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**Maintaining Soil Tilth and Preferred Soil
Structure under Intensive Field Mechanization
through Water Management and Tillage**

V.A. Snyder, A. Hadas

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Maintaining soil tilth and preferred soil structure under intensive field mechanization through water management and tillage

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TABLE OF CONTENTS

ABSTRACT.....	1
INTRODUCTION: Project Objectives.....	2
RESEARCH CONDUCTED UNDER OBJECTIVE 1.....	7
Chapter 1. Field and laboratory characterization of soil fragmentation in tillage- Israel.....	8
Chapter 2. Laboratory and field characterization of soil fragmentation in tillage- Puerto Rico.....	37
Chapter 3. Spectral analysis of cone penetrometer data for detecting spatial arrangement of soil clods.....	69
Chapter 4. Energy relations of propagating cracks in variably plastic soils.....	79
RESEARCH CONDUCTED UNDER OBJECTIVE 2.....	106
Chapter 5. Structured soil behavior under compressive and shear stresses.....	107
Chapter 6. Compression of soil aggregates.....	131
RESEARCH CONDUCTED UNDER OBJECTIVE 3.....	154
Chapter 7. Directional strength of aggregates as affected by aggregate volume and a wet/dry cycle.....	155
DESCRIPTION OF COOPERATION.....	161
EVALUATION OF RESEARCH ACHIEVEMENTS WITH RESPECT TO THE ORIGINAL RESEARCH PROPOSAL, WITH EMPHASIS ON BENEFITS TO AGRICULTURE.....	162
LIST OF PUBLICATIONS.....	170

ABSTRACT

A study was undertaken to develop field and laboratory criteria which would allow better prediction of soil fragmentation in tillage, and to evaluate experimental response of structured soil to compression and shear as compared to predictions from scaling theory. A minor objective was evaluation of wetting and drying effects on tilth mellowing.

Results suggest that the classical Mohr-Coulomb criterion commonly used to describe soil failure in tillage is of limited applicability. Rather, soil fragmentation seems to involve a two-stage process of crack propagation, associated with consecutive convergent and divergent soil deformation modes imposed by tillage implements. The extent of fracture is influenced by the strength distribution of structural flaws on the one hand and plasticity on the other, both effects being strongly influenced by soil moisture content. The theory of fracture of elastic/plastic materials is summarized, and utilized in the development of improved laboratory procedures for characterizing soil fracture behavior under conditions of variable plasticity. A procedure based on spectral analysis of penetrometer data is also developed in order to characterize the spatial distribution of aggregates produced by tillage.

In agreement with previous results and predictions of scaling theory, compression and shear behavior of aggregate beds was found to be independent of aggregate size under relatively moist conditions. For as yet unexplained reasons, beds of larger aggregates proved stronger than beds of small aggregates at low moisture content. The degree of aggregate confinement in a bed, affected by the spread in aggregate sizes, was found to markedly influence the resistance of individual aggregates to a given external load. Shear tests on whole soils of different textures showed the cohesion intercept at a given water content to increase with decreasing particle size, while the friction angle remained relatively constant. Both results were in qualitative agreement with predictions of scaling theory, although distortions due to unsimilarity and colloidal activity were evident.

Shrinking and swelling effects alone were found to cause effective tilth mellowing in vertisols. Soils with less clay activity seemed to require additional sources of energy such as raindrop impact in order to achieve effective mellowing. Very sandy soils on the other tended to fall apart under their own weight when wetted.

INTRODUCTION: Project Objectives

This project was initiated in view of the serious deterioration of soil structure observed in many regions and reported in the literature. Structure deterioration has been attributed to the intensive use of increasingly large agricultural equipment, and intensification of tillage and traffic on arable soils under improper soil density and moisture conditions.

The project was essentially an attempt to develop simple, physically based and readily obtainable laboratory and/or field criteria to determine the proper timing of tillage or traffic operations to obtain or maintain a desired tilth. On the assumption that the effective stress principle governs mechanical behavior of unsaturated as well as saturated soils, and assuming applicability of the scaling properties of effective stresses derived by Snyder and Miller (1985) and Snyder (1990) for soils dominated by capillary phenomena, it was hypothesized that soil water content, matric potential and void ratio would be appropriate soil characterization criteria. Consequently the primary objective of the Project was:

1. To quantify the effects of antecedent soil water content, matric potential and bulk density on soil tilths produced by specific tillage or traffic operations in fields of different soil textures and clay mineralogies.

A related secondary objective was:

2. To determine the influence of initial soil bulk density, water content and matric potential on the final bulk density and

hydraulic properties of soil compressed at different loading rates, load magnitudes and compression-shear ratios.

Due to the fact that soil tilths produced in the field change over time, and that the processes involved are still incompletely understood, a third objective was:

3. To quantify changes in soil tilths, natural crusting and plow layer subsidence in these soils with time after tillage as a function of initial size/strength properties of aggregates produced by tillage and by subsequent variations in soil moisture regime.

It was recognized within the context of Objective 1 that, because of the expense and difficulty in controlling variables in field experiments, much of the research for measuring and establishing relationships between soil behavior and the proposed criteria would need to be conducted under controlled conditions in the laboratory. This would require that the laboratory procedures used should correlate well with actual soil field behavior. It was assumed that laboratory tests already published in the literature, in particular the Utomo and Dexter (1983) friability index and the drop-shatter test (Marshall and Quirck, 1950) would be adequate laboratory criteria for characterizing soil fracture behavior in tillage.

Major limitations of these indices became apparent during the early phases of experimentatation. Both indices are quite time and labor consuming, which limits their effectiveness as screening tests for studying effects of many variables. Since their theoretical basis derives from elastic fracture mechanics, applicability to highly plastic soils - an important aspect of this

study - is severely limited. The primary mode of failure in both tests is tension induced by unconfined compressional diametric loading, whereas tillage usually involves shear and compression in addition to tension, under semi-confined loading conditions.

