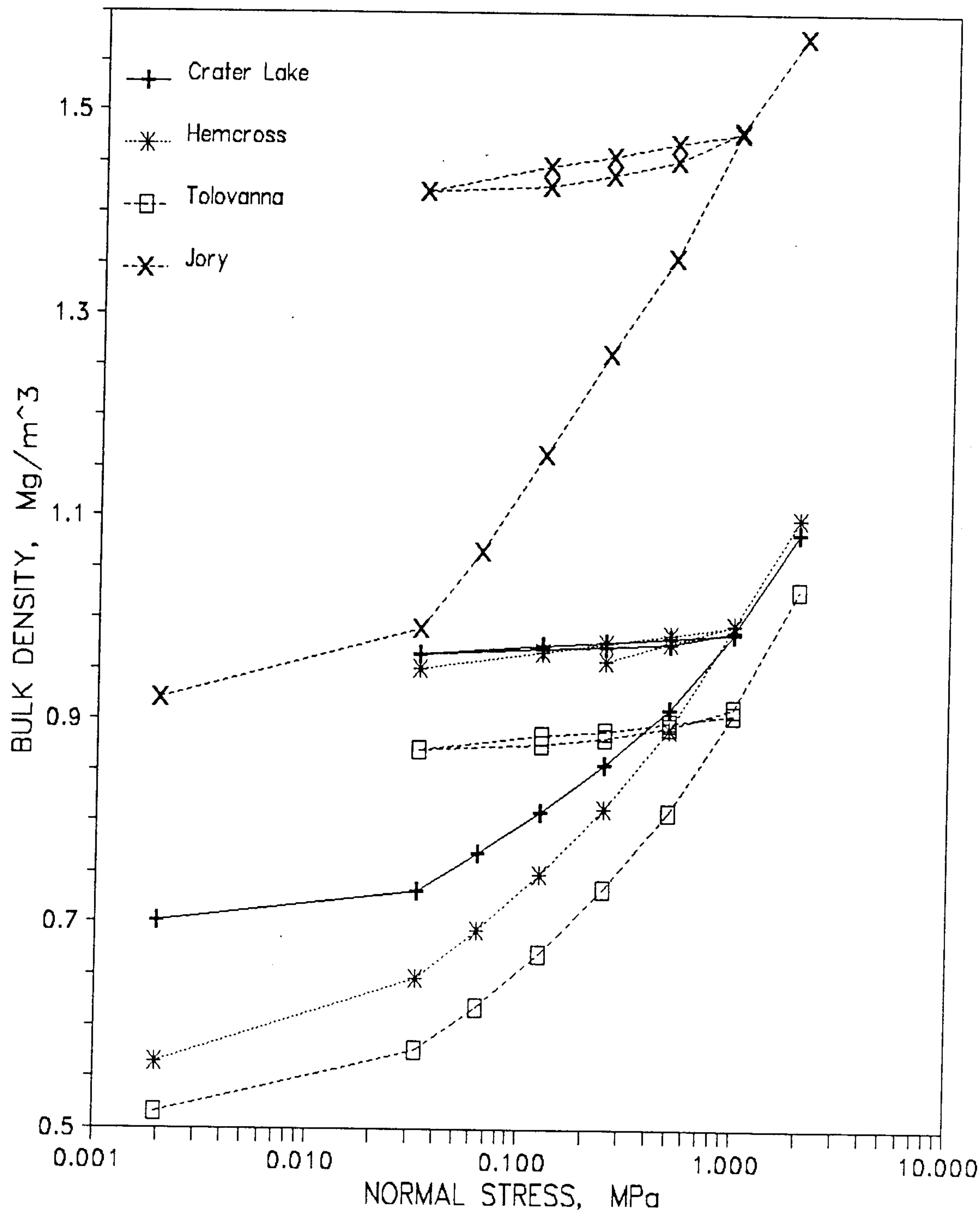


Figure II.5. Typical compression and rebound curve for each soil.

Table II.2. Parameters for nonlinear model (Eqn 2) of all normal stresses, and the compression index, C_c and rebound index, C_r, of soil at normal stresses greater than 0.1 MPa. Data are results of consolidating three samples over a range of normal stresses between 0.033 and 1.96 MPa.

Soil	Nonlinear Model Parameters					
	ρ_o	A	B	C	C_c	C_r
Crater Lake	0.718b†	-0.236a	-0.109b	7.686b	0.231bc	0.0046c
	(0.010)††	(0.011)	(0.005)	(0.025)	(0.004)	(0.0006)
Hemcross	0.560c	-0.389c	-0.142c	10.848a	0.275b	0.0079bc
	(0.026)	(0.020)	(0.010)	(0.153)	(0.013)	(0.0027)
Tolovanna‡					0.262bc	0.0134ab
					(0.025)	(0.0008)
Jory	0.953a	-0.320b	-0.084a	7.457b	0.320a	0.0143a
	(0.028)	(0.035)	(0.002)	(0.539)	(0.014)	(0.0014)

† Parameters in each column followed by the same letter are not significantly different ($p < 0.05$).

†† Average standard error of parameters.

‡ B and C parameters of nonlinear model were not significantly different from zero.

compression index of the Crater Lake soil was lower than those of the fine-textured andic soils. This was expected because a one-dimensional consolidation test is an inefficient method of consolidating coarse-textured soil (Lambe and Whitman, 1979).

The compression index of the Jory, Hemcross, and Tolovanna soils were less than those reported by Larson et al. (1980) for disturbed samples of agricultural soils. They did not test a soil similar to the Crater Lake soil. They reported an average compression index of 0.36 for two andic soils from Hawaii, which is higher than the average compression index of 0.26 measured for the Hemcross and Tolovanna soils (Table II.2).

The rebound index, C_r, of the Crater Lake soil was the smallest while those of the Tolovanna and Jory soils were highest, probably because of their high organic matter content and matrix of mixed, layer silicate minerals (Table II.2). The bulk density of the soils decreased only a small amount, namely from 0.02 to 0.05 Mg/m^3 when normal stress was reduced from 0.98 to 0.032 MPa (Figure II.5). Values of the rebound index were similar to those of agricultural soils containing layer silicate minerals (Stone and Larson, 1980).

The nonlinear model of compression, Equation 2, predicted bulk density as a function of normal stress for each sample of the Crater Lake, Hemcross, and Jory soils (Table II.2). Equation 2 failed to predict the bulk density of the Tolovanna soil because the B and C parameters were not significantly different from zero. The normal stress between 0.002 and 1.96 MPa only included 8 measurements for each sample. More measurements of bulk density at normal stresses within this range would probably have resulted in significant parameters for the Tolovanna soil (McBride, 1989).

Composite Compression Curves

The data for all normal stresses included 19 or more individual measurements of bulk density and normal stress that were used to develop a composite compression curve for each soil using Equation 2. The parameters for a composite compression curve were significant for the Crater Lake, Tolovanna, and Jory soils (Table II.3). The B and C parameters were not significantly different from zero for the Hemcross soil. Variation in bulk density of individual samples reduced the fit of the model to the data (Figure II.6). The range in initial bulk density at a normal stress of 0.002 MPa, exceeded the range in compressed bulk density at higher normal stresses. The variation in compressed bulk density suggests that differences in fabric and structure responsible for the variation in undisturbed bulk density were not destroyed by compression.

The values of ρ_o, which are an estimate of the bulk density of a soil at a normal stress of zero, were higher than the average initial bulk density measured at a normal stress of 0.002 MPa (Tables II.1 and II.3). As a result, Equation 2 does not accurately predict bulk density at low stresses. Modification of Equation 2 to account for the variation in bulk density should improve the fit of the model to the data at all stresses.

Table II.3. Parameters for nonlinear models (Eqns 2 and 5) of the consolidation of individual samples. Samples were consolidated to a specific normal stress. The D parameter adjusts Equation 5 for variation in initial bulk density of samples. The standard error of each parameter is in parenthesis.

Soil	n	Parameters ρ_o	A	B	C	D	R^2
Equation 2							
Crater Lake	21	0.718 (0.030)	-0.229 (0.040)	-0.108 (0.020)	10.419 (3.855)	-	0.939
Hemcross	20	-	-	-	-	-	-
Tolovanna	19	0.523 (0.018)	-0.407 (0.048)	-0.109 (0.032)	5.841 (1.722)	-	0.966
Jory	23	1.003 (0.029)	-0.266 (0.032)	-0.083 (0.020)	6.619 (2.038)	-	0.955
Equation 5							
Crater Lake	42	0.703 (0.004)	-0.245 (0.015)	-0.112 (0.012)	12.210 (1.743)	1.225 (0.301)	0.980
Hemcross	40	0.576 (0.005)	-0.371 (0.040)	-0.132 (0.028)	8.113 (1.713)	1.499 (0.424)	0.975
Tolovanna	38	0.478 (0.003)	-0.447 (0.018)	-0.135 (0.013)	10.539 (0.903)	1.596 (0.200)	0.992
Jory	46	0.988 (0.003)	-0.292 (0.013)	-0.079 (0.008)	6.091 (0.540)	0.980 (0.069)	0.994

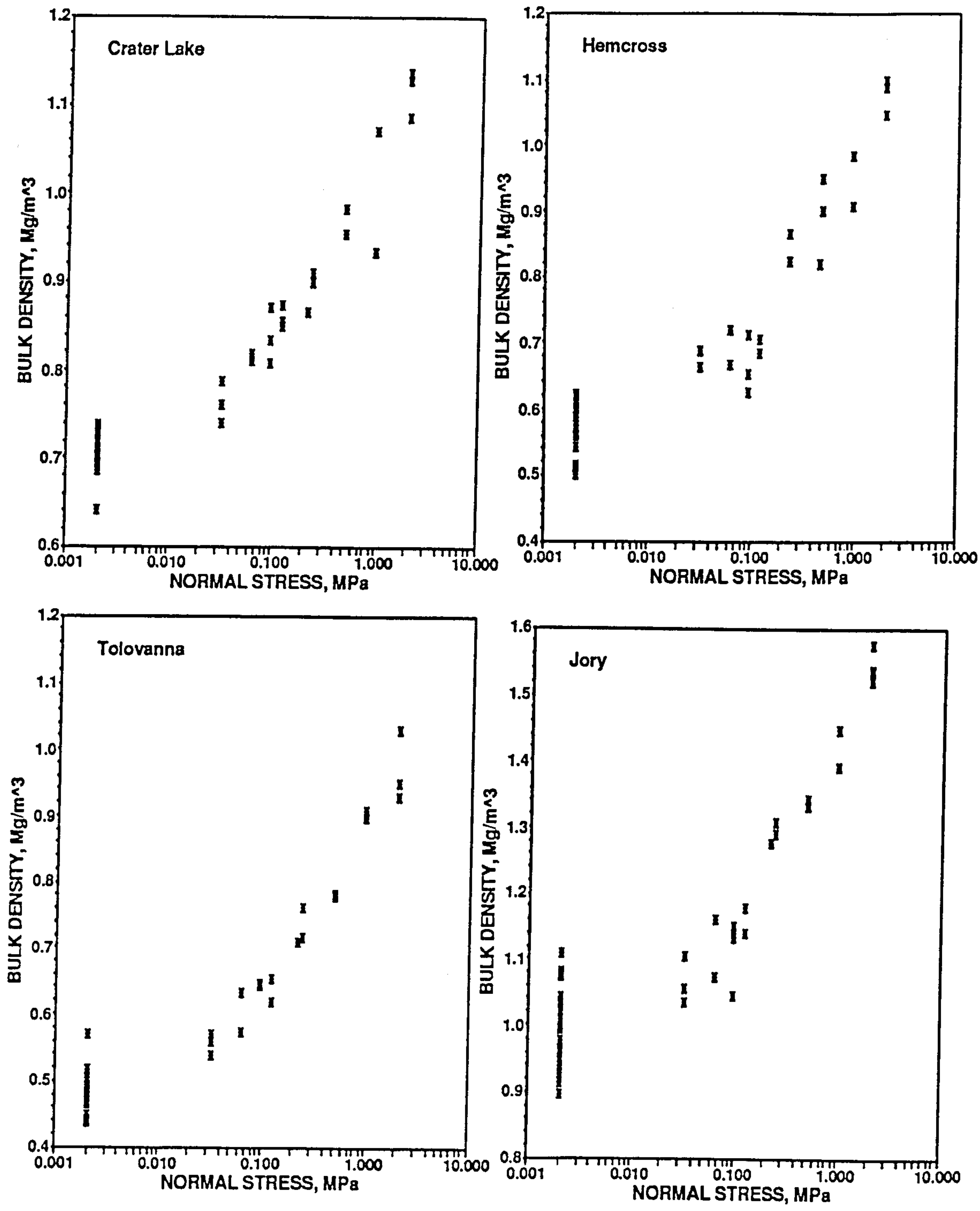


Figure II.6. Bulk density as a function of normal stress of individual samples of each soil. Bulk densities at the lowest normal stress indicate the range in initial bulk densities of each soil.

Several modifications to Equation 2 were evaluated to account for the variation in bulk density among samples and improve the fit of the model to the data. The first modification was to include initial bulk density, ρ_i, at a normal stress of 0.002 MPa in the data set. This provided a measure of bulk density at a low normal stress but did not improve the estimate of ρ_o. Inclusion of these data, however, was important for determining the shape of the compression curve at lower stresses in subsequent modifications. The second modification was to normalize the estimated bulk density at a normal stress of zero, ρ_o, for variation in initial bulk density of each sample:

$$\delta_i = \rho_i - \frac{\overset{n}{\Sigma} \rho_i}{n}, \qquad [3]$$

where ρ_i is the bulk density of individual samples. This modification reduced the mean square error of the model for each soil. The next step was to add a second variable and parameter to Equation 2 that adjusted the compression curve for variation in bulk density:

$$\delta_c = (\delta_i - 1) \cdot \rho_o, \qquad [4]$$

where δ_c is the adjustment of the compression curve for the difference in initial bulk density of individual samples. If the value of ρ_o was not normalized for the variation in initial bulk density, the parameter for δ_c was not significantly different from zero ($p < 0.05$).

When the value of ρ_o was normalized for variation in initial bulk density and δ_c was added as a second variable, the fit of the model to the data improved the prediction of bulk density for all soils. In its final form, the modified nonlinear model was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c) \cdot (1 - EXP(-C \cdot \sigma)). \qquad [5]$$

All parameters for each soil were significantly different from zero (Table II.3). The standard error of most parameters using Equation 5 was less than half of the standard error using Equation 2. The largest reductions in standard error occurred for the estimate of ρ_o. This indicates an improvement in the prediction of bulk density at low normal stresses. The bulk densities predicted with Equation 5 for each soil are shown in Figure II.7. The increase in bulk density of the Jory soil at a normal stress of 2 MPa was 0.05 Mg/m^3 larger than the increase in bulk density of the Hemcross and Tolovanna soils. Differences in the parameters for the nonlinear model were not indicators of the compressibility of these soils, because the parameters describe the difference between the logarithm of bulk density when the normal stress is zero and the logarithm of compressed bulk density (Eqns 2 and 5). As a result, the parameters are affected by the bulk density of the soils. For example, the difference between the logarithm of the bulk density of a soil at 0.5 Mg/m^3 and at 0.9 Mg/m^3 is 0.588, whereas the difference between the logarithm of a bulk density of a soil at 1.0 Mg/m^3 and at 1.4 Mg/m^3 is 0.336. The lowest bulk density in these two examples are similar to the initial bulk densities of the Tolovanna and Jory soils (Table II.1). Thus, parameters describing a specific increase in bulk density of a soil with a low bulk density are larger than those of a soil with a high bulk density.

Equation 2 fits a compression curve to the data obtained from

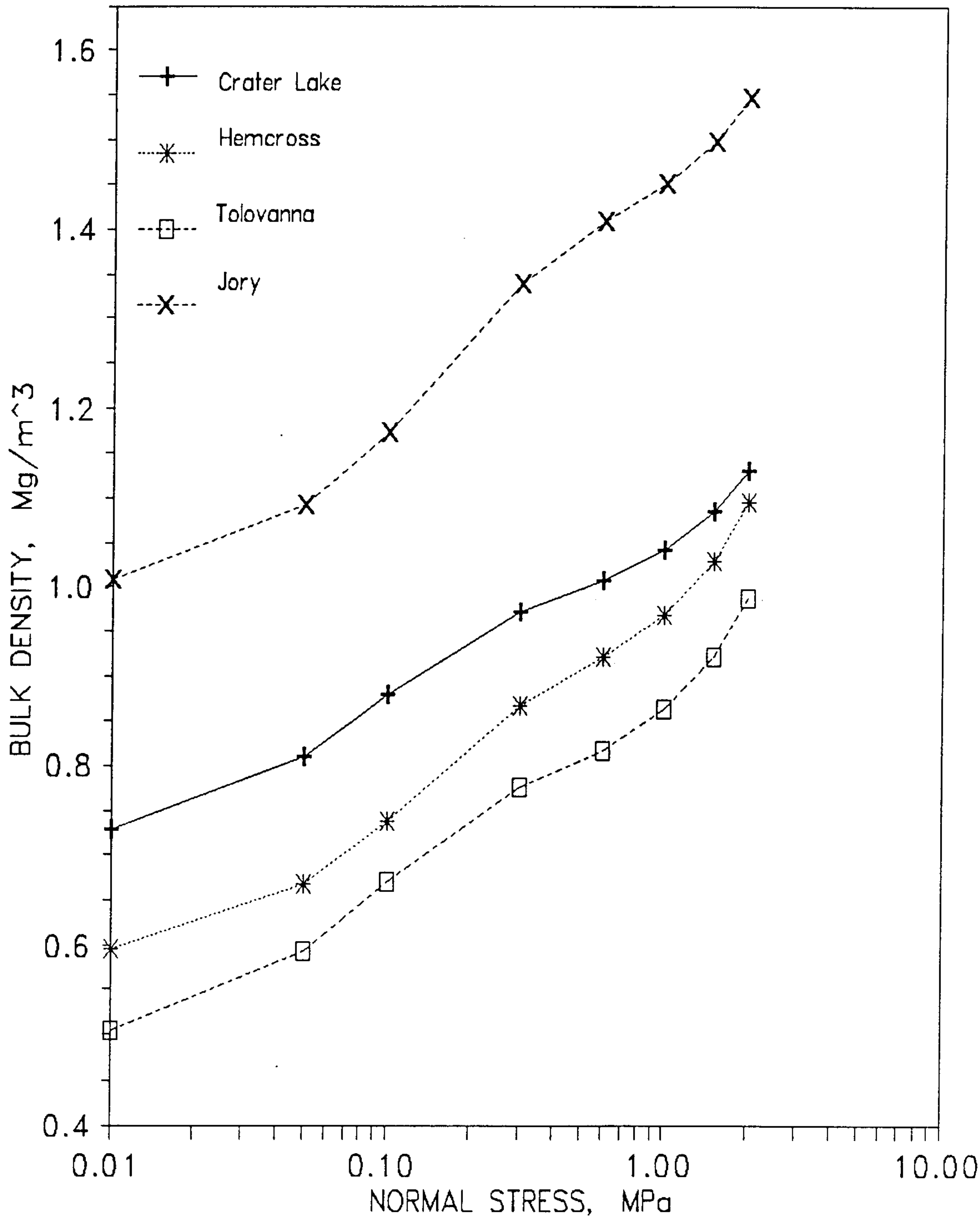


Figure II.7. Bulk density calculated using Equation 5 as a function of normal stress.

compression of an individual sample compressed at several normal stresses. Equation 5 fits a general compression curve to data obtained from individual samples when the data is adjusted for the variation in bulk density (Tables II.2 and II.3). As a result, the parameters of Equation 5 described the average compression curve for each soil.

Shear Strength

All soils behaved similarly in the direct shear tests. Samples continued to consolidate as they were sheared and a peak shear stress was not measured at any normal stress (Figure II.8). Therefore, the shear strength of all soils was the shear stress that was measured at 10 percent strain. The compression of soil, and failure to develop a peak shear stress during shear, are characteristic of normally consolidated soils (Mitchell, 1976; Lambe and Whitman, 1979).

Variation in the relationship between shear strength and normal stress of each soil was low (Figure II.9). The shear strengths of the andic soils were similar despite the large differences in soil properties (Table II.4). Shear strength of the andic soils was significantly higher than that of the Jory soil. The cohesion intercepts of the Jory and Crater Lake soils were significantly different from zero while the cohesion intercept of the Hemcross and Tolovanna soils were not. The cohesion intercept of other Oregon Coast Range soils are small or have been assumed to be zero (Yee and Harr, 1977; Schroeder and Alto, 1983). The high cohesion intercept of the Crater Lake soil is unusual

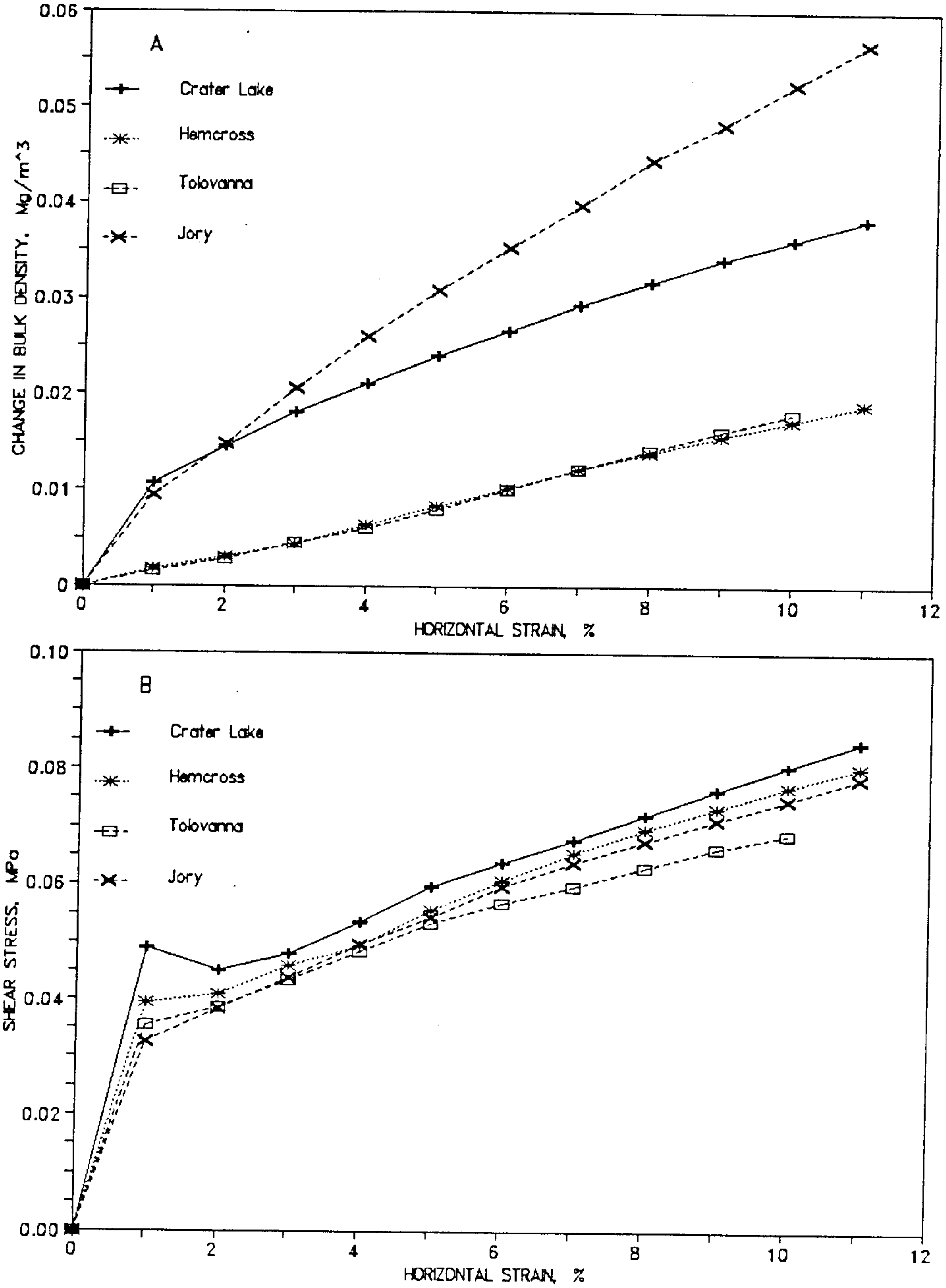


Figure II.8. Example of changes in bulk density (A), and shear stress (B) as a function of horizontal strain for each soil. The normal stress was 0.094 MPa.

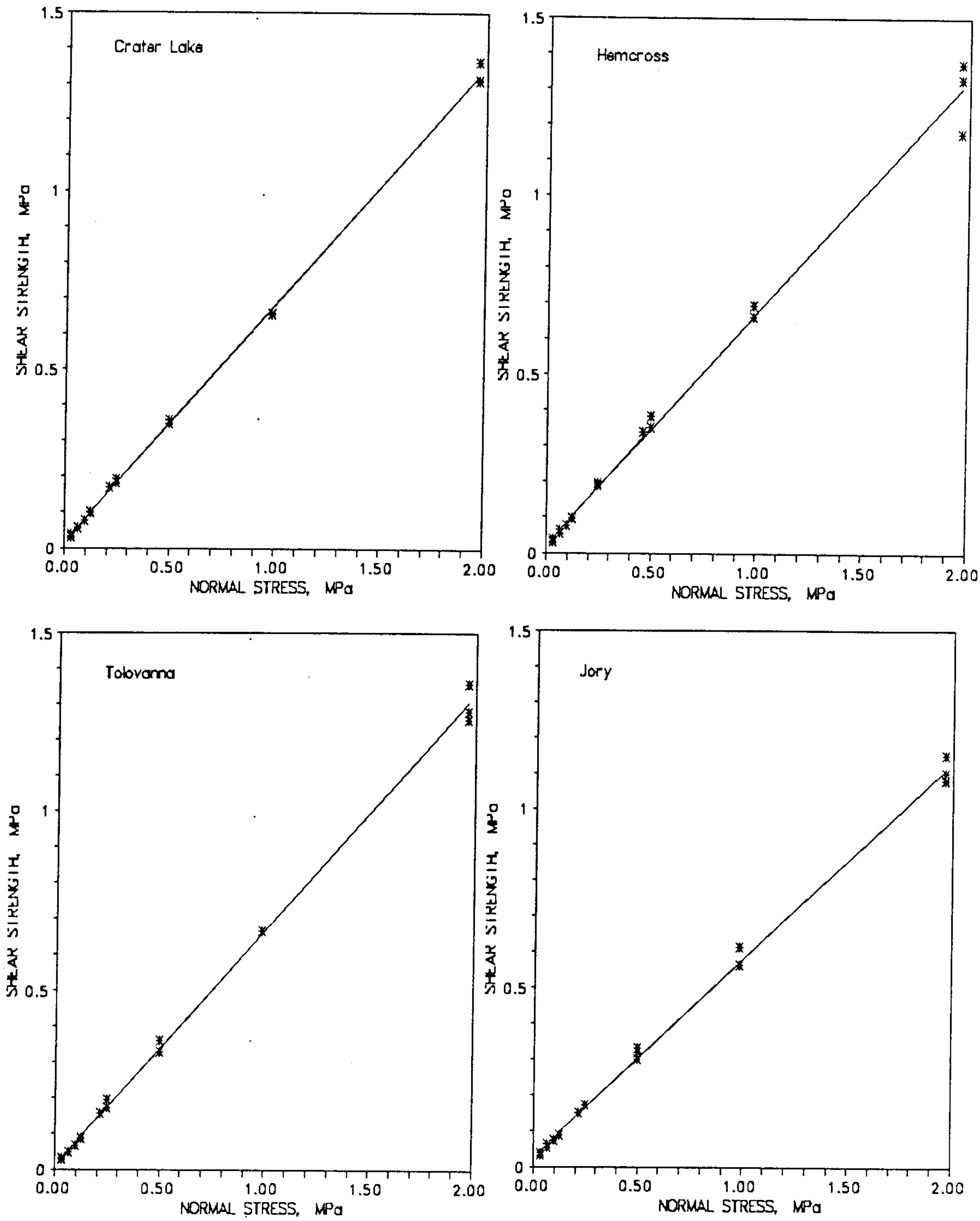


Figure II.9. Relationship between shear strength and normal stress of individual samples, and the shear strength line (Table II.4) for each soil.

Table II.4. Cohesion intercepts, c, and angles of friction, ϕ, of undisturbed, saturated samples of four western Oregon forest soils. Parameters were calculated for two ranges in normal stress.

Soil	σ (0.033 to 1.96 MPa)				σ (0.033 to 0.10 MPa)			
	n	c	ϕ	R^2	n	c	ϕ	R^2
		MPa	°			MPa	°	
Crater Lake	21	0.015†	33.7	0.999	8	0.010†	35.3	0.990
Hemcross	20	0.021	33.0	0.993	8	0.011	34.1	0.943
Tolovanna	19	0.011	33.3	0.998	7	0.008†	31.9	0.993
Jory	23	0.028†	28.9	0.998	9	0.015†	32.9	0.970

† Cohesion intercept is significantly different from zero ($p < 0.05$).

for a soil with a sandy loam texture (Table II.1), but the samples contained numerous roots which often held the soil together even after the shear test. Roots generally increase the shear strength of soil (Waldron et al., 1983).

The shear strength line of each soil was approximately linear (Figure II.9). An analysis of the shear strength of results at normal stresses less than 0.1 MPa indicated that the shear strength lines were slightly curved near the origin (Table II.4). The cohesion intercepts of results at lower normal stresses were smaller than the cohesion intercepts of results at normal stresses between 0.033 and 1.96 MPa. A smaller cohesion intercept near the origin generally increased the angle of friction of the shear strength line of results at lower normal stresses.

DISCUSSION

The consolidation of saturated soils is initially controlled by the development of pore water pressures when normal stress increases (Terzaghi and Peck, 1967). Consolidation of the saturated forest soils of this study was rapid (Figure II.3) because of their high porosities. Field capacity of these soils is at a degree of saturation of 60 percent or less (unpublished data). A high air-filled porosity is typical of most forest soils in western Oregon (Dyrness, 1969; Brown, 1975). Because these soils are seldom saturated under natural conditions, they are likely to undergo considerable compression when wet and a mechanical stress is applied, regardless of the duration of the loading.

The compression curves indicate that these soils were only slightly overconsolidated when saturated (Figure II.5 and II.7). The high turnover of roots (Santantonio, 1982) and other biological activity (Fogel and Hunt, 1979; Dick et al., 1988) which occur in these soils are probably important factors disturbing and mixing them and, thus, preventing their permanent overconsolidation. Because these soils were not overconsolidated, the predicted bulk density increased from 15 to 30 percent with the application of a normal stress of only 0.1 MPa (Figure II.7). This compression occurred in the part of the compression curve that was not described by the compression index of the soil. Equation 1 cannot predict bulk density at these low stresses, but Equation 2 or 5, depending on the method used to obtain data, did predict bulk density at the lower normal stresses.

Differences in the compressibility of individual samples also affected the parameters obtained with Equation 2. Bailey et al. (1986), needed at least 4 samples of disturbed soil to predict the most variable parameter within ±10 percent at 95 percent probability (Snedecor and Cochran, 1967). From 1 to 14 samples were needed to predict at least one of the parameters with a similar degree of accuracy for the undisturbed samples in this study. Three samples of the Crater Lake soil were the fewest number needed, and 14 samples of the Jory soil were needed because of the variability of the B parameter (Table II.2). These estimates of sample size were based on the variability of the three individually consolidated samples. The variation in bulk density of the three samples of each soil used in Equation 2 was less than that observed in the samples used in Equation 5 and in the shear strength analysis (Table II.1). The variation in bulk density of the samples used in Equation 5 was also less than measured in other surface forest soils from the Western Oregon Cascades (McNabb et al., 1986). This suggests that more samples are needed to accurately estimate parameters. How variability of undisturbed soil affects parameters needs additional study.

The advantage of Equation 2 as a nonlinear model of soil compression is that it predicts bulk density at all normal stresses (Bailey et al., 1986; McBride, 1989). This is particularly important when soils are normally consolidated, because small applied stresses result in large increases in bulk density (Figure II.7). Equation 5 also predicted bulk density at all stresses. The advantage of Equation 5 over Equation 2 is that the parameters obtained are an average for the soil. This average includes the variation in undisturbed bulk density. The adjustment for the variation in initial bulk density, however, is only effective when the stress history of the individual samples are similar. When differences in overconsolidation affect the shape of the compression curve of individual samples, the fit of Equation 5 to the data will result in parameters which less accurately predict the bulk density.

Based on the few measurements of shear strength, andic soils generally have a low shear strength (Maeda et al., 1977). This is reasonable, because soil with a low bulk density are expected to have fewer contacts between particles. The shear strength of soil increases in proportion to the number of contacts between particles, regardless of texture and mineralogy (Mitchell, 1976). Previously reported values of shear strength, however, have been of disturbed or remolded samples, or of soils from deeper, less well-aggregated soil layers (Pope and Anderson, 1961; Maeda et al., 1977). These factors affect the strength of the soils. Surface soils generally have a high shear strength (Fountaine and Brown, 1959). Yee and Harr (1977) reported that aggregation contributed to the higher shear strength of forest soils in the Coast Range Mountains of western Oregon.

Several factors apparently contribute to the high shear strength of the fine-textured andic soils in this study. Andic soils contain a combination of noncrystalline, crystalline, and intergrade clay minerals which affect their physical properties (Paeth et a., 1971; Taskey et al., 1978; McNabb, 1979). The noncrystalline minerals often cover the other clay minerals and particles. The shear strength is apparently affected by where, and how strongly, the water is held within the minerals. Most of the soil water is weakly held within the mineral structure and noncontinuous pores of the fine-textured andic soils (Maeda et al., 1977). Only a monolayer of water is tightly held on the irregular mineral surfaces. Therefore, contact between mineral surfaces, when the soil is saturated, is relatively high. This is in contrast to layer silicate minerals where most of the water is retained in continuous pores and thick layers of water that are adsorbed on the surface of clay minerals, which reduces contacts between particles and, consequently, soil strength. During consolidation of the fine-textured andic soils, soil water is presumably displaced from within the mineral structure of the noncrystalline minerals as well as from between soil aggregates. The displacement of water from within these minerals could increase the contacts between particles in fine-textured andic soils more rapidly during compression than in soils containing layer silicate minerals.

The Jory soil was more compressible than the andic soils because the Jory soil was weaker than the andic soils. The compression index, and the increase in the bulk density calculated using Equation 5, was higher for the Jory soil than for the andic soils (Table II.2 and Figure II.7). The shear strength of the Jory soil was also lower than the shear strength of the andic soils (Table II.4).

Because pore water pressures dissipated rapidly, the normal stresses used to compress samples were also a measure of the compressive strength of these soils (Lambe and Whitman, 1979). Therefore, soil strength determined the compressibility of these soils, rather than the bulk density of the soil. Undisturbed and compacted bulk densities are not direct indicators of the susceptibility of soil to compaction, although bulk density is related to the strength of a specific soil.