In addition to these difficulties, a more fundamental problem became evident. The Utomo and Dexter friability test and the Marshall and Quirck drop-shatter procedure are load and energy controlled processes, respectively. Tillage, on the other hand, is a displacement-controlled process, where the independent dynamic variable imposed on the system is the displacement path of the soil boundaries imposed by the geometry of the tillage implement. This distinction of tillage as a controlled displacement process is more than academic. As will be discussed later both from theoretical and observational points of view, the ability to move or displace soil elements relative to each other in a controlled manner is a fundamental reason why tillage implements are such effective fracturing devices. Tillage simulation tests which depend on energy or load control thus fail to simulate fundamental characteristics of the tillage process.

During the course of reviewing literature related to these issues, it became evident that the general theory of fracture mechanics, particularly with regard to materials exhibiting significant plasticity, has advanced much further in the past 20 years than seems to be realized by soil scientists. These advances, combined with critical state soil mechanics, offer potential for significant improvement in the general theory and research methodologies associated with soil mechanical behavior under

tillage.

As a result of the above considerations, it was decided by the investigators to postpone Objective 1 for later study. Attention was directed instead to careful field observation of soil fracture characteristics under tillage, and to improving laboratory and field measurements of soil friability. Considerable effort was also devoted to reviewing and summarizing recent literature on theoretical and experimental advances in the fracture mechanics of elastic/plastic materials, and applying some of these principles in preliminary experiments on soils.

Objective No. 2 as stated in the original proposal was maintained essentially unchanged. Emphasis was placed on the mechanical behavior of aggregate systems with different geometric mean diameters but identical log standard deviations. These systems exhibited Miller and Miller (1956) scaling with respect to macroscopic (inter-aggregate) porosity, while intra-aggregate pore-size distribution was constant. A valuable contribution to this phase of the research was made by Dr. T.P. Fyfield under a 6-month Postdoctoral Fellowship sponsored jointly by the British Council and the current BARD Project at the Volcani Center, Bet Dagan, Israel.

Research on Objective 3 was also undertaken essentially as planned, but on a smaller scale.

Following BARD guidelines, each chapter of this Report has been written in the form of a scientific paper. The chapters are grouped as logically as possible according to their relation to Project Objectives. Each chapter was written primarily by the

scientist(s) who conducted the corresponding phase of the research. Two of the chapters have already been published in scientific journals and are included in the form of reprints. A final chapter summarizing results, conclusions and Project achievements is included at the end of the Report.

RESEARCH CONDUCTED UNDER OBJECTIVE 1

CHAPTER 1

FIELD AND LABORATORY STUDIES ON SOIL FRAGMENTATION IN

TILLAGE. I. ISRAEL

Work conducted in Israel by A. Hadas

INTRODUCTION

As mentioned earlier, one of the first steps taken in this Project was to observe tillage operations in the field carried out at different moisture contents. In light of previous studies (Wolf and Hadas, 1987) which suggested soil failure modes other than the classic shear model, special attention was given to evaluating modes of failure and implications for the relevance of friability indices which have been proposed in the past.

THEORETICAL

Application of Mohr-Coulomb soil mechanics principles to tillage predicts that the size of the fragments obtained from the soil bulk by the process are independent of shear-cohesion and are slightly affected by angle of internal friction (Payne, 1956).

Observations published in the literature (Utomo and Dexter, 1981); Hadas and Wolf, 1984, 1987) suggest that the application of these principles to predicting the degree of soil fragmentation by tillage is incorrect. Various factors other than those of the classical soil mechanics are probably involved. Utomo and Dexter (1981) have defined an index of soil friability. This term - "soil

friability" - was defined as the tendency of a large mass of unconfined bulk soil to fragment, crumble and break up into smaller clods and aggregates under stress (Bodman, 1949). Previous attempts were made by others (Christiansen, 1930; Bodman, 1949; Jamison, 1954) who used various methods to define tilth or soil matrix susceptibility to failure under stress (e.g. stress-strain relationships, modulus of rupture) but no useful quantifiable indices of soil fragmentation were evolved even though a better insight to soil failure was attained. Assuming that an hierarchial order of soil structure exists (Brewer, 1964; Hadas, 1987b; Dexter, 1988), it is accepted that the smaller aggregates are the stronger and less susceptible they are to failure under stress. Utomo and Dexter (1981) refined Bodman's definition (Bodman, 1949) that "soil friability" is "the tendency of a mass of unconfined soil to break down and crumble under applied stress into a particular range of smaller fragments". Their definition of "friability" was based on work by Braunack et al. (1979) which was based on Weibull analysis of brittle failure under tensile stress detailed below.

Tensile fracture of a stressed soil sample or matrix can be attained by initiating a crack or activating an existing one and forcing it to elongate and spread into a larger one or form a combination or branching of cracks throughout the soil sample or matrix, unltimately ending in complete fragmentation of the stressed matrix.

Following an hierarchial order of soil structure as proposed by Brewer (1964), and observed by Hadas (1987b) and Dexter (1988),

it is assumed that the soil matrix of a given volume is composed of smaller, sub-volume elements connected together within a volume of material fraction which is large enough to contain a "representative" set of cracks, pores or flaws of various sizes within the sub-elements and the connecting matrix between them. These smaller volumes are made of even smaller sub-volume elements interconnected by flawed, cracked and slightly more porous material than the material from which these subvolume elements are made. This arrangement sequence towards volume diminishing soil subvolume elements or structural units extends from the bulk soil volume to the micro aggregates within it. Hadas (1987b) has shown that the smaller the volume, total porosity and average pore size of a sample, the less cracks or flaws it will have and consequently greater stress will be needed to crack it. In other words, the matrix of a smaller volume exhibits higher tensile strength at failure (Hadas, 1987b). Furthermore, one can assume that if no interaction exists between flaws of the same material, let alone between the connecting matrix and the elemental subvolumes, then stress fields around each flaw are mutually independent and the strength of each soil volume is determined by the stress at which the most severe flaws within it begin to propagate. Hadas (1987a) has shown a relationship between air entry pore radius and tensile strength at failure, an observation which supports this induced assumption and may hint toward air-entry pore diameter of a given soil sample volume, to be the largest flaw of a given sample. If these assumptions are correct; one may follow the same reasoning to

conclude that the strength of a given volume is determined by the strength of the weakest element. By applying the concept of the weakest link, failure of the soil volume under consideration will be determined by localized stresses under which the most severe crack within the connecting material will propagate and form a continuous cleavage plane.