Andic soils have been considered less susceptible to compaction because of their low, undisturbed and compacted bulk density, and relatively small reduction in porosity when compared to more dense soils compacted with a similar compactive effort (Howard et al., 1981). These data indicate that the compressibility of the andic soils and the more dense Jory soil were similar at low applied stress (Figure II.7). At higher normal stresses, the compressibility of the andic soils was less than for the Jory soil. The Crater Lake soil was less compressible than the other soils because coarse-textured soils are less easily consolidated in static, one-dimensional consolidation tests (Lambe and Whitman, 1979).

Although an increase in bulk density is a measure of the compressibility of these soils, the small differences in compressibility resulting from mechanical stresses may not have a similar effect on the growth of plants. An increase in bulk density increases the resistance of soil to penetration by roots (Taylor and Gardner, 1963; Zisa et al., 1980), but the increase in bulk density also changes several other soil properties which may affect the growth of plants

(Greacen and Sands, 1980; Froehlich and McNabb, 1984). Therefore, the effects on the growth of plants of changes in soil properties resulting from soil compaction must be assessed separately from the effects of mechanical stress on the compression of soil.

CONCLUSIONS

Consolidation of these forest soils was rapid because of their high porosity. Low normal stresses resulted in relatively large increases in bulk density which indicated that the undisturbed soils were normally consolidated. Because of the rapid dissipation of pore water pressure during consolidation and high compression at all normal stresses, compaction of wet soil by machines is expected to cause large increases in bulk density regardless of the duration of loading.

Undisturbed and compacted bulk densities were not a measure of the susceptibility of the soils to compression. The compression index and increase in bulk density calculated using the nonlinear model of soil compression indicated that the bulk density of the Jory soil increased more rapidly for a specific normal stress than did the bulk density of the andic soils. The increase in bulk density of the andic soils was always smaller than the increase in bulk density of the Jory soil, because the Jory soil is weaker than the andic soils. The lower strength of the Jory soil was confirmed by results showing the Jory soil had a significantly lower shear strength than the andic soils.

The nonlinear model of soil compression (Eqn 5) described the average compression curve for each soil. Equation 5 was an effective method of adjusting the compression curve of samples for the variation in bulk density of individual samples. The ability to adjust compression curves for variation in bulk density potentially allows Equation 5 to be expanded to include variables for the effects that soil water has on compression.

The andic soils were less compressible than the more dense Jory soil because of higher soil strength, rather than low bulk density. This apparently resulted because most of the water was retained within the mineral structure and noncontinuous pores, while only a thin layer of water was tightly held on the mineral surfaces of andic soils. As a result, contact between particles, and consequently soil strength, was high. The effect that noncrystalline minerals have on soil strength needs to be confirmed in more carefully controlled soil tests.

Although the differences in the compressibility of saturated soil are small, differences in the compressibility of partly saturated soil are most likely larger. The effect of soil water on compressibility have yet to be directly integrated into a general model of soil compression. Furthermore, compression of soils of different bulk densities may not have a similar effect on other soil properties important to plant growth. More information is needed on how soil compression affects soil properties important to tree growth and the site specific conditions when changes in those properties affect the growth of trees.

Chapter III

COMPRESSION OF SATURATED AND PARTLY SATURATED SOILS

INTRODUCTION

Reduction of crop yields by compaction of agricultural soils has concerned agronomists for thousands of years (Philip, 1974; McKibben, 1971). Compaction is becoming a more serious problem as the size of machines increases, fields are less frequently tilled, and soils are more deeply compacted (Chancellor, 1977; Soane et al., 1981b; Culley and Larson, 1987; Voorhees, 1987). Although compaction of agricultural soils is an obvious problem, compaction of forest soils is a relatively new concern of forest managers (Greacen and Sands, 1980; Froehlich and McNabb, 1984).

In a review of compaction of forest soils (Greacen and Sands, 1980), a survey of the literature between 1970 and 1977 found that compaction reduced the growth of tree species in 92 percent of the 26 studies cited, whereas the average for other crops was only 80 percent. In the Pacific Northwest, compaction from a single harvest can reduce the growth of trees for decades; Wert and Thomas (1981) found that the growth of a stand of Pseudotsuga menziesii (Mirb.) Franco in western Oregon was reduced by 11.8 percent, 31 years after harvesting. After 31 years, the bulk densities of soil in skid roads were significantly higher than those of soil away from the skid roads. Forest soils remaining compacted four or more decades after harvesting have been found at several locations in the Pacific Northwest (Froehlich et al., 1985).

The growth of trees is reduced on compacted soil in Pacific Northwest forests in spite of the fact the bulk densities of the soils are often low before and after compaction (Froehlich and McNabb, 1984). Low bulk density is a common characteristic of forest soils in the region, particularly in western Oregon, where many of the soils are andic soils containing noncrystalline minerals (Huddleston, 1979). Andic soils have low compacted bulk densities, as well as several other unusual soil physical properties (Maeda et al., 1977; Howard, et al., 1980; Kinloch, 1987). Understanding how the properties of andic soils affect their compression is an important first step in assessing the potential occurrence of soil compression, with possible adverse consequences in plant growth (Voorhees, 1987).

Several methods are used to study the deformation process (Freitag, 1971; Chancellor, 1977; Soane et al., 1981a). Predicting compacted bulk density with impact tests of disturbed soil (example, Proctor test) has been partially successful in some agronomic situations. The method is most suitable when soils are wetter than the optimum water content for maximum compaction (Raghavan et al., 1976). The method has also been used to compare compactibilities of forest soils (Howard et al., 1980). The Proctor test and similar tests that use less force to compact the soil, however, have overestimated the bulk density of undisturbed forest soils compacted by machines (Froehlich et al., 1980).

Larson et al. (1980), used the compression index of partly saturated, disturbed soils obtained in one-dimensional consolidation tests, to describe the compression of soils. The compression index is the slope of the linear part of the compression curve describing the relationship between bulk density and the logarithm of normal stress. The compression index was related to clay content, type of clay mineral, and degree of saturation. The compression index of about 20 percent of the soils changed as matric pressures increased from -0.1 to 0 MPa. A decrease in matric pressure affected the shape of the compression curve at lower normal stresses. As a result, soils became more overconsolidated as the matric pressure decreased. Measured compression indexes have also differed from those predicted with the equations of Larson et al. (1980) when soils were well-aggregated or when soil management practices affected their stress history (Gupta and Larson, 1982; Culley and Larson, 1987). These factors make predicting compression of partly saturated soil for a specific normal stress difficult.

Bailey et al. (1986), proposed a nonlinear model that described the entire compression curve:

$$\ln(\rho_c) = \ln(\rho_o) - (A + B\cdot\sigma)\cdot(1 - \mathrm{EXP}(- C\cdot\sigma)), \qquad [1]$$

where ρ_c is bulk density following compression (Mg/m^3), ρ_o is the bulk density of soil at zero stress and is estimated by regression (Mg/m^3), σ is the normal stress applied to compress the sample (MPa), and A, B, and C are parameters describing the shape of the compression curve. The primary advantage of the nonlinear model is that it predicts bulk density at the boundary conditions of low and high normal stresses.

An example of a compression curve of bulk density predicted with Equation 1 is shown in Figure III.1. The effect that changes in parameters have on compression curves are shown by multiplying each parameter by two while the other two parameters remain at their original value. The normal stress is plotted both normal scale (Figure III.1A) and logarithmic scale (Figure III.1B). The latter scale is used when the compression index and overconsolidation stress of saturated soil are calculated (Terzaghi and Peck, 1967). Increases in bulk density at lower normal stresses are more obvious when plotted on the logarithmic scale. Multiplying the A parameter by two caused large increases in bulk density at all normal stresses compared to changes in the other parameters. Multiplying the B parameter by two had a negligible effect on bulk density at normal stresses less than 0.1 MPa but caused a large increase in bulk density at higher normal stresses. Multiplying the C parameter by two did not affect bulk density at normal stresses greater than 1 MPa.

Equation 1 has also been used to analyze data obtained by compressing numerous undisturbed samples, each consolidated to a specific normal stress (Chapter II). Variation in the bulk density of samples reduced the fit of Equation 1 to the data. Two variables were added to Equation 1, which adjusted the data for some of the effects that differences in undisturbed bulk density had on soil compression. The added variables were δ_i and δ_c so that the revised equation was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + D \cdot \delta_c + B \cdot \sigma) \cdot (1 - EXP(-C \cdot \sigma)), \qquad [2]$$

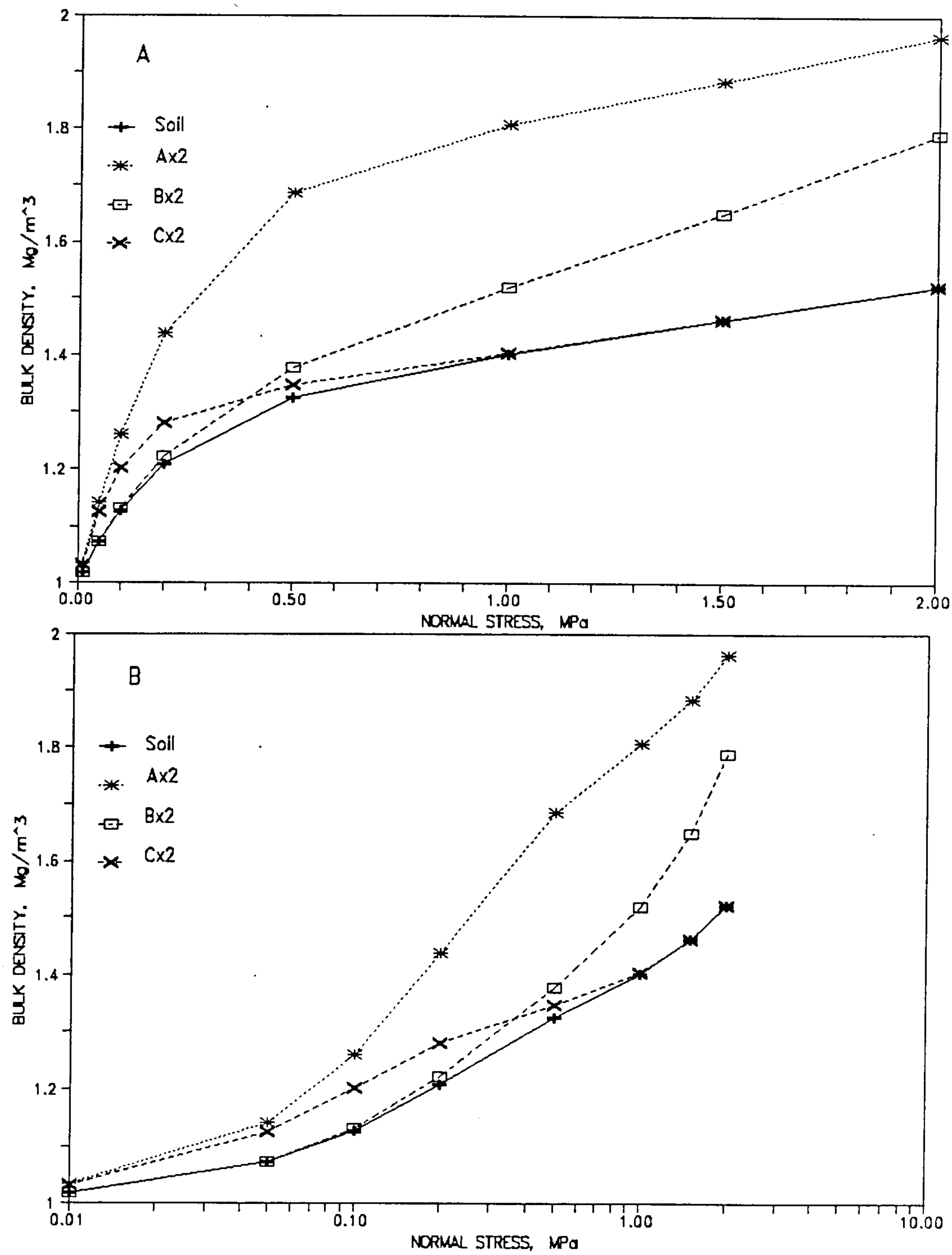


Figure III.1. Relationship between bulk density of a sample of western Oregon forest soil, predicted with Equation 1, and normal stress (normal and logarithmic scale). Effects of multiplying each parameter by two are also shown. Parameters for the sample were $\rho_o = 1.009$, $A = -0.254$, $B = -0.081$, and $C = 5.800$.

where δ_i normalizes ρ_o for differences in initial bulk density, ρ_i, and is:

$$\delta_i = \rho_i \; / \; \frac{\sum^{n} \rho_i}{n}. \qquad [3]$$

The δ_c adjusts the compression curve for differences in compressed bulk density:

$$\delta_c = (\delta_i - 1) \cdot \rho_o, \qquad [4]$$

and D is the parameter of δ_c.

The D parameter in Equation 2 was significant for all soils evaluated in this study (Chapter II). Equation 2 also provided a more accurate estimate of the value of ρ_o than did Equation 1. The value of ρ_o is approximately equal to the average undisturbed bulk density of these soils. The advantages of Equation 2 were that the parameters described the average compression curve for each soil and demonstrated that data for samples with different bulk densities could be combined. Thus, Equation 2 has the potential to be expanded to include the effects that differences in water content have on soil compression.

The one-dimensional consolidation test was used to compress samples. Four soils from western Oregon forests were chosen to represent specific types of soil materials (Chapter II). Three of the soils were andic soils or had andic properties. Andic soils were emphasized because of their low undisturbed and compressed bulk density. The compressibility of the andic soils was contrasted with a fourth soil which contained layer silicate minerals. Equation 2 was used to calculate compression parameters of undisturbed samples of each soil. Parameters were compared for saturated samples and for samples collected and tested at several in situ soil water contents. A general model of soil compression, which included soil water, was developed.

MATERIALS AND METHODS

Site Selection

The criteria and procedures used for selecting the four western Oregon forest soils were described in Chapter II. The soils represent different types of materials: 1) a coarse-textured volcanic ash; 2) a low plasticity soil with a high organic matter content >15 percent; 3) a medium- to fine-textured soil with low plasticity; and 4) a fine-textured soil with moderate to high plasticity. Table III.1 gives the taxonomy and selected physical properties of the soils.

All sites had an overstory of mature conifer trees and an understory vegetation of shrubs. Samples were only collected from areas which had not been disturbed by previous partial harvests.

Soil Collection

Undisturbed core samples were collected over a range of 5 or 6 soil water contents by collecting samples during a summer season. During this time, soil water content ranged between that at field capacity to that at field dry water content. Field dry water content was the lowest in situ water content that occurred during the year. Although summer precipitation in western Oregon is low (Franklin and Dyrness, 1973), plastic sheeting was suspended above the understory vegetation at three locations at each site to prevent recharging of the surface horizons from rainfall and fog drip at coastal locations. This insured that samples were obtained with a wide range in natural soil water contents.

Table III.1. Classification and physical properties of four western Oregon forest soils. Data are for a composite sample of each soil from the 7- to 12 cm depth.

Soil	Classification	Particle Size Distribution Sand	Silt	Clay	Texture	Unified Class.	Atterberg Limits θ_l	I_p†	Organic Matter	Bulk Density
		- - - - kg/kg - - - -					- - - kg/kg - - -			Mg/m^3
Crater Lake	medial, frigid, Typic Vitrandept	0.635	0.308	0.057	SL	SM	NP††	--	0.026	0.746
Hemcross	medial, mesic Andic Haplumbrept	0.153	0.493	0.354	SiCL	MH	0.911	0.051	0.180	0.629
Tolovanna	medial, mesic Typic Dystrandept	0.350	0.392	0.258	L	MH-OH	1.170	0.090	0.254	0.529
Jory	clayey, mixed, mesic Xeric Haplohumult	0.348	0.355	0.297	CL	MH	0.555	0.187	0.060	0.987

† Atterberg limits: θ_l is the liquid limit; and I_p is the plastic index.
†† Nonplastic

A water release curve was constructed for each soil using undisturbed soil samples (Klute, 1986). At each site, four samples were collected at a depth of 10 cm in plastic rings with a height of 1 cm. The soil water content of each sample was measured at matric pressures of -0.01, -0.08, -0.2, and -1.5 MPa. Water release curves were used to estimate the gravimetric soil water content at matric pressures of -0.015, -0.03, -0.07, -0.2, -0.5, and -1.5 MPa. Undisturbed core samples were collected for compression tests at soil water contents near these estimates.

Periodic determination of gravimetric water content, complimented by measurement of matric pressures with a hand-held tensiometer, was used to determine sampling dates. Variability of in situ soil water content, the rate of soil water depletion, and remoteness of sites prevented the collection of all samples when soil water content was at the desired matric pressures.

Undisturbed core samples for compression tests were collected at three locations across an area of 100 m^2. The litter, duff, and surface soil were removed over an area of about 2 m^2 to a depth of 7 cm. A narrow trench was excavated to a depth of 50 cm along one side of the area to provide additional access. Samples were removed from the 7- to 12 cm soil depth. Samples were collected in PVC rings with an inside diameter of 7.45 cm and a height of 3.5 cm. The PVC ring was placed in a metal ring with a beveled cutting edge (Chapter II). This assembly was placed on the surface of the exposed soil and the soil was cut away from around the cutting edge by hand, while hand pressure on top of the ring gently pressed it into the soil (USDI, Bureau of Reclamation, 1974; Chapter II). Samples were discarded if rock fragments or roots greater than 2 mm in diameter were encountered; smaller roots were clipped. Rings were covered with polycarbonate discs, parafilm, plastic wrap, and tape for transportation and storage.

Compression Test

One-dimensional consolidation tests were performed on samples in the lever-action, consolidation frame described in Chapter II. Wet weight and height of each sample were recorded prior to testing. Samples were compressed for 512 min. Initial compression of samples was rapid, generally lasting only a few minutes (Chapter II). Measurement of compressed bulk density was delayed, however, until the secondary compression that followed had slowed. Chapter II contains additional information about the machine and the recording of data. Data were used to calculate the initial and the compressed bulk density, and the soil water content. Degree of saturation of each core sample was calculated from the water content and initial bulk density of each core, and from the particle density of a composite sample of each soil.

Core samples were compressed at one of seven normal stresses between 0.033 and 1.96 MPa. The loading increment was doubled with each step, beginning with the smallest normal stress. Only a few tests, mostly with saturated samples, were performed at a normal stress of 1.96 MPa. Two samples of the Crater Lake, Hemcross, and Tolovanna soils were compressed at all stresses and soil water contents. Tests with the Jory soil were performed in triplicate.

Previously collected data on the consolidation of these soils when saturated were included as an additional soil water content (Chapter II). Tests of saturated soils had also been performed in triplicate.

In addition, the data set also included the initial bulk density of each sample measured at a normal stress of 0.002 MPa. The porous stone and loadcap resting on the sample provided this normal stress. This data improved the accuracy of estimating the value of ρ_o and the shape of the compression curve at lower normal stresses (Chapter II).

Statistical Analyses

The Marquardt nonlinear curve fitting technique was used to estimate the parameters in all models of soil compression (SAS, 1986). Differences between predicted and measured bulk density, and parameters and soil water content were analyzed by linear regression. Multiple linear regression was used to predict compressed bulk density from the relationship between the logarithm of normal stress and variation in bulk density of samples when Equation 2 failed, or an A, B, or C parameter was not statistically significant.

RESULTS

Soil Properties

The mean soil water content and bulk density, and number of samples tested for each collection period are listed in Table III.2. Data are arranged by decreasing soil water content, θ_w. The first row of data is the saturated water content of each soil. This water content was calculated from the bulk density of individual samples and particle density because large macropores prevented the measurement of saturated water content (Table III.1). The remaining water contents were the soil water content at the time of collection.

The large differences in soil water content between saturated water content and the water content in the second row of data, which were near field capacity, are due to the high percentage of macropores in the soils. Approximately 40 percent of the pore space of the Jory, Hemcross, and Tolovanna soils and 65 percent of the pore space of the Crater Lake soil are filled with air at field capacity (Figure III.2). Having nearly half of the pore space in macropores is a physical property common to forest soils in western Oregon (Dyrness, 1969).

At least four significantly different soil water contents were collected for each soil. The fewest significant differences occurred in the Tolovanna soil. Variation in soil water content of individual samples of the Tolovanna soil, however, was high as indicated by the fact that collections with soil water contents of 0.658 and 0.590 kg/kg were not significantly different.

The initial bulk density of the Hemcross and Jory samples

Table III.2. Average soil water content and initial bulk density, and number of samples collected at water contents ranging from field capacity to field dry. Soil water contents in the first row are the saturated water content of samples collected near field capacity and saturated prior to testing. Soil water contents in the second row are the collected and tested water content of samples collected near field capacity.

Jory			Crater Lake			Hemcross			Tolovanna		
θ_w	ρ_i	n	θ_w	ρ_i	n	θ_w	ρ_i	n	θ_w	ρ_i	n
kg/kg	Mg/m^3		kg/kg	Mg/m^3		kg/kg	Mg/m^3		kg/kg	Mg/m^3	
0.594a	0.992bc†	23	1.016a	0.707d	21	1.301a	0.577e	21	1.638a	0.482d	18
0.375b	0.920d	18	0.324b	0.784a	12	0.745b	0.559e	12	0.769b	0.582a	12
0.322c	0.953cd	23	0.172c	0.773ab	18	0.612c	0.623d	18	0.755b	0.488c	12
0.285d	0.989bc	18	0.168c	0.746c	12	0.583c	0.631cd	11	0.658c	0.519b	12
0.169e	1.034a	18	0.098d	0.723d	12	0.397d	0.653bc	20	0.590c	0.534b	13
0.166e	1.015ab	21	0.077e	0.757bc	11	0.388d	0.666b	16	0.471d	0.597a	12
0.129f	1.009ab	19				0.338e	0.704a	12			

† Data in columns followed by the same letter are not significantly different ($p<0.05$).

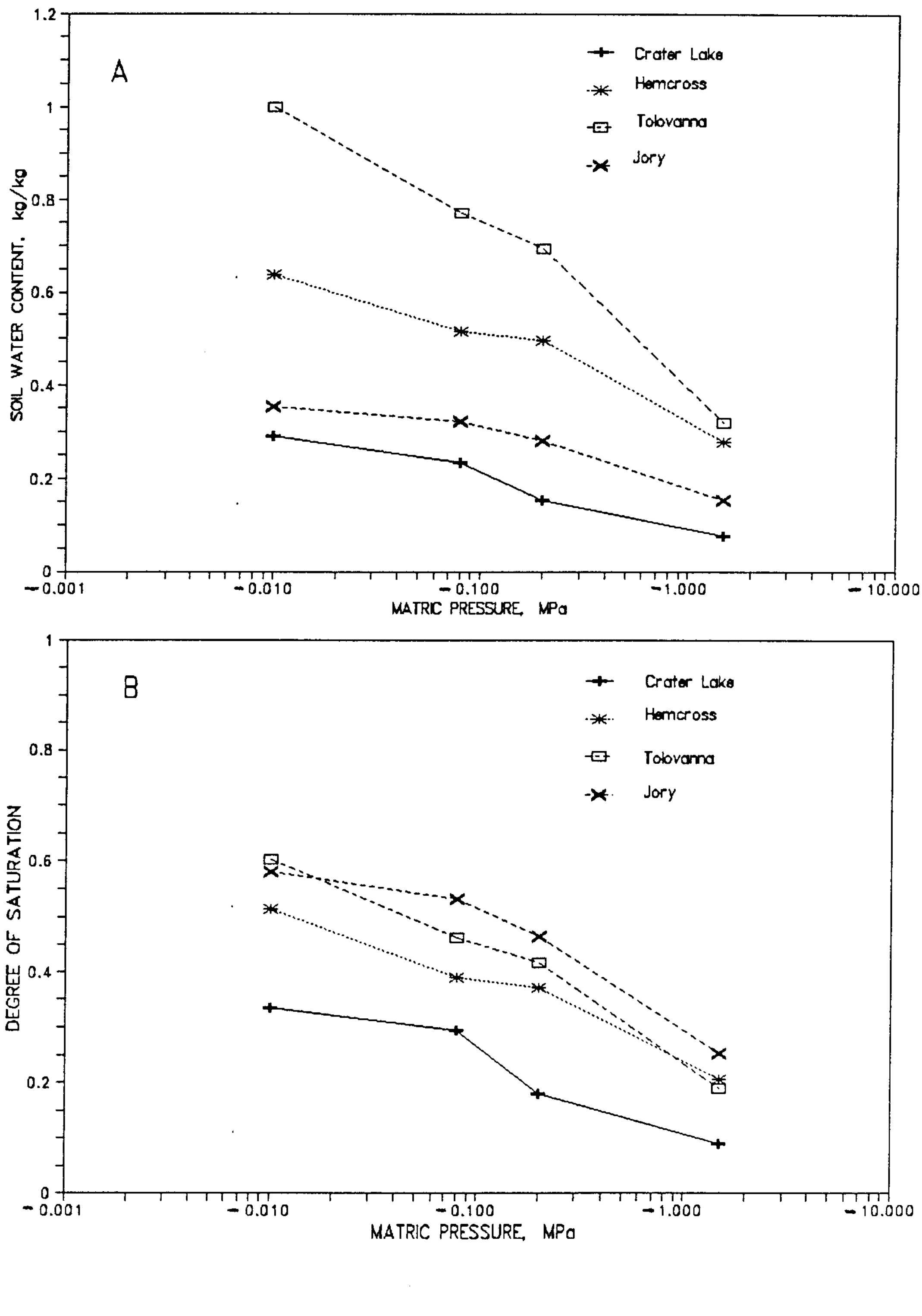


Figure III.2. Water release curves from tests using undisturbed core samples from the 9-11 cm layer as a function of soil water content (A) and degree of saturation (B).

increased significantly as soil water content decreased as follows:

Hemcross soil: $\rho_i = 0.785 - 0.276\theta_w$ ($R^2 = 0.82^{**}$, d.f. =6)

Jory soil: $\rho_i = 1.062 - 0.28\theta_w$ ($R^2 = 0.62^{*}$, d.f. =6)

These analyses included the initial bulk density of saturated soil, but the soil water content was assumed to equal the highest, partly saturated, water content. Samples for saturated tests were collected near this soil water content and saturated in the laboratory. Desiccation may have increased the initial bulk density of the Jory soil because it contained layer silicate minerals (Terzaghi and Peck, 1967).

The variation in initial bulk density among collections of all soils was high because of the sampling procedure (Table III.2). During each collection, several samples were collected over an area of about 2 m^2 at 3 locations within the 100 m^2 site. Each collection was at a new location to avoid the previous disturbance.

The explanation for the increased bulk density of the Hemcross samples with decreasing water content is more complex. Desiccation does cause volume change in soils containing noncrystalline minerals (Maeda et al., 1977), but a similar increase in the initial bulk density of the Tolovanna soil was not measured (Table III.2). Collection of samples was also hindered by disturbance of the site by mountain beaver (Aplodontia rufa (Raf.)). The disturbance made it increasingly difficult to obtain samples which did not include some soil from the B horizon.

From 11 to 23 individual compression tests were performed at each soil water content of a soil (Table III.2). Number of samples differed because tests with the Jory soil were performed in triplicate, whereas one less soil water content of the Crater Lake and Tolovanna soils was tested.

The fine-textured Hemcross and Tolovanna soils had higher water contents at each matric pressure than the Jory and Crater Lake soils (Figure III.2A). High soil water retention is characteristic of fine-textured andic soils of noncrystalline mineralogy (Maeda et al., 1977). Retention of water by the Crater Lake soil was similar to the Jory soil although the former had a coarser texture (Table III.1). Retention of water by the Crater Lake soil, however, was similar to other soils containing volcanic ash (Geist and Strickler, 1978). Expression of soil water content as a degree of saturation, θ_s, rather than reporting soil water content on a mass basis, eliminated the effects of bulk density on these reports (Figure III.2B).

Effects of Water Content on Soil Compression

Compression of samples resulted in a range of initial bulk densities at each normal stress (Figures III.3A, III.3B, III.3C, and III.3D). Data were grouped into classes of degrees of saturation based on the degree of saturation of individual samples rather than by collected soil water content. This was done because some soil water contents were not significantly different (Table III.2).

At a constant normal stress, samples at higher degrees of saturation were expected to have a higher bulk density than samples at lower degrees of saturation (Larson et al., 1980). Samples with the

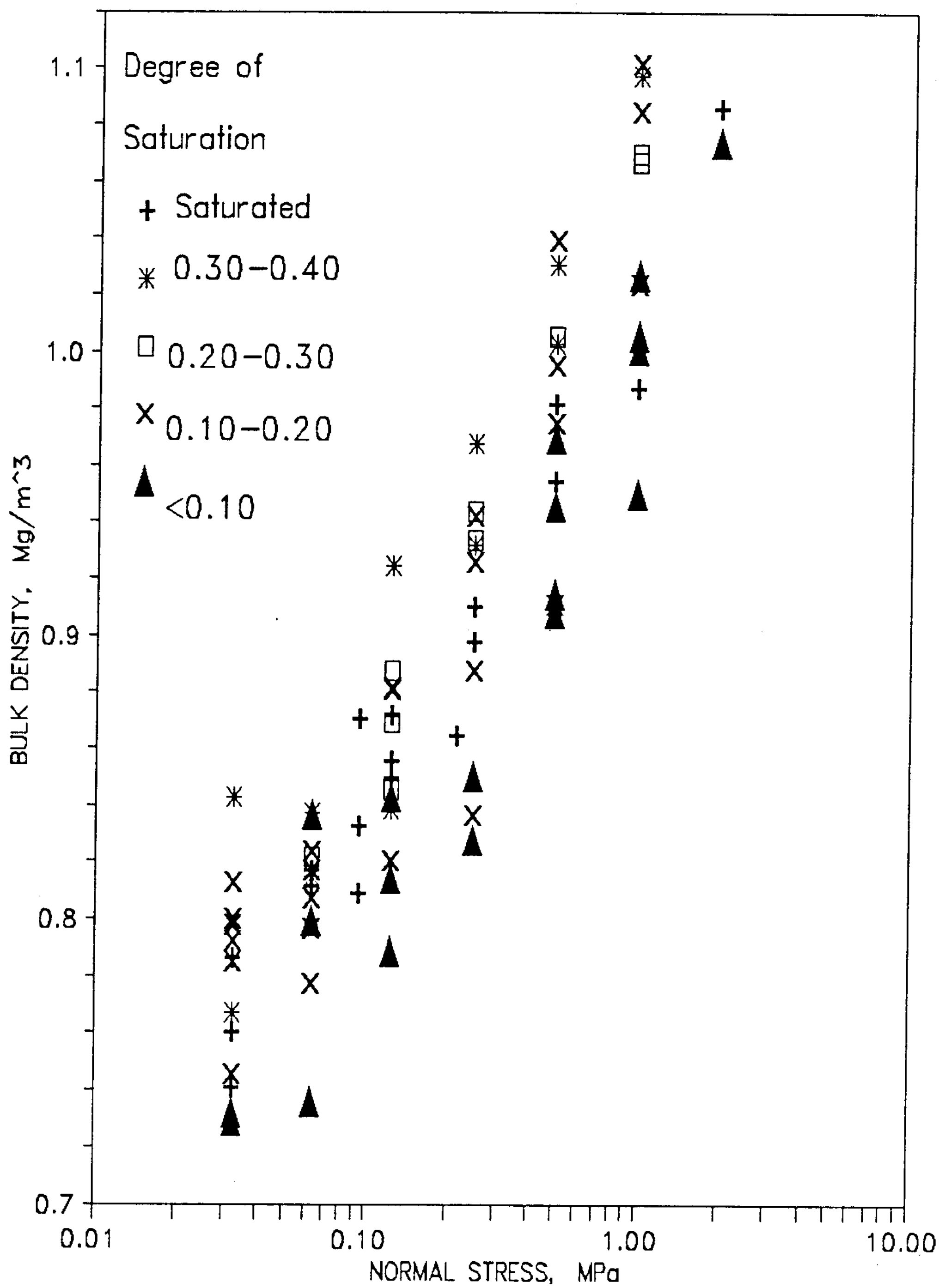


Figure III.3A. Bulk density of undisturbed core samples (7- to 12 cm depth) of the Crater Lake soil from the southern Oregon Cascade Mountains compressed in one-dimensional consolidation tests. Samples were compressed at soil water contents from saturated to dry (n=86).

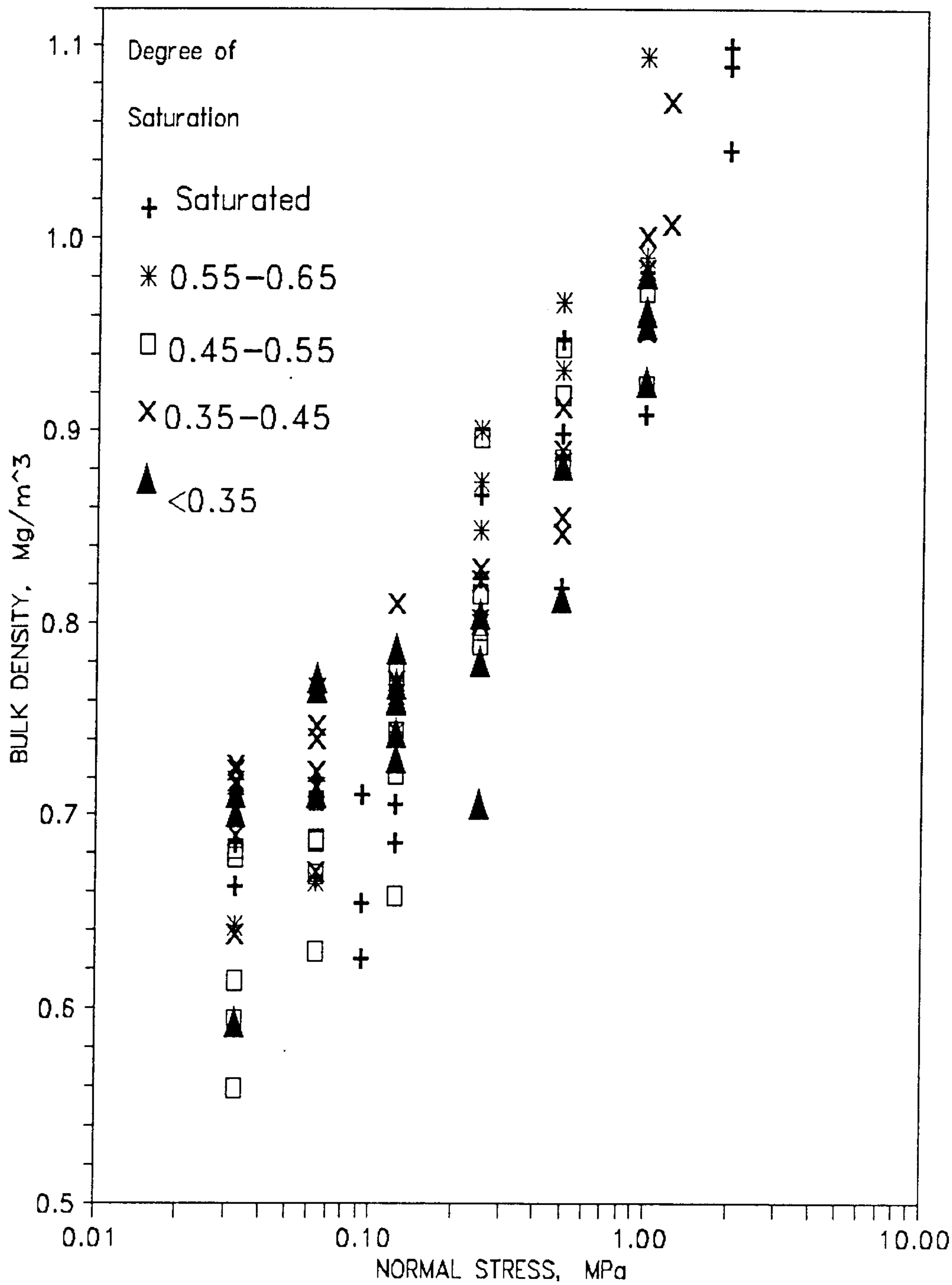


Figure III.3B. Bulk density of undisturbed core samples (7- to 12 cm depth) of the Hemcross soil from the northern Oregon Coast Range Mountains compressed in one-dimensional consolidation tests (n=109).