In simple terms these assumptions led to assume that the failure of a soil volume follows a cascading array of failures in which the first ones will be the weakest in strength or the largest in size and then if the stresses is much greater than that required for the first failure, and sufficient to fail the connecting matrix between the next subvolumes, smaller units will fail, an so on. This means that if due to the subvolume elements hierarchy, a stepwise function of increase in exhibited tensile strength with the diminishing soil unit volume exists (Hadas, 1987b), a corresponding stepwise function of degree of soil fragmentation under stress should be expected.

The following analysis is based on the work of Braunack et al. (1979), drawn on from an extensive review by Freudenthal (1968), and Lee (1972). Taking a soil volume element which has an unknown number of pores, cracks or flaws, each of which requires a tensile stress σ to overcome it or cause it to propagate, one may assume that a statistical distribution of critical stresses is associated with the cracks in that soil volume element. The distribution of minimal critical stresses, taking its lowest limits to be equal or greater than zero (no failure due to negative stresses) is given by

$$F(\sigma) = 1 - e^{-(\sigma/\sigma_0)^m} \quad (1)$$

where $F(\sigma)$ is the probability that the smallest critical stress σ is equal or lower than σ_0 , which is the strength of the considered volume element at which $F(\sigma) = 1 - e^{-1} = 0.63$. Both m and σ_0 are material constants for the sampled volume, and $1/m$ is proportional to the scatter of the flaws or crack strengths. Let there be n equal volume elements in a sampled soil volume V . The probability that the minimal critical strength is greater than σ for all n volume elements in the sampled soil volume is the joint probability $[1-F(\sigma)]^n$ of the individual probabilities $[1-F(\sigma)]$ for each individual volume element. Substituting Eq. (1) into this condition gives

$$[1 - F(\sigma)]^n = e^{-[n(\sigma/\sigma_0)^m]} \quad [2]$$

Solving Eq. (2) for the probability $F(\sigma)$ that σ is smaller than σ measured in the i^{th} sample volume (V), and recognizing that the total sampled volume V_0 is equal to n times the individual volumes V (i.e. $V_0 = n V$), one obtains

$$F(\sigma) = 1 - e^{-[n(\sigma/\sigma_0)^m]} = 1 - e^{-[(V/V_0)(\sigma/\sigma_0)^m]} \quad [3]$$

The mean critical stress $\bar{\sigma}$ for a given population of soil volume samples and the variance σ_v are given by the following equations (Braunack et al., 1979)

$$\bar{\sigma} = \int_0^\infty \sigma \cdot d[F(\sigma)] = \sigma_0 (V/V_0)^{1/m} \cdot \Gamma(1 + 1/m) \quad [4]$$

and

$$\sigma_v = \int_0^{\infty} (\sigma - \sigma_0)^2 \cdot d[F(\sigma)] = \sigma_0^2 (V/V_0)^{-1/m} \cdot \Gamma(1 + 1/m) \quad [5]$$

where Γ is the tabulated Gamma function of the bracketed arguments.

Both σ at failure and V , as well as $\bar{\sigma}$ and σ_v , are easily determined for given populations of i sampled, similar volumed soil fragments.

There is still a need to calculate m for a given population. This value, together with $\bar{\sigma}$ and σ_v , permits calculation of the characteristic parameters σ_0 and V_0 . To determine m , fracture strengths are measured on i samples of equal volume V . The measured strengths are then grouped in ascending order to form a cumulative strength distribution. The k^{th} value in this distribution is assigned a cumulative frequency

$$F(\sigma_k) = k/(i+1) \quad [6]$$

Subtracting both sides of Eq. [3] from one and taking natural logarithms twice yields

$$\ln(-\ln[1-F(\sigma_k)]) = \ln(V/V_0) + m \ln(\sigma_k/\sigma_0) \quad [7]$$

Substituting Eq. [6] into Eq. [7] and rearranging gives

$$\ln(-\ln[1 - k/(i+1)]) = \ln[V/(V_0\sigma_0^m)] + m \ln \sigma_k \quad [8]$$

Equation [8] shows that the slope of a regression of $X = \ln \sigma_k$ vs. $Y = \ln(-\ln[1 - k/(i+1)])$ is equal to the parameter m .

Once m is known, equations [4] and [5] can be rearranged to yield the characteristic stress σ_0 and volume V_0 for the aggregate population as

$$\sigma_0 = \sigma_v/\bar{\sigma} \cdot [\Gamma(1+1/m)] / [\Gamma(1+2/m) - \Gamma^2(1+1/m)] \quad [9]$$

and

$$V_0 = V \cdot ([\bar{\sigma}/\sigma_0][1/\Gamma(1+1/m)])^m \quad [10]$$

Consider $j = 1, 2, \dots, J$ groups of aggregates of different sizes, with k aggregates in each size group. If regressions corresponding to Eq. [7] are run for each size group, a family of J parallel lines will be obtained which will be shifted to the left (negative X-direction) by an amount equal to $\ln(V_{1+j}/V_j)/m$ for each volume increment. Rearranging Eq. [4] and taking the natural logarithm yields a relationship between $\bar{\sigma}$ and V ,

$$\ln \bar{\sigma}_j = 1/m \ln V_j + \ln [\sigma_0 V_0^{1/m} \cdot \Gamma(1+1/m)] = A + k \ln V_j \quad [11]$$

The parameter $k = 1/m$ was termed by Utomo and Dexter (1981) as "soil friability" and is a measure of the strength dispersion of flaws or cracks within a sampled soil volume V_0 which consists of the various subelemental volumes $V_{j=1,2,\dots,J}$ and has a characteristic tensile strength σ_0 .