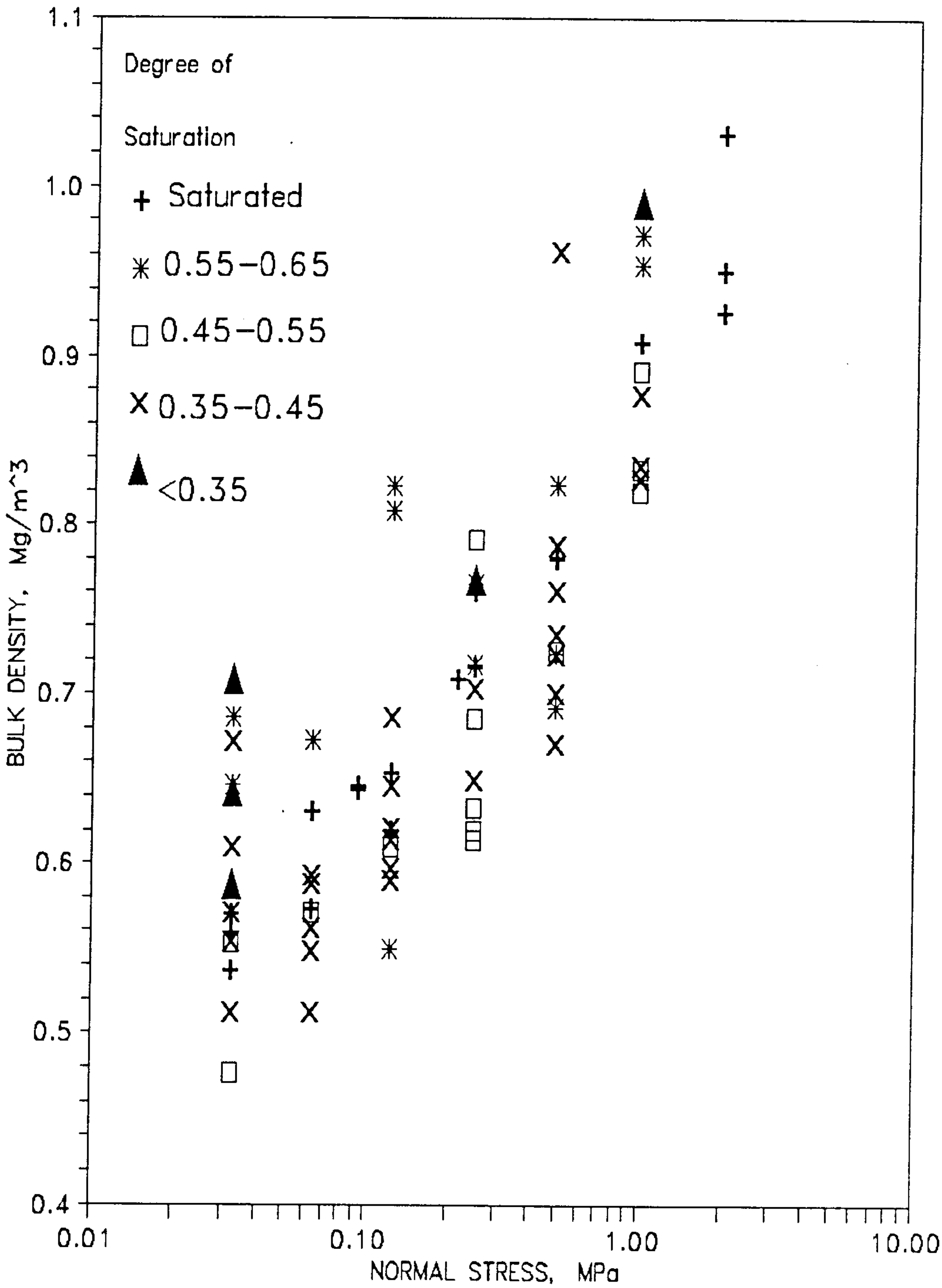


Figure III.3C. Bulk density of undisturbed core samples (7- to 12 cm depth) of the Tolovanna soil from the central Oregon Coast compressed in one-dimensional consolidation tests (n=79).

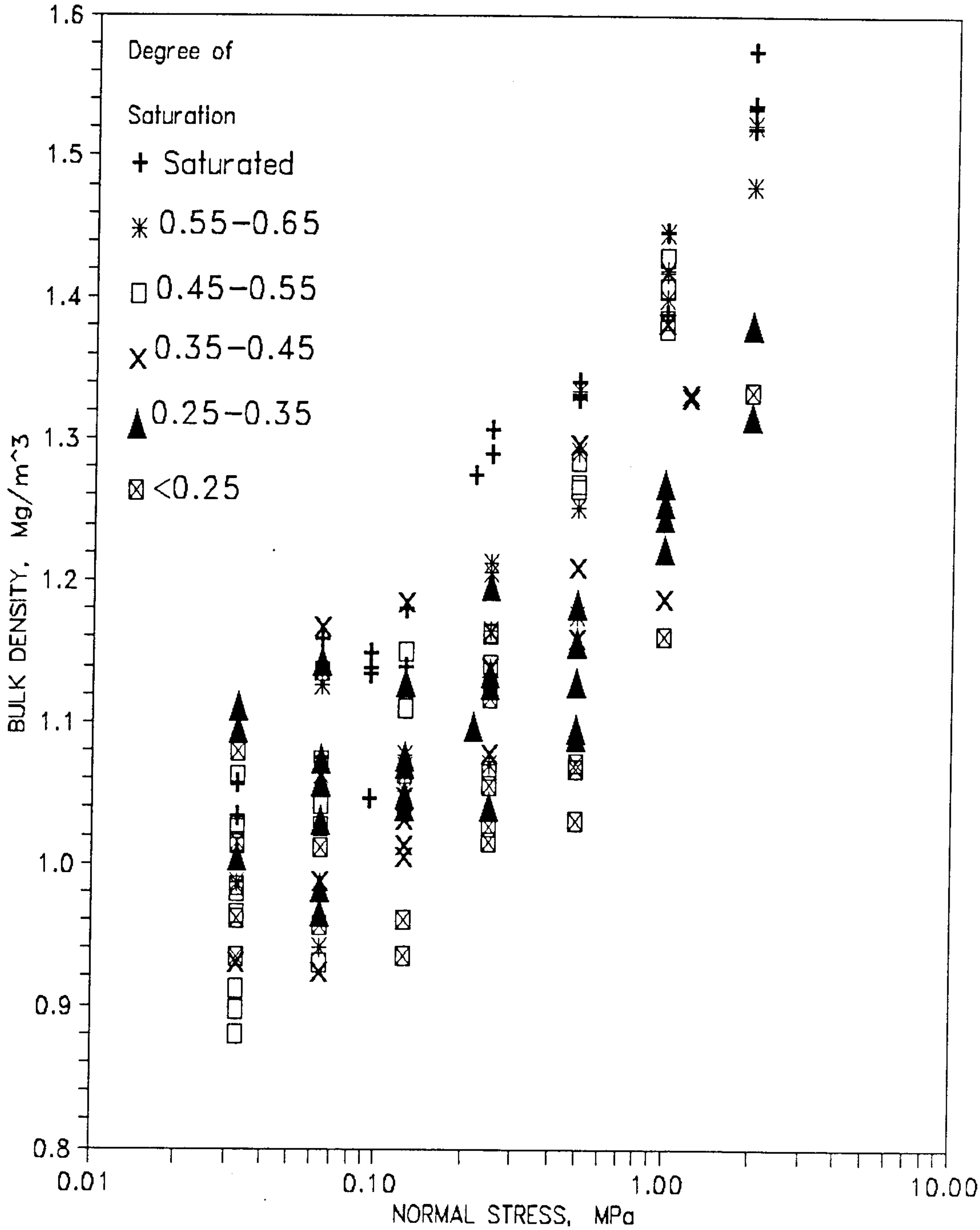


Figure III.3D. Bulk density of undisturbed core samples (7- to 12 cm depth) of the Jory soil from the southern Willamette Valley of western Oregon compressed in one-dimensional consolidation tests (n=140).

highest and lowest degrees of saturation, however, did not consistently have the highest and lowest measured bulk densities at each normal stress (Figures III.3A, III.3B, and III.3C). Only for the Jory soil did the saturated samples generally have the higher bulk densities, while samples with the lowest degrees of saturation had the lowest bulk densities for each normal stress (Figure III.3D). Variation in the initial bulk density of individual samples obscured much of the effect that changing soil water content had on compression of these soils. Samples with a high initial bulk density were more likely to have a high bulk density following compression, while samples with a low initial bulk density were more likely to have a low bulk density following compression. Part of the variation in bulk density is attributed to differences in the soil fabric and structure among samples, which was not destroyed by compression (Chapter II).

The parameters for Equation 2, which accounts for the variation in the bulk density of individual samples, are listed in Table III.3 for each soil. Parameters were not listed when the nonlinear regression failed to converge. This occurred for the Jory soil when the water content was about 0.17 kg/kg. When failure to converge occurred with a set of data, the nonlinear regression procedure was unable to estimate a unique set of parameters which would produce the lowest mean square error (Bates and Watts, 1988). Four other sets of these data had an A, B, or C parameter that was not statistically significant. These sets are also identified in Table III.3. Sets of data which failed to converge or had a nonsignificant A, B, or C parameter generally had

Table III.3. Parameters and mean square errors (MSE) for the nonlinear regression model (Eqn 2) of compression of soil collected and tested at different soil water contents. The highest soil water content of each soil was at saturation obtained by saturating samples that were collected near field capacity.

Soil/ Water	Parameters					MSE
	ρ_o	A	B	C	D	
kg/kg						10^{-3}
Crater Lake						
1.016	0.703	-0.245	-0.112	12.210	1.225	0.504
0.324	0.780	-0.184	-0.199	7.410	1.155	0.405
0.172	0.769	-0.192	-0.158	7.302	0.917	0.105
0.168	0.746	-0.472	-0.129	2.375	0.371	0.095
0.098	0.721	-0.233	-0.106	5.658	0.416	0.107
0.077	0.756	-0.172	-0.090	4.373	0.880	0.133
Hemcross						
1.301	0.576	-0.371	-0.132	8.113	1.499	1.304
0.745	0.555	-0.443	-0.100	8.088	-0.432†	0.173
0.612	0.620	-0.288	-0.228	7.501	0.821	0.162
0.583	0.628	-0.243	-0.205	8.157	1.037	0.040
0.397	0.650	-0.134	-0.300	16.224	0.761	0.185
0.388	0.665	-0.149	-0.226	9.339	1.246	0.057
0.338	0.703	-0.271	-0.050†	3.241	1.331	0.010
Tolovanna						
1.638	0.478	-0.447	-0.135	10.539	1.596	0.547
0.769	0.577	-0.435	-0.036†	5.710	0.029†	0.275
0.755	0.485	-0.154	-0.433	14.867	0.859	0.213
0.658	0.520	-0.485†	-0.041†	2.579†	1.559	0.239
0.590	0.531	-0.159	-0.340	12.765	0.313	0.319
0.471	0.594	-0.147	-0.372	15.056	0.208†	0.429
Jory						
0.594	0.988	-0.292	-0.079	6.091	0.980	0.179
0.375	0.921	-0.301	-0.111	4.057	0.796	0.259
0.322	0.945	-0.400	-0.030†	2.431	1.053	0.475
0.285	0.986	-0.241	-0.124	4.021	1.004	0.074
0.169	failed to converge					
0.166	failed to converge					
0.129	1.079	-0.108†	-0.087	2.187†	0.337†	0.147

† Parameter was not signficantly different from zero ($p > 0.05$).

a high correlation among all A, B, and C parameters ($r > \pm 0.99$). A high correlation indicated that the model contained more parameters than needed to describe the data (Bates and Watts, 1988).

When an A, B, or C parameter was not statistically significant, or the nonlinear regression failed to converge, a logarithmic transformation of normal stress was used to predict bulk density with a linear regression model. This model was:

$$\rho_c = a + b \cdot \ln(\sigma) + d \cdot \delta_r. \qquad [5]$$

Equation 5 is similar to the equation for calculating the compression of soil (Terzaghi and Peck, 1967; Larson et al., 1980), except that it includes the δ_r variable to adjust the compression curve for the variation in initial bulk density of individual samples, ρ_i:

$$\delta_r = \rho_i - \frac{\sum^{n} \rho_i}{n}. \qquad [6]$$

The analysis of the six sets of data using Equation 5 suggested two causes for the failure of Equation 2 to predict bulk density (Table III.4). The least common cause was an overparameterized model in which the effect of variation in initial bulk density on compression was small. The data for the Hemcross soil and the drier Tolovanna soil are in this category. The variation in initial bulk density was not significant and the logarithmic transformation of normal stresses accounted for most of the variation in the prediction of bulk density ($R^2 > 0.92$) (Table III.4). The successful transformation of these data indicated that the data were intrinsically linear (Draper and Smith, 1966).

Table III.4. Multiple linear regression parameters and regression coefficients for Equation 3. Equation 3 was used for soil water contents and soils when Equation 2 failed to converge, or contained A, B, or C parameters which were not significantly different from zero.

Soil/ Water Content	n	Regression Parameters a	b	d	R^2
kg/kg					
Hemcross					
0.338	12	0.939	0.071		0.92
Tolovanna					
0.769	12	0.899	0.078		0.63
		0.870	0.098	1.230	0.98
0.658	12	0.793	0.077		0.97
Jory					
0.169	18	1.050		0.902	0.48
		1.136	0.044	0.862	0.84
0.166	21	1.234	0.070		0.70
		1.222	0.075	0.665	0.81
0.129	19	1.202	0.071		0.57
		1.160	0.058	0.955	0.87

The second cause for the failure of Equation 2 was excessive variation in initial bulk density. This was inferred from the increase in the regression coefficient when the δ_r variable was included in a stepwise regression of Equation 5. The increase in the regression coefficient from including the δ_r variable was 0.11 to 0.48 for the remaining four sets of data (Table III.4). For the Jory soil at a soil water content of 0.169 kg/kg, δ_r variable accounted for more of the variation in bulk density following compression than did normal stress. The regression coefficients of the Jory soil remained lower than those of other soils when the δ_r variable was included in Equation 5. The lower regression coefficients suggested that the logarithmic transformation of normal stress less accurately described the variation in these data than it did for the other data. Therefore, it is concluded that compression of the Jory soil would have been best described by a nonlinear regression model if variation in initial bulk density had been less.

The regression coefficients obtained using Equation 2 were seldom significantly related to soil water content of any soil (Table III.5). Tests of saturated soil were included in these linear regressions because the samples were saturated during the test. The C parameter of the Crater Lake and Jory soils was the only parameter that was significantly related to soil water content. The C parameter of the Jory soil was not significantly related to water content, however, if the value of the C parameter at the lowest water content was excluded. This C parameter was not significantly different from zero (Table III.3).

Table III.5 Regression coefficients between parameters estimated for Equation 2 (Table III.3) and soil water content of each soil.

Soil	n	Regression Coefficient				
		ρ_o	A	B	C	D
Crater Lake	6	0.33	0	0	0.75*	0.41
Hemcross	7	0.61*	0.44	0.07	0.01	0
Tolovanna	6	0.41	0.12	0.12	0.12	0.34
Jory	4	0.11	0.01	0.01	0.72	0.01

The value f ρ_o of the Hemcross soil was also significantly related to soil water content. This was expected because of the significant relationship between initial bulk density and soil water content that was discussed earlier. Although previous studies using Equation 1 have suggested that compression parameters are affected by soil water content (Bailey et al., 1986; McBride, 1989), these analyses failed to confirm such relationships.

Equation 2 was also used to predict bulk density of each soil when data for all soil water contents were combined (Table III.6). The mean square errors of these nonlinear regressions were higher than those for the model using data for saturated soil (Table III.3). The larger error is attributed to the effects that soil water content had on soil compression. The Jory soil had the largest increase in mean square error because compression was affected most by changing soil water content (Figure III.3D). Thus, compression of these soils was affected by differences in soil water content but the differences did not result in parameters which were significantly related to soil water content.

Development of a Nonlinear Model of Soil Compression

The consistently larger mean square error that resulted from the prediction of bulk density with Equation 2 for all the data (Table III.6) indicated that the compression of these soils was affected by soil water content. Therefore, a general equation was formulated to account for

Table III.6 Parameters and mean square errors (MSE) of the nonlinear regression model of compression using Equation 2. The data included all tests of saturated and partly saturated samples of each soil.