The term $A = \ln V_j + \ln [\sigma_0 V_0^{1/m} \cdot \Gamma(1+1/m)]$ of Eq. [11] is the intercept value which yields the value of the extrapolated natural logarithm of tensile strength of a bulk soil volume of 1 m^3 (when IS units are used).

A large (very negative) slope value, k , signifies marked differences in internal strengths of the various arrays of aggregates of diminishing sizes, i.e. high internal strength of the subunits in contrast with low strength sub-volumes of the connecting matrix between them. In other words, the strength-size order between soil structure units becomes hierarchially more distinct. Stressing of such a soil by a tillage implement will in

principle cause it to crumble into smaller, internally stronger units.

Soils having small k values, on the other hand, present small differences in their internal strength. One would expect that tillage of these soils should present a narrower range of aggregate sizes.

It is important to note that the parameter k provides no information on average strength of the aggregate population, nor on the arithmetic difference between strengths of aggregates of different sizes. It is rather an indicator of how strength ratio varies with size ratio, through the relation $\sigma_1/\sigma_2 = (V_1/V_2)^k$. Thus, one can have the same k value for a population of aggregates which are all extremely strong, and for a population of very weak aggregates. Under the "friability" hypothesis, it is assumed that the degree of fragmentation under a given tillage implement should be the same for both systems. This requires that in tillage, the stresses required to fragment all subunits will be applied by the tillage implement independently of how high these stresses may be.

This assumption seems strongly questionable. Once the weakest links (between largest aggregates) have yielded, the resulting large fragments begin to move as free bodies (Wolf and Hadas, 1987). Because of the largely unconfined movement of these bodies after initial fracture, it is likely that stresses within them will never reach very high values. Whether or not further fracture of these fragments can occur has nothing to do with the k value, but

rather with how many points within the fragments are weak enough to fail under the limited stresses achievable in a free body condition. If anything, the A value of Eq. [10] would seem to be more relevant to this situation than the k value.

There are other reservations concerning use of the Utomo and Dexter friability index for estimating field aggregate size distributions obtained by tillage; (1) when tilling the soil most of the affected volume is partially or even completely confined whereas the Utomo and Dexter test involves tensile stressing under unconfined conditions; (2) large sub-volumes of the tillage-stress affected volume are compressed and sheared rather than subjected to tensile stress; (3) when moist soil is stressed then plastic deformation rather than tensile failure occurs; and (4) process such as plasticity that will alter the strength distributions of internal failure zones will change the soil friability.

These considerations point out the need for further field testing of Utomo and Dexter's (1981a) proposed friability index. This can be achieved provided the analysis is performed at the same soil water contents as tillage, and complementary analyses of aggregate size distribution prior to and after tillage are performed.

The simplest method to obtain aggregate and clod size distributions is by sieving a large volume of fragmented soil. There are various methods by which the sieved material weight-size data sets are analysed (Mazurak, 1950; Van Bavel, 1950; Gardner, 1956). Of these, the log-normal method proposed by Gardner (1956)

is usually preferred. The experimental procedure is rather simple; a sample is sieved through a sequence of screens of narrow size distribution differences. The retained material on each sieve is weighed and its weight fraction is calculated. The sieve diameter is plotted on the log scale of log-probability paper, and the fraction of aggregates smaller than a given diameter is plotted on the probit scale. The geometric mean diameter M_w is taken as the diameter corresponding to 50 percent probability, and the geometric standard deviation σ_g^* is calculated as the diameter read at 84.3% probability divided by the diameter at 50% (or the diameter at 50% probability divided by the diameter at 15.87% probability (Gardner, 1956)). This analysis is based on equal density of all aggregate sizes. In the case of fluctuations in aggregate densities, the corresponding weight fractions would need to be normalized to a common density.

EXPERIMENTAL

Field observations and trials were carried out on five sites. Soil physical and chemical characteristics from these sites are given in Table 1. The data were collected from experiments carried out by other groups (e.g. fields No. 1 and 2), from previous field trials carried out by this group and experiments specifically conducted under this Project. The studies were for the most part conducted on communal farms having the tillage equipment needed.

A. Observations in Field No. 1^{1/}

On a clayey Vertisol (Typic Chromoxerert), shallow ploughing operations to an average depth of 25 cm were performed during two successive years. The first year's tillage was conducted on an air-dry plot ($P_w = 6.7 \pm 1\%$ w/w), and a moist-wet plot ($P_w = 21.5 \pm 4.2\%$) after cumulative rainfall of 230 mm. Tillage in the second year was on a moist plot ($P_w = 14.2 \pm 3.2\%$), and a wet plot ($P_w = 33.2 \pm 2.1\%$) after cumulative rainfall of 343 mm. Energy required for tillage was determined from mean actual tractor field velocities and average draft forces recorded electronically by load cell and data logger at the implement hitch point.

Results and Discussion

Mean depth of ploughing, mean energy required for tillage and ambient soil moisture contents for the two years of observations are given in Table 2.

Although large differences in moisture contents existed between wet, moist and dry plots, no great differences in energy to fragment the soil were detected for any given year. This was despite the fact that quite different tilths were obtained as

¹Data were given by Prof. D. Wolf, Faculty of Agricultural Engineering, Technion, Haifa, Israel, from a Report by Hillel et al., 1968.

illustrated from the first year aggregate size distributions depicted in Fig. 1. Ploughing in air-dry soil produced the largest mean weight diameters whereas the smallest corresponded to the wet soil. The same patterns were observed for the geometric standard deviations. Thus, plowing the dry soil produced large clods with a relatively wide size distribution, whereas tilling the wet soil produced small aggregates with a narrower size distribution.

These results can be interpreted in at least two different manners. The first is from the viewpoint of the friability index k . If attainment of a given degree of fragmentation or aggregate size distribution is sought, then if the apparent "soil friability" is small, a larger amount of tillage work will be required (Utomo and Dexter, 1981b, Hadas, 1987b, 1990 and Hadas et al., 1978). In our case, the same amount of energy was expended to achieve all tilths; thus differences in clod distributions were apparently due to differences in k values. The inference is that there are different crack and pore network patterns at different moisture contents and different years. This is particularly possible in swelling and shrinking soil like the one under consideration.