Soil	Parameters					MSE
	ρ_o	A	B	C	D	
	Mg/m3					10^{-3}
Crater Lake	0.740	-0.197	-0.126	12.576	0.533	0.817
Hemcross	0.625	-0.250	-0.189	9.893	1.311	0.806
Tolovanna	0.523	-0.235	-0.260	17.706	0.788	2.267
Jory	0.981	-0.121	-0.161	10.851	0.858	1.828

changing soil water content by adding three new variables to Equation 2, as follows:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_l) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m + F \cdot \sigma \cdot \theta\ ) \cdot$$

$$(1 - EXP(-C \cdot \sigma + G \cdot \theta_m)). \qquad [7]$$

An expression of soil water was added for each parameter in the original equation (Eqn 1). The expression added for A was $E \cdot \theta_m$, the expression added for B was $F \cdot \theta_m \cdot \sigma$, and the expression added for C was $G \cdot \theta_m$. Soil water content was expressed as one minus the degree of saturation:

$$\theta_m\ (\theta_m = 1 - \theta_s). \qquad [8]$$

When soil water content is expressed as one minus the degree of saturation, Equation 7 simplifies to Equation 2 when the soil is saturated. This is an important boundary condition for a general model of soil compression.

Equation 7 included more variables than were required to describe the effect that soil water had on soil compression. Equation 7 was modified in several steps until the most accurate model of soil compression was obtained. The steps were to remove variables for soil water which did not improve the prediction of bulk density, add an exponent to remaining variables for soil water, and remove additional variables for soil water which were not significant. The criteria for selecting the final model was based on a reduction of the mean square error, uniform distribution of the residuals for the regression, and a determination that a plot of the fitted model accurately described the compression of the soil (Bates and Watts, 1988). The model with the

m

lowest mean square error was accepted if bulk densities calculated for different normal stresses and degrees of saturation resulted in nearly parallel compression curves at higher normal stresses (Burland and Jennings, 1962; Larson et al., 1980). The following discussion includes the intermediate models of soil compression and the parameters for these models when the model converged. Equation 13 was the model of soil compression which best fit these data.

The $G \cdot \theta_m$ expression was the first expression deleted from Equation 7. The nonlinear regression of the andic soils failed to converge and the G and B parameters for the Jory soil were not significantly different from zero (Table III.7). Parameters for models which failed to converge are not listed because they are not a unique set of parameters (Draper and Smith, 1966).

Decreasing soil water content changes the shape of the compression curve at lower normal stresses (Terzaghi and Peck, 1967; Larson et al., 1980). As a result, soil water was considered to have an important effect on the value of the C parameter of partly saturated soil (Figure III.1). The first modification of Equation 7 was to include the θ_m variable directly in the expression with the C parameter as follows:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m + F \cdot \sigma \cdot \theta_m) \cdot (1 - \mathrm{EXP}(-C \cdot \sigma \cdot (1 - \theta_m))). \qquad [9]$$

By subtracting the θ_m variable from one, the θ_m variable is deleted from the expression containing the C parameter when the soil is saturated. Parameters in Equation 9 were significant for all soils

Table III.7. Parameters and mean square errors (MSE) For Equations 7, 9, and 10. Data included all tests of saturated and partly saturated samples of each soil.

	Parameters								
Soil	ρ_o	A	B	C	D	E	F	G	MSE(10^{-3})
<u>Equation 7†</u>									
Jory	0.985	-0.435	-0.001††	3.525	1.057	0.403	-0.131	0.033††	0.522
<u>Equation 9</u>									
Crater Lake	0.744	-0.164	-0.138	19.282	0.993	-0.215	0.134		0.505
Hemcross	0.628	-0.286	-0.141	9.470	1.483	-0.168	0.119		0.603
Tolovanna	0.529	-0.410	-0.125	11.781	0.444	0.205	-0.162		0.781
Jory	0.984	-0.330	-0.061	5.438	1.234	0.520	0.041		0.357
<u>Equation 10</u>									
Crater Lake	0.744	-0.175	-0.119	26.782	0.798	-0.082			0.671
Hemcross	0.628	-0.284	-0.139	11.192	1.361	-0.065			0.638
Tolovanna	0.529	-0.425	-0.123	9.550	0.520	0.093			0.843
Jory	0.985	-0.343	-0.048	5.445	1.255	0.092			0.364

† Nonlinear regressions of andic soils failed to converge.
†† Parameter was not significantly different from zero ($p < 0.05$).

when the θ_m variable was included in the expression containing the C parameter (Table III.7). The mean square error of Equation 9 for the Jory soil was smaller than the error for Equation 7.

The expression containing the F parameter was then deleted from Equation 9:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m) \cdot (1 - \mathrm{EXP}(-C \cdot \sigma \cdot (1 - \theta_m))). \quad [10]$$

Parameters in Equation 10 were significant for all soils but the mean square error of each soil was larger than the error of Equation 9 (Table III.7).

In the second step of developing a nonlinear model of soil compression, an exponent, H, was added to the θ_m variables in Equations 9 and 10. The equation including the F parameter was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m^H + F \cdot \sigma \cdot \theta_m^H) \cdot (1 - \mathrm{EXP}(-C \cdot \sigma \cdot (1 - \theta_m^H))), \quad [11]$$

and without the variable for the F parameter:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m^H) \cdot (1 - \mathrm{EXP}(-C \cdot \sigma \cdot (1 - \theta_m^H))). \quad [12]$$

A value of 0.001 was added to the θ_m^H variable for these analyses so that the derivatives in the Marquardt nonlinear regression procedure could be solved for saturated soil.

Equation 11 failed to converge for all soils. As an alternative, values of 0.5, 1.5, and 2 were assigned to H. At least one assigned exponent for each soil resulted in an E or F parameter which was not significantly different from zero, or else the model failed to converge

(Table III.8). The larger exponents resulted in more frequent failure of Equation 11 for these soils.

Equation 12 failed to converge for the Hemcross soil and the E and F parameters were zero for the Crater Lake soil (Table III.9). The estimated values of H for the Tolovanna and Jory soils were 2.304 and 1.945, respectively. Values of 0.5, 1.5, and 2 were also assigned to Equation 12. A small decrease in the mean square error of the Tolovanna and Jory soil occurred when a value of 2 was assigned to H rather than estimating it with Equation 12. The mean square error of the Crater Lake and Hemcross soils also decreased as the assigned exponent increased. Therefore, the exponent of the θ_m variable in Equation 12 was assumed to be 2 for all soils (Table III.9). The model with the θ_m variable having an exponent of 2 was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m^2) \cdot (1 - EXP(-C \cdot \sigma \cdot (1 - \theta_m^2))). \qquad [13]$$

Equation 13 resulted in a smaller mean square error for the Jory and Hemcross soils but a larger error for the Crater Lake and Tolovanna soils (Table III.8, III.9, and III.10). The E parameter in Equation 13 was not significantly different from zero ($p<0.05$) for the Hemcross soil. Deletion of the $E \cdot \theta_m^2$ expression from Equation 13 reduced the mean square error of the Hemcross soil but not the mean square error of the other soils.

Equation 13 was the best general model of soil compression which fit these data. Equation 13 resulted in the lowest mean square error of the Jory soil and the Hemcross soil when the E parameter was

Table III.8. Parameters and mean square errors (MSE) for Equation 11 when values of 0.5, 1.5, and 2 were assigned to the H parameter. Date included all tests of saturated and partly saturated samples of each soil.

Soil	ρ_o	A	B	C	D	E	F	MSE
	Mg/m^3							10^{-3}
H parameter is equal to 0.5								
Crater Lake	0.744	-0.132	-0.156	36.147	1.087	-0.224	0.131	0.659
Hemcross	0.629	-0.233	-0.169	12.400	1.543	-0.309	0.214	0.653
Tolovanna	0.530	-0.395	-0.133	12.652	0.512	-0.038†	0.040†	0.853
Jory	0.984	-0.286	-0.079	7.004	1.352	-0.139	0.109	0.364
H parameter is equal to 1.5								
Crater Lake	0.744	-0.188	-0.126	14.531	0.892	-0.190	0.129	0.443
Hemcross	0.628	-0.311	-0.131	8.875	1.388	-0.035†	0.023†	0.588
Tolovanna	0.528	-0.408	-0.126	11.436	0.441	0.374	-0.324	0.791
Jory	0.984	-0.347	-0.054	4.914	1.154	0.200	-0.008†	0.314
H parameter is equal to 2								
Crater Lake	failed to converge							
Hemcross	0.631	-0.400	-0.554	6.493	1.633	-0.075†	-0.690	0.290
Tolovanna	0.530	-0.473	-0.447	8.673	0.551†	0.279†	-2.186	1.183
Jory	failed to converge							

† Parameter was not significantly different from zero ($p < 0.05$).

Table III.9. Parameters and mean square errors (MSE) for Equation 12 when H was estimated by regression, and when values of 0.5, 1.5, and 2 were assigned to the H parameter. Date included all tests of saturated and partly saturated samples of each soil.

Soil	ρ_o	A	B	C	D	E	H	MSE
	Mg/m^3							10^{-3}
<u>H parameter determined in Equation 12</u>								
Crater Lake	0.728	-0.262	-0.094	16.302	0.611	0	0	0.834
Hemcross	failed to converge							
Tolovanna	0.528	-0.411	-0.135	7.916	0.606	0.261	2.304	1.134
Jory	0.984	-0.340	-0.064	4.671	1.087	0.271	1.945	0.285
<u>H parameter is equal to 0.5</u>								
Crater Lake	0.743	-0.147	-0.134	55.435	0.865	-0.090		0.839
Hemcross	0.628	-0.215	-0.178	21.651	1.248	-0.010		0.803
Tolovanna	0.529	-0.391	-0.134	13.519	0.485	-0.004†		0.853
Jory	0.985	-0.312	-0.046	7.478	1.405	-0.024		0.443
<u>H parameter is equal to 1.5</u>								
Crater Lake	0.744	-0.197	-0.108	18.742	0.743	-0.067		0.579
Hemcross	0.628	-0.311	-0.131	9.070	1.371	-0.017†		0.587
Tolovanna	0.528	-0.407	-0.142	9.469	0.548	0.172		0.975
Jory	0.984	-0.345	-0.057	4.933	1.150	0.192		0.313
<u>H parameter is equal to 2</u>								
Crater Lake	0.744	-0.214	-0.102	15.137	0.706	-0.050		0.524
Hemcross	0.627	-0.323	-0.130	8.184	1.362	-0.034†		0.569
Tolovanna	0.528	-0.416	-0.136	8.102	0.582†	0.247		1.101
Jory	0.984	-0.339	-0.063	4.641	1.089	0.272		0.280

† Parameter was not significantly different from zero ($p < 0.05$).

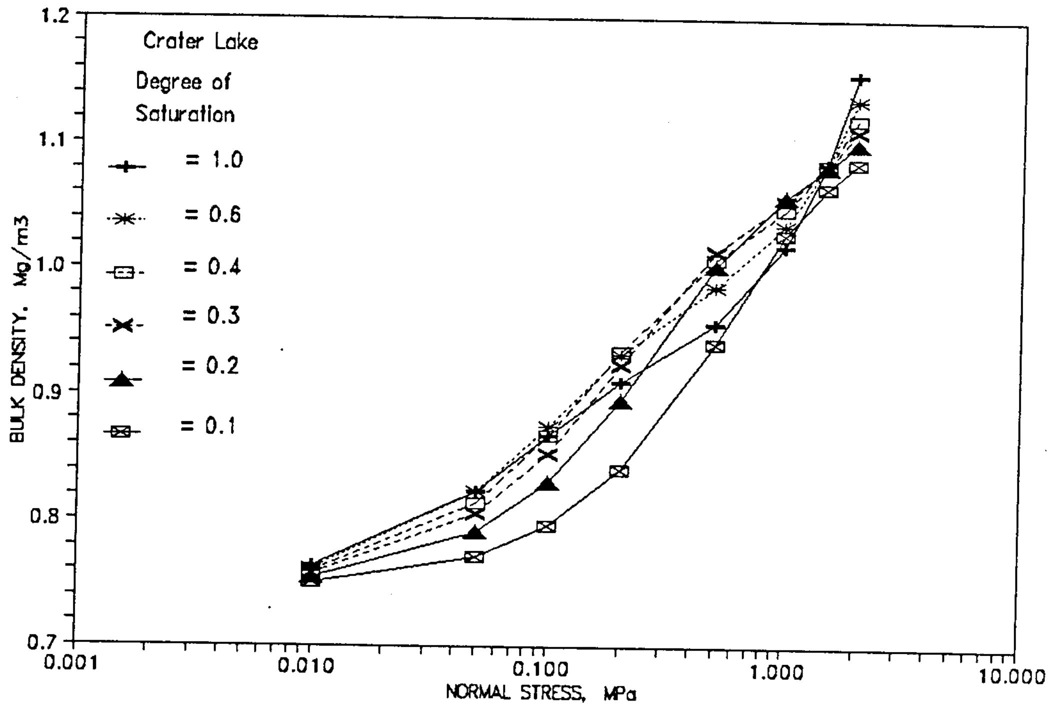
Table III.10. Parameters and mean square errors (MSE) for Equation 13. Data included all tests of saturated and partly saturated samples of each soil.

Soil	ρ_o	A	B	C	D	E	MSE
	Parameters						
	Mg/m^3						10^{-3}
Crater Lake	0.744	-0.214	-0.102	15.137	0.706	-0.050	0.524
	0.002	0.010	0.009	1.372	0.105	0.015	
Hemcross	0.627	-0.323	-0.130	8.184	1.362	0.034†	0.569
	0.001	0.014	0.012	0.554	0.074	0.023	
Tolovanna	0.528	-0.416	-0.136	8.102	0.582	0.247	1.101
	0.002	0.023	0.018	0.769	0.104	0.040	
Jory	0.984	-0.339	-0.063	4.641	1.089	0.274	0.280
	0.001	0.011	0.007	0.279	0.041	0.013	
E Term Delected From Equation 9							
Hemcross	0.627	-0.317	-0.128	8.093	1.406		0.527
	0.001	0.013	0.012	0.546	0.069		

† Parameter was not significantly different from zero ($p < 0.05$).

zero (Table III.10). Equation 13 was accepted as the model of the Crater Lake and Tolovanna soils following rejection of intermediate models with lower mean square errors. Models with low mean square errors were judged unacceptable when the plot of the compression curves failed to meet the criteria that the curves be nearly parallel at higher normal stresses (Figure III.4). The plot of the compression curves of the Crater Lake and Tolovanna soils with Equation 13 produced more typical compression curves (Figure III.5). Although the compression curves of the Crater Lake soil continued to converge or cross at higher stresses, the lines were more nearly parallel. The range in predicted bulk density at normal stresses between 0.05 and 0.5 MPa was also narrower when calculated using Equation 13. The narrower range in predicted bulk density was similar to the narrow range of measured bulk density (Figure III.3A). The compression curves of the Hemcross soil also converged at normal stresses greater than 1 MPa, although the compression curves of the Tolovanna soil did not (Figure III.5). Changes in the slope of the compression curves of partly saturated soils at higher stresses are not uncommon. Larson et al. (1980) reported a significant difference in the compression index of partly saturated soil, as degree of saturation changed, for about 20 percent of the soils. Jennings and Burland (1962) also reported that compression curves of a silty sand converged at higher normal stresses and degrees of saturation.

The average difference between the lowest initial bulk density and the highest predicted bulk density for the andic soils was 0.52



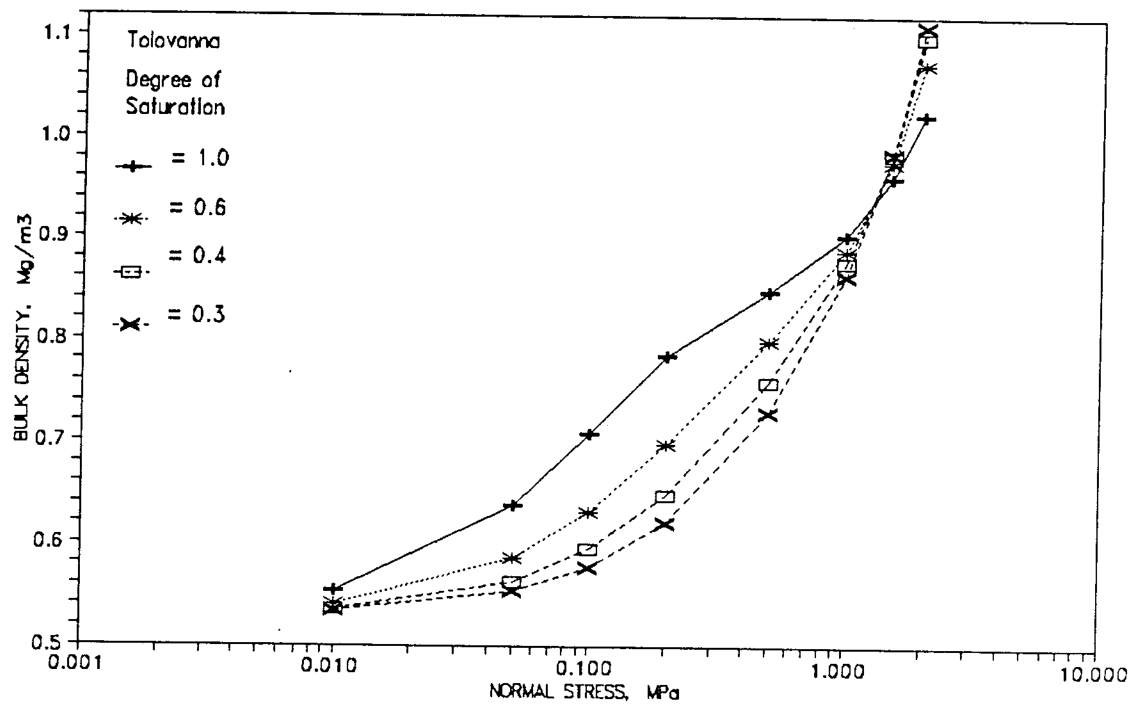


Figure III.4. Effects of degree of saturation on the relationship between bulk density and normal stress of the Crater Lake and Tolovanna soils. Bulk density of the Crater Lake soil was calculated using Equation 11 and a value of 1.5 for the H parameter, and Equation 9 was used to calculate the bulk density of the Tolovanna soil.

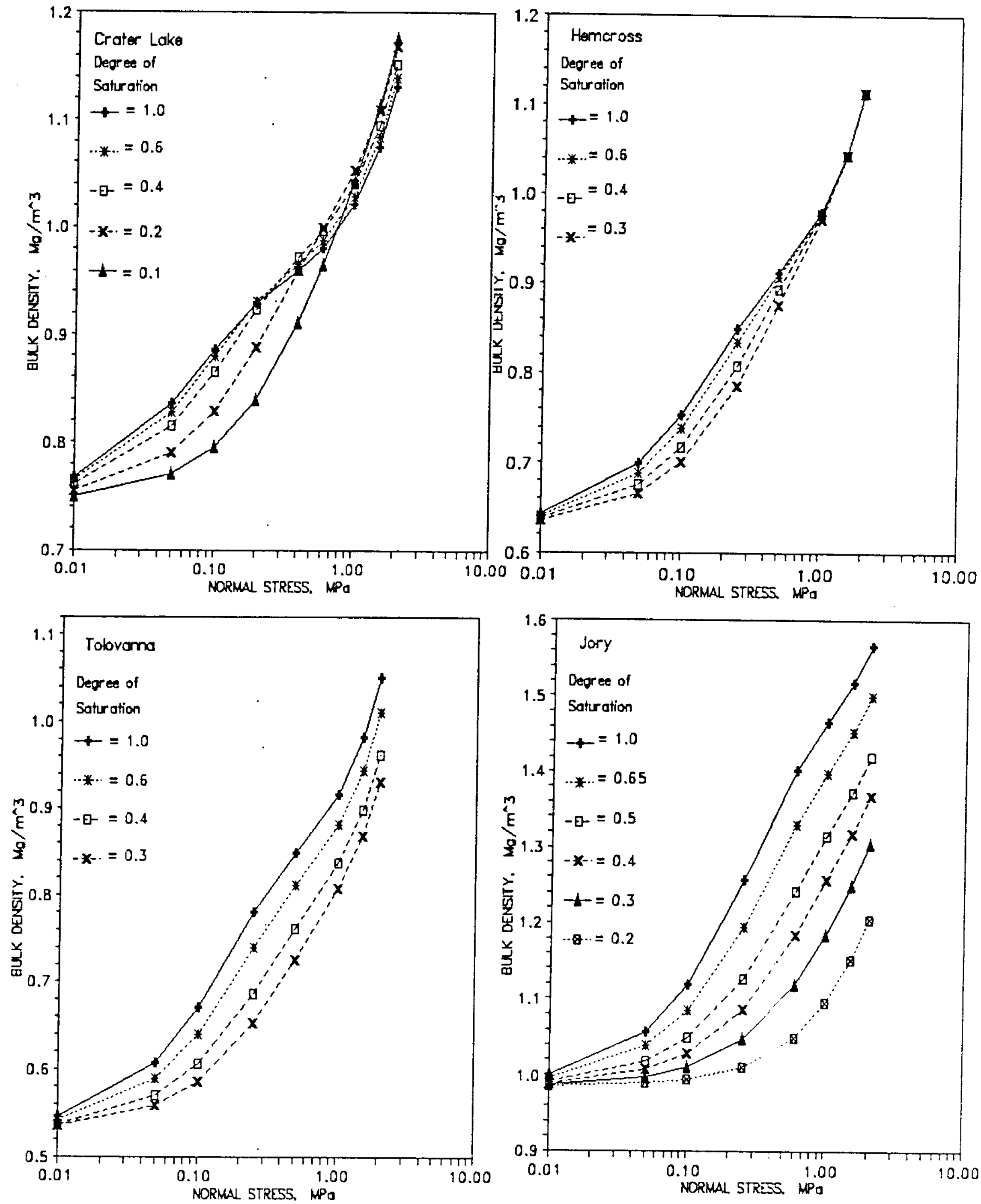


Figure III.5. Effects of degree of saturation on the relationship between the compressed bulk density and normal stress of four western Oregon forest soils. Bulk densities were calculated using Equation 13 (Table III.10). The degrees of saturation for each soil were limited to those existing at the time of sampling.

Mg/m^3, the difference for the Jory soil was 0.69 Mg/m^3. Equation 13 was used to calculate the bulk density of each sample using the measured initial bulk density, normal stress, and degree of saturation of each sample. These calculated bulk densities were compared to the measured bulk densities of each soil by regression analysis (Table III.11). Equation 13 more accurately predicted bulk density of the Hemcross and Jory soils than it did the Crater Lake and Tolovanna soils. The mean square error of the predicted bulk density of these two soils averaged 0.03 Mg/m^3. The mean square errors of the predicted bulk density of the Crater Lake and Tolovanna soils were higher. The higher errors were another indication of the difficulty of fitting a model to these data.

The A, B, and C parameters for Equation 13 were all significantly correlated with one another (Table III.12). The correlation coefficients between A and B parameters were consistently the highest for all soils. The effect that this high correlation between A and B parameters has on predicting bulk density is small, however, because the B parameter only affects the bulk density of soil at higher normal stresses (Figure III.1). Furthermore, changes in these parameters tend to offset each other because, as the A parameter becomes more negative, the B parameter becomes less negative. This is confirmed by comparing the bulk densities calculated using Equation 2 and parameters for saturated soil (Table III.6), and the bulk densities calculated using Equation 13 for saturated soil (Table III.9). The parameters for each soil in Equation 2 and 13 differed by 10 to 30

Table III.11. Range in measured bulk densities of undisturbed and compressed soils, and the regression coefficients of bulk density predicted with Equation 13 and the bulk density (independent variable) measured on soils compressed at normal stresses between 0.033 and 1.96 MPa.

Soil	n	Range in Bulk Density	Regression parameters b_o	b_1	R^2	Std. Err.
		Mg/m^3				Mg/m^3
Crater Lake	84	0.729-1.198	0.054	0.931	0.900	0.034
Hemcross	108	0.559-1.099	0.029	0.957	0.945	0.029
Tolovanna	76	0.477-1.029	0.042	0.930	0.901	0.040
Jory	138	0.881-1.574	0.031	0.950	0.950	0.031

Table III.12. Nonlinear regression correlation matrix of soil compression parameters estimated with Equation 13. The $E{\cdot}\theta_c^2$ term was deleted from Equation 13 for the analysis of the Hemcross soil (Table III.10).

Soil/ Parameter	ρ_o	A	B	C	D
Crater Lake					
A	0.171				
B	0.083	-0.718			
C	-0.300	0.541	-0.683		
D	-0.121	-0.074	-0.425	0.593	
E	0.109	0.003	0.283	-0.440	-0.607
Hemcross					
A	0.045				
B	0.102	-0.926			
C	-0.281	0.852	-0.804		
D	0.114	-0.501	0.525	-0.512	
Tolovanna					
A	0.029				
B	0.109	-0.875			
C	-0.295	0.775	-0.774		
D	0.045	0.165	-0.019	-0.036	
E	0.037	-0.585	0.307	-0.199	-0.355
Jory					
A	0.057				
B	0.021	-0.849			
C	-0.194	0.840	-0.813		
D	0.114	-0.370	0.399	-0.524	
E	0.031	-0.217	-0.051	0.102	-0.204

percent but resulted in similar bulk densities of saturated soil (Figure II.6). Differences in the value of ρ_o were responsible for most of the differences in the calculated bulk densities of each soil.

Correlation coefficients between A, B, or C parameters, and the soil water content (E) or variation in initial bulk density (D) parameters were generally lower than the correlation between the original parameters (Table III.12). These lower correlation coefficients were added confirmation that inclusion of the expressions containing the D and E parameters in Equation 13 were valid and that bulk densities predicted by Equations 2 and 13 would converge for saturated soil. This was demonstrated by the similarities in the predicted bulk density of saturated soil with Equations 2 and 13 (Figure III.6).

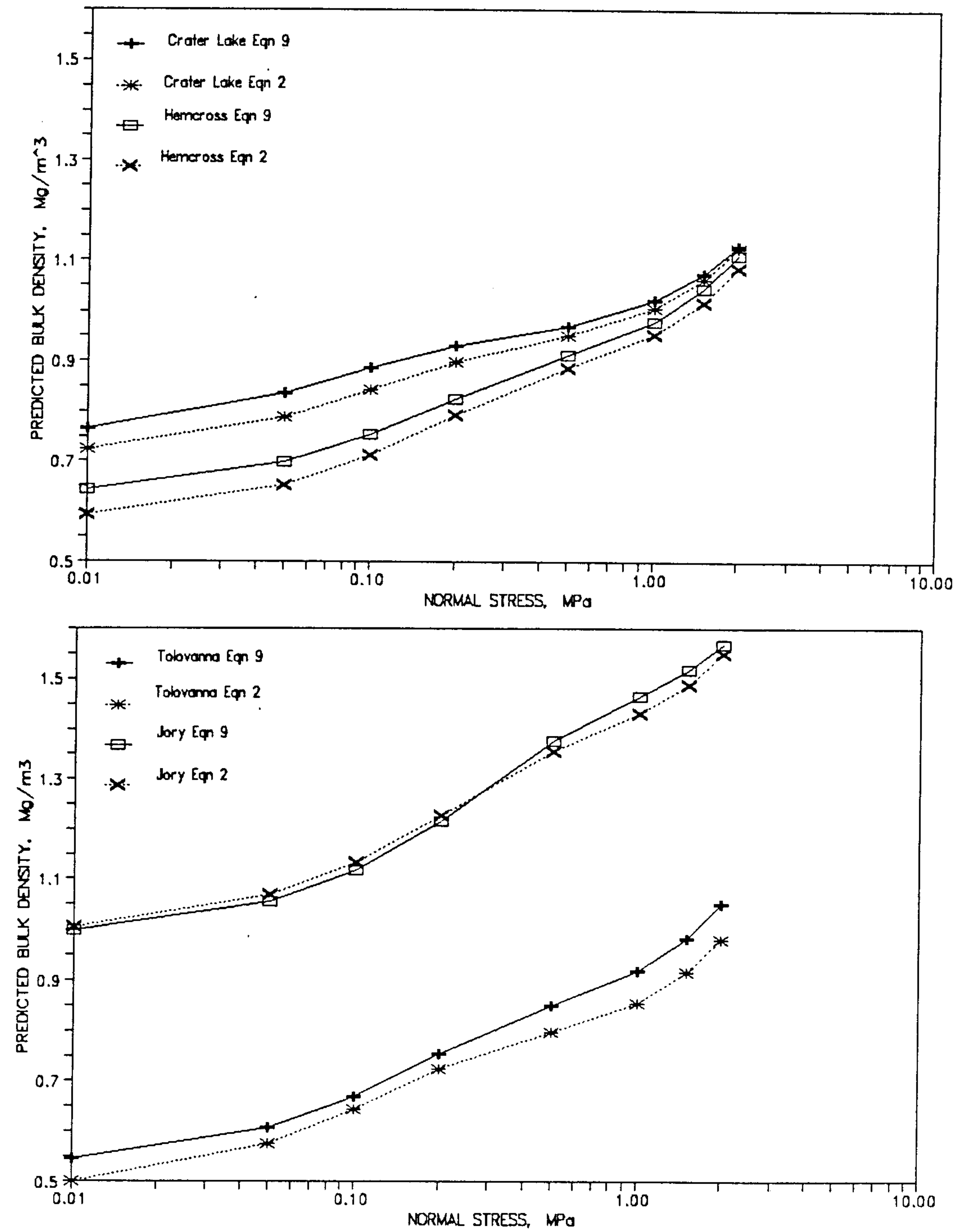


Figure III.6. Effects of parameters on the relationship between bulk density of saturated soil and normal stress. The bulk density of saturated soil was calculated using the parameters for the general soil compression model (Eqn 13; Table III.10) and the parameters for saturated soil using Equation 2 (Table III.2).

DISCUSSION

The advantage of the nonlinear model of soil compression proposed by Bailey et al. (1986), is that it satisfied the boundary conditions of predicting bulk density of soil at low and high stresses (Figure III.1). Modification of this nonlinear model of soil compression in Chapter II for the variation in bulk density of individual samples provided the basis for combining data for samples compressed at different water contents. The adjustment of the model for variation in bulk density was most successful when the shape of the compression curves of each soil were similar. Large differences in bulk density apparently affected the shape of the compression curve of individual samples. This occurred when Equation 2 was used to predict the bulk density of the Jory soil at low water contents (Table III.3) and reduced the fit of Equation 13 to the data of the Tolovanna soil (Table III.11). Therefore, the fit of Equations 2 and 13 to data obtained from compression of samples with diverging compression curves, or extreme values of bulk density, is reduced. Such situations are most likely to occur when samples with different stress histories, i.e., past soil treatments, are combined.

Previous analyses using Equation 1 found that soil water content affected the parameters (Bailey et al., 1986). Soil water content was also a significant variable in multiple linear regression models that predicted the parameters from soil properties (McBride, 1989). A test of the hypothesis that parameters were related to soil water content with these data failed to confirm such a relationship

(Table III.5). Additional testing is needed to determine if parameters are affected by soil water content.

Equation 13 is a general model of soil compression for these soils but selection of this model was complicated by the complex effect that soil water had on compression (Figure III.5). Part of the difficulty is attributed to the unique water retention characteristics of andic soils, which resulted in the convergence of the compression curves at high normal stresses. These soils also had a high porosity at field capacity (Figure III.1), which reduced the range of sampled water contents of partly saturated soil. This increased the difficulty of fitting the model to the data because of the absence of data for degrees of saturation between that found at field capacity and that found in saturated soil. Although this absence of data increased the difficulty of fitting the model to the data, the difference in the compressibility of soil between saturation and field capacity was small (Figure III.5). It is also uncertain whether the exponent to the θ_m variable is 2 in Equation 13 for other soils if the effect of soil water on the compressive strength of partly saturated soils differs from these forest soils. Therefore, the value of the H parameter in Equation 12 for other soils with different compressibilities near field capacity needs to be determined.

Excessively high correlations between parameters ($r > \pm 0.99$) indicate an overparameterized model (Bates and Watts, 1988). Such a model contains more parameters than are needed to describe the data set. The high correlation among A, B, and C parameters did not affect the fitting of Equation 13 to these data (Table III.12). But, the high correlation does limit the direct comparison of parameters and the prediction of parameters from other soil properties. This weakness of the nonlinear model of soil compression is best illustrated by comparing the A, B, and C parameters for the Jory soil when saturated (Eqn 2) with those of Equation 13 (Tables III.3 and III.10; Figure III.6).

Equation 13 is a more comprehensive model of soil compression than the equations developed by Larson et al. (1980) for predicting the compression of partly saturated soil. The Larson et al. equations predicted bulk density from the relationships between bulk density at a constant low normal stress and degree of saturation, and bulk density and the logarithm of normal stress. Equation 13 simplified the prediction of bulk density by using a single equation that did not require an estimate of bulk density at a constant low stress which is affected by degree of saturation. Equation 13 also predicted bulk density for a wider range of matric pressures than the equations of Larson et al. (matric pressure > -0.1 MPa). The Larson model would also require establishing more than one relationship between estimated bulk density at a constant stress and degree of saturation if this model was to include a wider range of soil water contents.

The principal advantage of Equation 13 is that it predicts the bulk density for any normal stress with a single set of parameters. Compression of soil at low stresses, which result in small increases in bulk density, has not been adequately described by previous equations. Being able to do this has important practical significance when predicting bulk density for agronomic and forestry applications. The stresses necessary to cause a small increase in bulk density vary widely because desiccation and management practices have the greatest effect on the compressibility of soil in this range of bulk densities.

The consolidation of these soils when saturated was rapid at all normal stresses (Chapter II). As a result, the normal stress applied during compression was determined by the compressive strength of the soil (Lambe and Whitman, 1979). The Jory soil was more compressible when saturated than were the andic soils. Therefore, the strength of the Jory soil was less than the compressive strength of the andic soils (Figure III.5). This was confirmed by the measurement of shear strength in direct shear tests (Chapter II). The shear strength of the Jory soil was significantly less than the shear strength of the andic soils. As water content decreased in partly saturated soil, the compressibility of the Jory soil increased more rapidly than did the compressibility of the andic soils. As a result, the Jory soil was less compressible at low water contents than were the andic soils. The differences in compressibility of these soils are apparently related to how decreasing soil water content affects the contacts between particles.

Soil strength is directly related to the number of contacts between particles, regardless of the texture and mineralogy of the soil (Mitchell, 1976). Soils of different bulk densities have similar strength if the number of particle contacts are equal. Increasing the bulk density of a specific soil, increases the number of contacts between particles, which results in an increase in soil strength. Therefore, the compressibility of soil is dependent on increasing the number of contacts between particles until the increase in soil strength is sufficient to withstand the applied stresses.

The increase in bulk density of the Crater Lake soil was also low because coarse-textured soils are less easily compressed in static compression tests than tests which use more dynamic loading techniques (Lambe and Whitman, 1979). Soil water content had a greater effect on soil compression at lower normal stresses than at higher normal stresses because high normal stresses apparently exceeded the increase in soil strength resulting from the decrease in matric pressure (Tezaghi and Peck, 1967; Vomocil et al., 1968). Part of the decrease in compressibility of the Crater Lake soil may have resulted from a decrease in the matric pressure within the porous grains of volcanic ash (Borchardt et al., 1968; Flint and Childs, 1984), rather than between particles.