A second way to look at the results is that due to changing moisture contents (air-dryness to a soil at -98 KPa matric potential equivalent water content), the tensile strength levels diminished throughout the soil volume. Even if the same k value persists for the same two water contents, a greater number of points within the moist soil mass would fall below a "critical strength"

at which failure would occur during the displacement process imposed by the tillage implement. Conversely, stresses at air-dryness could cause failure only between the larger clods where more severe, discernible cracks exist and thus soil units between cracks are torn from their position and moved rather than stressed, sheared and fractured (Wolf and Hadas, 1987).

Due to the fact that these field observations were not supported by aggregate strength analysis, no clearcut conclusions can be drawn on the suitability of the k concept or some other formulation for describing soil friability. These issues were further examined in subsequent field tests.

B. Observations in Field No. 2 (Soil No. 3)

On a vertic, clay soil (Typic aquic-chromoxerert) ploughing tests were carried out. (Data furnished by D. Wolf as for field No. 1).

The field plots were ploughed on slightly moist to wet soil, to depths of 20 to 25 cm. The energy required for ploughing was measured by a dynamometer connected to a data logger and the tractor actual field velocity. Soil tilth obtained was determined by sieving the fragmented soil and calculating the geometric mean diameter ϕ and the geometric standard deviation σ_g . Soil cohesive strength was determined by vane shear, as described for field No. 1.

Results and Discussion

In contrast to the previous experiment (Field no. 1), the energy required for ploughing varied with soil water content in a hyperbolic or exponentially decreasing rate (Table 3 and Fig. 4a). For similar water contents the energy required seemed to be almost the same. In Figure 4 aggregate size distributions obtained during the first year's tests are given as for the first site (field No. 1). As in the previous case, the wetter the soil was the smaller the geometric mean aggregate diameter and standard deviation were. Thus, the wetter the soil was the more apparently friable it was.

In Figure 7 the mean tensile strength of air-dry aggregates of different size groups sampled from soil tilth obtained at different initial soil moisture contents are given. The strength values measured for the smaller aggregates produced under wet conditions are slightly lower than those produced under the moist to dry conditions. For all other aggregate sizes, aggregates produced under wet conditions were stronger than those produced under dry conditions.

Correlative analysis of the natural logarithm of the mean tensile strength ($\ln \bar{\sigma}$) and the natural logarithm of aggregate volume ($\ln V$) shows the aggregates of the wetter soil to have lower k values but still within a class termed as "friable" by Utomo and Dexter (1981b). The intercept values A suggest that, provided the bulk soil had the same properties as the sampled aggregate population, the tensile strength of the bulk soil increased by 19 kPa (32.8%) when tilled wet ($P_w = 28.4\%$ w/w) as compared to that of

the bulk soil when tilled at the lower water content (16.3%). These observations allow a certain insight into the strength distribution of aggregates produced by tillage at different soil moisture contents. However, the k and A values obtained cannot be used as indicators of friability since they were determined after tillage. In order to adequately represent soil conditions at time of tillage, soil friability indices must be determined immediately prior to tillage, and the development of a special methodology is required in order to sample the soil accordingly and break it in a proper, normative way to produce repeatable results. Such a procedure was examined in the following field trial.

C. Loessial Soil Field No. 3 (Soil No. 1)

In this experiment the possibility of determining "soil friability" prior to tillage on a loessial silty loam soil (Typic Calcic Haploxeralf) was examined.

Deep ploughing was performed by "Ghirardi" type plough, four bottoms, to a depth of 30 to 35 cm on a "dry" plot (3 to 5% Pw top to bottom of furrow) and on a "moist" plot (8% to 14% Pw top to bottom of furrow). The approximate soil water matric potentials at these water contents were - 1470 and - 98 kPa, respectively.

Prior to tillage two large soil monoliths (50 to 60 kg each) were dug out carefully to a depth of 30 cm by spades from each plot. Aggregates were broken carefully, then sieved and crushed in an unconfined compression cell (Soil Test Model No. U-160-A). Tensile stress σ at failure was determined for the aggregates

following Hadas and Lennard (1988), using the equation

$$\sigma = 0.711 \frac{4 F}{\pi d^2} \quad [12]$$

where F is the compressive force at failure and d is the aggregate diameter across which stress was applied.

Energy expenditure to till the soil was calculated from measured draft forces and tractor field velocity for both plots as described before. Field aggregate size distributions were determined by sieving after the obtained soil tilth had reached air dryness, in order to minimize further diminution of aggregate sizes during sample handling and sieving.

Results and Discussion

Plow depth, energy expenditures and soil moisture contents are given in Table 4, and calculated "soil friability" values for the soil samples prior to tillage are given in Figure 8.

Energy requirements for tillage were practically the same at both water contents. These results are similar to those observed in Field no. 1, but differ from those in Field no. 2 where tillage energy decreased significantly with increasing water content.

Correlating the $\log_e$ of the mean tensile stress $\bar{\sigma}$ vs $\log_e$ of the mean volume V of the aggregate size classes following equation [10] yielded the "k" values, or "soil friability" indices. The indices were practically the same (0.167 ± 0.021 and 0.191 ± 0.022) for the "dry" and "moist" plots, respectively. The intercept value "A" showed a slightly reduced tensile strength for the moist bulk soil.

The negligible differences in k value do not seem very predictive of the completely different aggregate size distributions obtained by tillage as shown in Fig. 9. One should seek an explanation for these "discrepancies" from the hypotheses that the k-value is an indicator of field soil friability. This is particularly true in view of the fact that the site chosen was on a non-cracking soil, selected specifically to avoid over-cracking of the soil bulk which would invalidate the friability test predictions.

E. Field experiment No. 4 (Soil No. 4)

This experiment was carried out to assess the effects cracks have on tillage (plowing) efficiency on dry and moist plots.

On a clay sandy soil (Typic Rhodoxeralf), the physical and chemical characteristics of which are given in Table 1, cracking patterns on air-dry plots after wheat were measured and photographed. The horizontal dimensions of visible soil structure units were determined from the photographs. The vertical extent of the observed cracks (e.g. depth) could not be determined

The plots were then irrigated by sprinklers to a rainfall equivalent of 140 mm of water, then one week later the field was trafficked to evaluate soil compaction by heavy tractors.

The compacted tracks (8 m apart) were marked and the field was ploughed to a nominal depth of 30 cm with a "Ghirardi" moldboard plow of five 16" bottoms and a total width of 2m. The largest clods

were measured (in three principal dimensions) and weighed. The remaining material (smaller than 50.4 mm) was sieved and the separates were weighed. Soil moisture distributions, resistances to cone penetration and bulk densities prior to tillage were taken in three transects perpendicular to a line of sprinklers, across a plot of 24 m width (the transects were 8 m apart). Shear strength of the apparently uniformly wetted area was determined by a shearograph (Soiltest model D-250X) at two depths prior to and after tillage.

On a dry, adjacent field (bare after tomato), crack distribution was measured in a similar way, and the aggregate sizes produced by the same plow were determined following the procedures listed above.

On a third field on the other side of the first (bare after wheat), next to the wet-irrigated plot, an inclined parabolic chisel (5 inclined, parabolic times, with a total width of 2.0 m) was used to chisel-plow the area to a nominal depth of 40 cm. The draft forces were recorded and the peaks were determined in order to find any harmonic modes pertinent to the vertical dimensions of the clods recorded by photographs. The inclined, parabolic chisel plough used was similar to "Howard's" paraplow which affects the soil by tensile stressing it.

Results and Discussion

Soil resistance to cone penetrometer increased with soil depth and distance from the last sprinkler (due to decline in water

content) as expected from changes in soil water contents and bulk densities given in table 5, b, d. Shear strength did not differ with depth as given in Table 5c.

Measured dimensions of clods prior to and after wetting and ploughing are given in Table 6. The largest clods observed on all plots (wet and dry) after plowing were of similar dimensions, even though the observed crack distributions prior to plowing were not always the same. Thus, one should consider the existence of the similar patterns of "weakest flaws", which may or not coincide with the observable (pre-tillage) crack pattern caused by the crop and soil wetting state. The extent to which the size-scale of the tillage implement controls the size of largest clods, and the physical characteristics (i.e. fractal nature ?) of crack networks which would allow such control, should also be considered. A similar question arose from field tillage results in Puerto Rico, as will be seen in Chapter 2 of this Report.

Aggregate size distribution after tillage was much more influenced by soil moisture content when the smaller (< 50 mm) aggregates were considered. The geometric mean diameter (including large clods) diminished from 274 mm in the dry plots to 22 mm in the wet treatment, while the corresponding geometric standard deviations decreased from 68.5 to 12.2. This means that by wetting the cracked soil, a greater degree of fragmentation of the fine fraction (< 50 mm) was obtained. The fine fraction was about 55% and 38% of the total tilled soil mass for the wet and dry treatments, respectively.

These patterns suggest that on both dry and wet plots a plow first "cuts" and tears (separates) the large, existing soil units bounded by the cracks and depth of ploughing. Further breakup is caused by stressing these units to failure on wet, weaker structure; and by self-abbasive size diminution of very strong units or fracture at weaker cracked sections in dry, very hard soils. The higher percentage of surviving large clods on the dry plots probably can be attributed to the fact that the stresses applied were equal or smaller than those required to cause the clods to fail; whereas stresses applied to the wet clods cause further breakdown by failure of the water-weakened soil matrix. Note again that the degree of soil breakup appears to be influenced, at least in part, by the distribution of locations within the soil mass which are weaker than a given (absolute) strength threshold, a mechanism which is unaccounted for in the k-index friability concept.

Measured mean draft forces for ploughing the dry and wet plots with the Ghirardi plow were 86,600 KN and 75,800 KN, yielding an average specific resistance to tillage (defined as draft force per unit area of tilled soil) of 138 and 102 KPa, respectively. For the inclined-parabolic chisel the draft force was 70,500 KN, and the corresponding specific resistance to tillage was 117 kPa. These almost equal specific resistances to tillage suggest that the above proposed processes of cutting, moving, reorienting and slightly failing the soil clods may be, or is, the right sequential pattern of occurrences in the tillage process as suggested by Wolf and

Hadas (1987).

F. Field No. 5 (Soil No. 5)

On a non-shrinking sandy clay loam (Typic Rhodoxeralf), a tillage experiment was carried out to reassess whether the fragmentation process there resembles the friability concept or whether it follows same sequential processes mentioned above for tillage in a cracked soil. The sensitivity capabilities of various methods to define the tilth obtained by tillage were also examined.

Materials and Methods

Tillage operations were carried out at three soil moisture contents on a sandy loam soil (Typic Rhodoxeralf), after two successive years in wheat as a forage crop . Two tillage treatments were used: duck-foot cultivator operated to a nominal depth of 15 to 20 cm, and rototiller to a depth of 20 cm. Soil moisture distribution with depth, vane shear cohesion, aggregate size distribution and tensile strength at air dryness were determined, using the same procedures as described before. No apparent cracks could be discerned even at low water contents or air dryness.

Results and Discussion

Bulk density and water content distributions for the sub-plots prior to tillage and soil cohesion values with depth prior and post tillage are given in table 7 (a) and (b). Cohesion and

bulk density were reduced to the greatest extent by cultivation with the duckfoot cultivator, which produced larger soil units and a wider aggregate size distribution than the rototiller (Table 8 and 9). The higher bulk density values under the rototiller compared to those under the cultivator, are apparently due to the better spatial arrangement (in vertical direction) of a narrower size range of aggregates so as to allow a more compact aggregate bed.

The values of the geometric mean aggregate diameters ϕ_m and standard deviations σ_g in Table 9 correspond to the entire tilled layer. However, it becomes apparent that these two parameters are insufficient to completely characterize the aggregate bed when different layers within the tilled horizon are examined. This is illustrated in Fig. 12, where aggregate size distributions for different depths are presented. Both the geometric mean diameter and geometrical standard deviation tend to increase with depth. Thus, a coarser tilth with a wider aggregate size distribution exists at the bottom of the tilled layer. This seems to imply that larger aggregates and clods were left at the bottom of the tilled layer and fine tilth "trickled" down into the same zone.