The compressibility of the Jory soil was high when saturated because layer silicate minerals are forced apart by adsorption of a strongly held, thick layer of water on the surface of these minerals (Mitchell, 1976). The adsorption of multiple layers of water on mineral surfaces reduced the number of particle contacts between clay minerals, and between clay minerals and other soil particles, at high degrees of saturation. The strength of the Jory soil when saturated was low for this reason (Chapter II). The thickness of the water layer between particles decreased and the attractive forces between particles increased as the matric pressure decreased (Mitchell, 1976). As a result, the strength of the Jory soil increased due to the combination of a decreased matric pressure and an increased number of contacts and chemical bonds between particles. Thus, the Jory soil became more resistant to compression at lower degrees of saturation than did the andic soils.

The fine-textured Hemcross and Tolovanna soils had a high strength when saturated because only one layer of water molecules is strongly adsorbed on the surface of noncrystalline minerals (Wada and Harward, 1974; Maeda et al., 1977). Most of the water in fine-textured andic soils is retained within the mineral structure and in noncontinuous pores. The thin layer of water adsorbed on mineral surfaces apparently did not reduce contacts between particles. As a result, the strength of these soils when saturated was high despite their low bulk densities (Chapter II). As these soils dried, the increase in contacts and bonds between these noncrystalline minerals was small because most of the soil water was lost from within the mineral. This loss of water apparently increased the matric pressure within the mineral structure, which increased the strength of the soil and made the soil less compressible at lower stresses (Figure III.5). At higher normal stresses, noncrystalline minerals were apparently compressed in a manner similar to that of saturated soil.

The increase in bulk density associated with a specific applied stress, rather than the undisturbed or compacted bulk density of the soil, determined the compressibility of these undisturbed forest soils. When soil water content was above that of field capacity, the andic soils were less compressible than was the Jory soil. This was so because of the high strength of the andic soils rather than because of their low compacted bulk densities. As the water content decreased, the strength of the Jory soil increased more rapidly than did the strength of the andic soils. As a result, the Jory soil was less compressible than the andic soils at low water contents.

These results have several implications with respect to forest management practices. Undisturbed and compacted bulk density is not an indicator of the compressibility of different soil materials. Differences in the compressibility of these forest soils is not as great as commonly assumed; for a given mechanical stress, all soils will compact until the soil develops sufficient strength to support the stress, or the soil fails. Soil water content has less effect on the compressibility of andic soils than it has on cohesive soils, particularly at higher stresses (Figure III.5). As a result, operating machines on dry cohesive soils will cause much less compaction of the soil than when the soil is moist. Differences in water content will have less effect on the compression of andic soils. Under field conditions, the natural range of soil water contents in the Hemcross and Tolovanna soils which may affect the compressibility of these soil is even less than sampled because the soils were covered to prevent rewetting. These soils occur near the Pacific Ocean and regularly receive more precipitation in the form of rainfall or fog drip than the other soils.

The small differences in the compressibility of these soils from a mechanical stress are not likely to have a similar effect on the growth of plants. The increase in soil strength may reduce the penetration of soil by roots, which can affect the growth of plants (Taylor and Bruce, 1963; Greacen and Sands, 1980). But soil compression may also affect aeration, the availability of water and nutrients, and soil organisms important to the growth of plants on these soils differently. Furthermore, all these factors must be considered with respect to how they alter the environment of the site for a specific species (McNabb and Campbell, 1985). The nonlinear model is one method of predicting the compression of soil resulting from mechanical stress; the effect of soil compression predicted by such a model on the growth of plants must be determined separately and on a site specific basis.

CONCLUSIONS

A nonlinear model of soil compression (Eqn 13) accurately predicted the bulk density of undisturbed forest soils for a wide range of applied stresses and soil water contents. The fit of the model to data was reduced when the data included samples with extreme bulk densities or samples with different compression curves. High correlations among some parameters affected the interpretation of individual parameters and the ability to relate parameters to other soil properties, but did not affect the accuracy of the model.

Solution of the nonlinear model was more difficult for andic soils because changes in soil water had a smaller, more variable effect on the compression of noncrystalline minerals than it did the soil that contained layer silicate minerals. The high porosity of all soils, and the narrow range of partly saturated water contents of some soils, also made solving the model more difficult. These factors affected the determination of the parameter exponent for the soil water variable, and the selection of the final model. Additional tests of cohesive soils are needed to determine the exponent in the expression for soil water and interpreting the parameters of Equation 13.

Soils with a high strength were less compressible than the soil with low strength, regardless of the bulk density of the soil. The increase in bulk density resulting from an applied normal stress was a measure of the difference in the compressibility of these soils. Soil compressibility varied depending on how soil water affected strength. Differences in texture and clay mineralogy dominated the differences in strength of these soils as water content decreased.

The compressibility of these forest soils was similar near field capacity. Thus, all soils must be considered susceptible to compression when wet. A larger decrease in the compressibility of the cohesive soil occurred as soil water content decreased than occurred in andic soils. This was assumed to occur because of large changes in the layer of water on the surface of layer silicate minerals, which did not occur in noncrystalline minerals. As a result, cohesive soils are more compressible when wet and less compressible when dry than are andic soils. Therefore, compaction of cohesive soils is expected to be much less if machines only operate on dry soil but the difference in compaction of andic soils will be much less, particularly at higher normal stresses.

The nonlinear model of soil compression is an important first step in the development of a model of soil compression from mechanical stresses for a wide range of stresses and water contents. Field trials are needed to validate the nonlinear model of soil compression. The effect that differences in soil compressibility predicted by the nonlinear model has on the growth of plants also remains to be determined.

CHAPTER IV

SUMMARY

Consolidation of saturated, undisturbed soil was rapid because of the high macroporosity of the soil. At least 40 percent of the pore volume was in air-filled pores at field capacity, -0.10 MPa. The relatively large increase in bulk density at low normal stresses, and the continuous increase in bulk density and shear stress with increasing strain indicated that the soils were normally consolidated. Because of the rapid dissipation of pore water pressures during consolidation and high compressibility of the soils at low normal stresses, compaction is expected to cause large increases in bulk density of these soils when wet, regardless of the duration of the stress.

When saturated, the more dense Jory soil, which contained layer silicate minerals, had a significantly higher ($p<0.05$) compression index than did the andic soils. The increase in bulk density when calculated using a nonlinear model of saturated soil compression was also larger for the Jory soil than for the andic soils. The direct shear test confirmed that the strength of the Jory soil was significantly less ($p<0.05$) than that of the andic soils. Therefore, it was concluded that andic soils were less compressible than the more dense Jory soil because of a high soil strength rather than a low bulk density. However, differences in the compressibility of saturated soils at low stresses was small regardless of the bulk density.

A nonlinear model of soil compression was adapted to predict bulk density from compression of individual samples. This model was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c) \cdot (1 - EXP(-C \cdot \sigma)), \qquad [1]$$

where ρ_c is the compressed bulk density, ρ_o is the estimated bulk density at zero stress and is estimated by regression, δ_i normalizes ρ_o for the variation in initial bulk density of individual samples, ρ_i:

$$\delta_i = \rho_i - \frac{\sum^{n} \rho_i}{n}, \qquad [2]$$

σ is the normal stress, and δ_c adjusts the compression curve for the variation in bulk density:

$$\delta_c = (\delta_i - 1) \cdot \rho_o. \qquad [3]$$

A, B, C, and D are parameters estimated by nonlinear regression. This model was used to predict the bulk density of saturated soil and of samples collected at different in situ water contents.

Parameters for all saturated soils were significantly different from zero ($p<0.05$). Equation 1 predicted the bulk density of the average compression curve for each soil when δ_i was one. When Equation 1 was used to estimate parameters for partly saturated soil, parameters were not always significantly different from zero and the model failed to estimate parameters for the Jory soil at some lower water contents. Parameters were seldom related to the water content of the soil.

Equation 1 was expanded to include variables for water content of partly saturated soil. The equation was modified in several steps. Several intermediate models were rejected because of a high mean square error of the model or inconsistencies in the compression curves for different degrees of saturation. The final model for these soils was:

$$\ln(\rho_c) = \ln(\rho_o \cdot \delta_i) - (A + B \cdot \sigma + D \cdot \delta_c + E \cdot \theta_m{}^2) \cdot$$
$$(1 - \mathrm{EXP}(-C \cdot \sigma \cdot (1 - \theta_m{}^2))). \qquad [4]$$

The $\theta_m{}^2$ variable is:

$$\theta_m{}^2 = (1-\theta_s)^2, \qquad [5]$$

where θ_s is the degree of saturation. When the soil is saturated, Equation 4 simplifies to Equation 1. Three parameters (A, B, and C) described the general relationship between bulk density and applied normal stress. One parameter (D) adjusted the compression curve for the variation in bulk density of individual samples. The E parameter adjusted the compression curve for the effects that soil water had on the compressive strength of the soil.

The fit of Equations 1 and 4 to the data was affected by differences in the bulk density of individual samples. Extreme values of bulk density prevented Equation 1 from fitting the data of the Jory soil at some lower water contents and increased the mean square error of Equation 4 for predicting the bulk density of the Tolovanna soil. The fit of Equation 4 to the data of the Crater Lake and Tolovanna soils was also affected by the narrow range of partly saturated water contents and the high macroporosity of soils. These factors reduced the fit of Equation 4 to the data and occasionally resulted in intermediate models having a lower mean square error than Equation 4. The intermediate models were rejected because of inconsistencies in the compression curves calculated using these models. Least difficult to fit a model to was the Jory soil data.

The exponent to the soil water variable in Equation 4 could not be determined for the Crater Lake and Hemcross soils but was assumed to be two following trial solutions of the model with different exponents for the variable for water content. The exponent of the Jory and Tolovanna soils was near two. A value of two for the exponent was assumed for all soils.

The advantage of the nonlinear model of soil compression is that one equation accurately predicted the bulk density of undisturbed soils at all normal stresses and water contents. The disadvantages are that the A, B, and C parameters were often significantly correlated. Parameters for soils with a low undisturbed bulk density were also larger than those for soil with a high undisturbed bulk density. These factors do not affect the accuracy of the model to predict the bulk density of a specific soil, but do reduce the ability to relate parameters to other soil properties or to make direct comparisons of parameters among soils. More tests of Equation 4 are needed, particularly of cohesive soils, to determine the range of parameters, effects of soil water on the compressive strength of soil, and the exponent of the variable for soil water.

The undisturbed and compressed bulk densities of these soils were not an indicator of their compressibility by mechanical stress. At low normal stresses, the compression of soils when saturated were similar regardless of their undisturbed bulk density. With decreasing water content, differences in the compressibility of the soil increased at all normal stresses. The difference in compressibility was dominated by differences in soil texture and clay mineralogy and not bulk density.

The coarse-textured Crater Lake soil was less compressible than the other soils because coarse-textured soils are less easily compressed by static loading. Clay mineralogy dominated the compression of the fine-textured andic soils and the Jory soil because clay mineralogy determined where soil water was retained within the soil and how strongly the water was held. The higher compressibility and lower strength of the Jory soil when saturated was attributed to the thick layer of strongly held water surrounding the layer silicate minerals. The thick layer of water reduced the number of contacts and bonds between particles, which has been directly related to soil strength. The compressive strength of the Jory soil increased and the soil became less compressible than the andic soils at lower degrees of saturation because the decrease in the thickness of the layer of water surrounding clay minerals apparently resulted in a large increase in particle contacts and bonds. Most of the soil water in the fine-textured andic soils was apparently lost from within the mineral structure and noncontinuous pores of the noncrystalline clay minerals and, therefore, caused smaller changes in the contacts between particles and compressive strength of the Hemcross and Tolovanna soils. As a result, differences in water content had a greater effect on the compression of the Jory soil than on the compression of the Hemcross and Tolovanna soils. Differences in the compressibility of the Hemcross and Tolovanna soils in the field are small because of the narrowness of the natural range of soil water contents.

The implications of these results for the management of forest soils are several. Differences in the compressibility of soils are not determined by bulk density but by soil strength. Differences in the compression of soil when wet are small; all soils tested readily compressed at all stresses. Andic soils have a low compacted bulk density because of high soil strength and not because of low bulk density. Compressibility of andic soils are less affected by decreasing soil water content than is cohesive soil. Therefore, compaction of cohesive soils can be reduced by operating machines on dry but such reduction in the compaction of andic soils are expected to be small, particularly when the stresses applied to the soil are high or the natural range of water contents is narrow.

These results define the compressibility of soil as a response to mechanical stress. How differences in the compressibility of soil affects plant growth on a site specific basis has yet to be determined. Compression increases soil strength, which increases the resistance of the soil to penetration by roots. But, compression of soil also affects several other soil properties important to plant growth that were not measured in this study. The effects of soil compression on these properties, and the site specific conditions when they may affect plant growth must be determined before the effects of compaction on plant growth can be accurately predicted.

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APPENDICES

DEFINITIONS

Bulk density (ρ) - The dry mass of soil divided by the volume that it occupies. The term is most commonly used in agronomy to describe soil density.

Cohesion (c) - The estimated shear resistance of a soil when the normal stress is zero. True cohesion is generally negligible in the absence of chemical bonding between particles.

Cohesive soil - Fine-textured soils dominated by the physico-chemical properties of clay minerals. Soil behavior is dominated by soil water content and associated changes in soil volume.

Cohesionless soil - Coarse-textured soils that contain gravel and sand sized particles and only a small percentage of silt and clay. Physico-chemical properties have a negligible effect on soil behavior.

Consolidation - The change in volume of soil with the passage of time that results from the application of a static, external load. Volume change is partially controlled by dissipation of pore water pressures according to Terzaghi's consolidation theory. Therefore, it is used to describe tests of saturated soil in these chapters.

Consolidation test - A test that measures the one-dimensional consolidation of soil caused by applying a static, external load and measuring the volume change of soil over time.

Compaction - The volume change produced by momentary application of a load to a soil.

Compression - The change in volume of a soil produced by the application of a static external load. Volume change is not constant but decreases for similar increases in external load. Because compression of partly saturated soil is not controlled by dissipation of pore water pressure, the term is used to describe volume change of partly saturated soil.

Compression index (C_c) - The slope of the linear portion of the relationship between bulk density and logarithm of normal stress when the applied normal stresses are greater than the overconsolidation stress.

Compression curve - The relationship between bulk density and normal stress.

Compressibility - The relative change in volume of a soil produced by the application of a static external load.

Degree of saturation (θ_s) - The fraction of the volume of soil water in a soil to the volume of pores. Values range between 0 for dry soil and 1 for saturated soil.

Dilation - The decrease in bulk density or increase in volume of a sample during during a shear test.

Effective stress - The difference between total stress and pore water pressure. It is the actual grain to grain stress between soil particles.

Friction angle (ϕ) - A measure of the frictional resistance of a soil to shear stresses. Defines the angle of the relationship between shear strength and normal stress.

Matric pressure (μ_m) - The hydraulic head or hydrostatic pressure resulting from capillary and adsorptive forces due to the soil matrix. Matric pressure is a negative gauge pressure relative to atmospheric pressure.

Normal stress (σ) - The sum of all stresses acting perpendicular to a plane. In the one-dimensional consolidation and direct shear tests, the normal stress is acting perpendicular to the sample.

Overconsolidation - A characteristic of soil that has undergone volume change from a higher normal stress than currently is applied to the soil. Overconsolidation results when erosion exposes a more dense, deep soil layer. Desiccation causes an apparent overconsolidation of partly saturated soil because of the surface tension of water increases the soil strength.

Particle density - The ratio of the mass of soil particles to the volume occupied by the particles.

Primary consolidation - The compression of saturated soil that is partly controlled by the dissipation of positive pore water pressures. In partly saturated soil, primary consolidation refers to the rapid compression of soil but its completion cannot be accurately defined.

Porosity - The fractional volume of soil occupied by pores.

Rebound - The small volume increase resulting when a load is removed from a soil.

Secondary consolidation - The slow compression of soil that continues after the positive pore pressures have dissipated during consolidation of saturated soil. Secondary consolidation also occurs during compression of partly saturated soil but cannot be accurately separated from primary consolidation.

Shear strength line - The relationship between shear stress and normal stress when measured in a direct shear test.

Shear strength (τ) - The shear stress measured at failure. In a direct shear stress, the shear stress acts horizontally to the sample.

Shear stress - The sum of all stresses acting tangential to a shear plane.

Soil water content (θ_w) - The mass of water relative to the mass of dry soil particles. Also: mass wetness, gravimetric water content.

Strain (%) - The ratio of displacement to the total length of a material from the application of an external force, expressed as a percentage.

Void ratio (*e*) - The ratio of the volume of voids, or pores, to the volume of soil particles. The term is most commonly used in engineering to describe soil density.

LIST OF SYMBOLS

Symbols	Description	Units
c	Cohesion intercept	MPa
d.f.	Degrees of freedom	
MSE	Mean square error	
R^2	Regression coefficient	
δ_i	Variable to normalize ρo for variation in bulk density	
δ_c	Variable to adjust the compression curve for variation in bulk density	
δ_r	Initial bulk density minus the average bulk density	Mg/m_3
θ_m	One minus the degree of saturation	
θ_s	Degree of saturation	
θ_w	Gravimetric water content	kg/kg
μ	Pore water pressure	MPa
ρ	Bulk density	Mg/m^3
ρ_c	Compressed bulk density	Mg/m^3
ρ_o	Bulk density at a normal stress of zero	Mg/m^3
ρ_i	Bulk density of core samples at a normal stress of 0.002 MPa	Mg/m^3
σ	Total normal stress	MPa
σ'	Effective normal stress	MPa
τ	Shear strength at failure	MPa
ϕ	Friction angle	Degree