These observations suggest that when implement performance is to be evaluated, both sampling methods - sampling by layers and the whole tilled depth - are required in order to fully characterize the tilth produced. Further spatial resolution could be obtained by referencing points of tilth measurement with respect to forces superimposed in the soil volume by the tillage implement; for

example, measuring tilth at the position where a cultivator shank passed through the soil vs. at the mid-point between shanks. These same considerations apply not only to aggregate size distributions but also to other tilth-qualifying criteria such as water content, bulk density and aggregate strength distributions.

Analysis of tensile strength characteristics was performed on air-dry aggregates sampled prior to and after tillage by cultivator and rototiller at different soil water contents. Tensile strengths for different aggregate sizes, as well as characteristic tensile strengths σ_0 and characteristic volumes V_0 , are given in Table 10. Figure 14 shows two sample regressions between $\ln [-\ln(P(\sigma))]$ and $\ln \bar{\sigma}$ of aggregates of two size classes (2-4 and 25.4 to 50.8 mm). The data given in Figure 14, typical to all sets analyzed, show a rather good fit ($r^2 = 0.974$ and 0.887 for the large and small aggregates, respectively).

The mean tensile strength $\bar{\sigma}$ for air dryness in Table 10 show that tillage in general had no effect on the internal strength of the aggregates produced. The standard deviations are so large that practically no significant differences can be pointed out, but only a trend. Except for the 3 to 4 mm aggregate size, under all circumstances aggregates produced by rototilling were of the same strength or only slightly weaker than those produced by the cultivator. Under all cases tensile strength diminished with an increase in aggregate size.

The mean volume characteristic strength σ_0 was nearly constant for all aggregate sizes and moisture contents, except for the

tilled, smallest aggregate sizes where a decrease in both size and moisture content increased the aggregate strength (these data and those for the mean characteristic volume element V_0 were calculated from pooled replicate data). The calculated mean characteristic volume elements V_0 varied within the range of 5/8 to 1/10 of the mean aggregate volume. This suggests that the analyzed aggregates, selected at random from a rather large particle population, represented a rather uniform infrastructure within the analyzed aggregate population.

All these strength observations lead to the conclusion that whatever was done to these aggregates by tillage caused them to shed a part of their volume, probably at their edges, rather than fail. In other words, no intra-aggregate tensile failure occurred, but rather diminution by abrasive friction between aggregates and inter-aggregate failure by tension, shear or compression. Under these circumstances, calculations of A (e.g. the tensile strength of a unit volume of 1 m^3) should be almost the same independently of whether the average aggregate tensile strength $\bar{\sigma}$ or the characteristic strength σ_0 is regressed against either V or V_0 . These regressions are given in Table 11 a,b,c. The intercept values A for all these regressions are practically the same, varying only between 0.27 and 3.68 KPa. This result strongly supports the conclusion that failure generally occurs between aggregates or within large clods but not within aggregates.

Summary and Conclusions:

Results from this study support the concept suggested by Wolf and Hadas (1987) that the maximum stresses and corresponding energy expenditures achieved in tillage are largely a function of the weakest flaws in the soil, namely zones of weakness between the largest soil units. Stresses developed in the soil mass will be no larger than those required cause these units to separate from each other and begin moving as semi-confined independent bodies. These "threshold" stresses will vary with the type of loading (i.e. tension/compression/shear ratios imposed by the geometry of the tillage implement), and with soil properties such as strength of the zones of weakness and soil plasticity. The extent of fracture within these large structure units is largely a function of the spatial distribution of internal flaws which posses failure strengths below the threshold stresses. In dry soils where effective stresses are high, the rupture strength of most of the internal flaws is above threshold, resulting in minimal fracture of the primary soil units. However, more fragmentation would be expected in moist soils where effective stresses are lower and a greater proportion of the internal flaws are below threshold. This direct relation between fragmentation and moisture begins to deteriorate in very wet soils due largely to plasticity which causes sealing and subsequent strengthening of flaws during the course of deformation.

Such a model of fragmentation is inconsistent with the friability concept proposed by Utomo and Dexter (1981) and which

has received wide acceptance in recent years. The critical parameter in this theory is the "k-value" which relates the tensile strength ratio of two aggregates to the corresponding aggregate size ratio. It is totally independent of the absolute strength of the aggregates, which divorces it entirely from any situation where fragmentation is governed by an absolute stress threshold such as that postulated above.

Furthermore, the k-value yields no information on the spatial frequency of cracks in a given size range; an aggregate with one crack of a given size will fail at the same stress as a macroscopically identical aggregate with ten cracks of the same size, provided the cracks are far enough apart that stress interactions between them do not occur. This situation would occur, for example, in two mediums with very different cracking intensities but the same average crack size and standard deviation. The k-values (and A values as well) would be the same in both systems, thus predicting the same degree of fragmentation under a tillage situation. The threshold model, on the other hand, would predict greater fragmentation of the intensely cracked system, which seems much more in tune with field observations. These theoretical considerations regarding limitations of the k-value as a friability index were supported by limited field data, which showed only small variations in k for soil conditions which yielded very different tilths under field tillage situations.

It is interesting to note that, if the two hypothetical media discussed above were to fracture at all their zones of weakness,

the result would be two aggregate populations with different geometric mean diameters (a function of cracking intensity) but identical geometric standard deviations. Such similarities in aggregate populations are not at all uncommon, a fact which suggests that major differences in fracture patterns may often result from "similar" cracking patterns which differ primarily in their "scale" (intensity) while remaining similar in their relative size distributions. More data in support of this possibility will be discussed within the context of field experiments conducted in Puerto Rico (Chapter 2)

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Table 1: Characteristics of soils on field sites used for observations and field measurements¹⁾

USDA soil designation	% on w/w basis				Water content % on w/w basis				Proctor (mg·m ⁻³ /Pw)	Specific surface area (m ² ·g ⁻¹)
	Clay	Silt	Sand	Organic matter	CaCO ₃	Matric potential (kPa) -32 -98 -1470	Plastic limit	Liquid limit		
Calcic Haploxeralf	17.3	28.3	54.4	0.7	11.4	17.3 13.9 7.8	20.7	31.4	1.821/14.8	98
Typic Chromoxerert	35.2	12.1	52.7	1.2	13.6	26.1 21.2 14.8	24.0	46.6	1.572/21.8	203
Typic Chromoxerert	46.9	15.6	37.5	0.9	15.1	28.5 22.9 18.5	25.5	48.8	1.590/22.6	209
Typic Rhodboxeralf	27.5	18.6	53.9	-	18.4	25.2 21.7 11.7	-	-	-	-
Typic Rhodoxeralf	11.3	10.7	78.0	0.9	-	9.4 6.3 4.2	-	20	1.838/12.4	58

1) Data given by (1) J. Dan of the Div. of Soil Pedology, Institute of Soil and Water, ARO and (2) unpublished sets measured by C. Smith and A. Hadas.

Table 2: Depth of ploughing, energy to till, water content at tillage for two years are given for Revadim site¹

	1st year		2nd year	
	dry	moist-wet	slightly moist	wet
Depth [cm]	20	20	30	30
Energy for tillage [kg·m·m ⁻²]	2209±293	2070±358	1597±451	1759±118
P _w (w/w%)	6.7±2.1	21.5±4.2	14.4±3.2	33.2±2.1

¹ Data were given by Prof. Dan Wolf, Technion, Haifa.

Table 3: Depth of ploughing, energy to till, water content at tillage for two years for Gan Shmuel Site¹.

		1st year		2nd year	
		dry	wet	dry	wet
Tillage depth [cm]	-	20	20	25	25
Energy for tillage	-	2143±364	1793±249	2861±321	126
	[kg·m·m ⁻²]				
Water content at	-	16.3±3.7	28.4±3.1	12.1±2.4	27.2±3.4
time of tillage					
P _w (%w/w)					

¹ Data given by Prof. Dan Wolf, Technion, Haifa, were partly given by Hillel et al. (1969).

Table 5: Water contents, soil resistances to penetrations as functions of position and depth (field 4).

Distance [m]	Across irrigated field between		Distance [m] from last sprinkler			
	sprinklers	1	4	6	9	15
Depth [cm]	[a] <u>Mean soil resistances to cone penetrometer [kPa]</u>					
0 - 5	227±144	167±121	233±124	283±194	450±176	810±275
5 -10	554±292.1	417±231	680±214	960±315	960±484	4220±1930
10 -15	1034±572	920±624	1160±420	1820±930	2302±1243	5980±1980
	[b] <u>Gravimetric water contents Pw (w/w)</u>					
0 - 5	27.7±1.4	25.3±1.2	-	24.8±1.4	-	23.9±1.5
5 -10	24.6±2.1	24.3±1.1	-	23.8±1.8	-	19.7±3.1
10 -15	23.6±2.4	23.8±1.2	-	22.9±1.1	-	18.4±3.6
	[c] <u>Shear strength [kPa]</u>					
0 - 2	$32.5(\pm 5.20) + 0.484(\pm 0.068)\sigma_n^*$					
10 -12	$34.7(\pm 13.7) \pm 0.436(\pm 0.173)\sigma_n^*$					
	[d] <u>Bulk density [mg ÷ m⁻³]</u>					
0 - 5	1.284 ± 0.094					
5 -10	1.379 ± 0.063					
10 -15	1.398 ± 0.048					

* normal stress [kPa]

Table 6: Measured dimensions of ^{large} clods as seen from above prior to ploughing and after [cm $\pm$ s.d.]

Pre-tillage	Ploughed	Chisled dry	Remarks
natural			
82.3($\pm$ 14.1)*43.1($\pm$ 10.3)	35.8($\pm$ 4.1)*28.3($\pm$ 13.1)	48.4($\pm$ 10.7)	¹ (50-60%w/w)
(Ploughed wet & dry plots)	*21.5($\pm$ 2.3)	*36.7($\pm$ 8.6) *32.4($\pm$ 8.9)	After wheat (wet soil)
45.4($\pm$ 7.8)*31.4($\pm$ 6.3)	38.2($\pm$ 5.3) *27.2($\pm$ 5.4) *22.3($\pm$ 4.1)	-	² After tomato (dry soil) (70-80% w/w)
	35.2($\pm$ 7.8)*20.3($\pm$ 5.6) *16.2($\pm$ 2.6)	-	(65-75% w/w)

¹ The weight percentage of clods larger than 50 mm is based on measurements (see for instance pictures and figures Nos. 10 and 11 for sieved material).

² The "natural" clod dimensions were taken at random across the field, as the first set of ploughed clods. The second set below the first one was taken from a sector lighter in colour than the remainder of the field.

Table 7a: (a) Bulk density, water content P_w prior to tillage and
(b) cohesion values pre- and post- tillage (field 5).

(a) Depth cm	Bulk density [$\text{mg} \cdot \text{m}^{-3}$]	Water content [% w/w] P_w^1		
		<u>"wet"</u>	<u>"moist"</u>	<u>"dry"</u>
0 - 5	1.24 ± 0.04			
5 -10	1.50 ± 0.03	8.1 ± 1.3	2.3 ± 1.4	1.8 ± 0.6
10-15	1.58 ± 0.05			
15-20	1.64 ± 0.03	9.0 ± 2.4	5.7 ± 2.2	3.2 ± 1.4
20-25	1.60 ± 0.04			
25-30	1.58 ± 0.03	8.6 ± 1.8	6.8 ± 1.9	4.4 ± 1.6
30-35	1.63 ± 0.04			
40-45	1.61 ± 0.03	8.4 ± 1.9	8.3 ± 2.2	5.1 ± 1.6

(b) Cohesion [kPa] by vane shear

[cm]	----- Cultivator -----				-----Rototiller-----		
	<u>"moist"</u>	<u>"wet"</u>	<u>"moist"</u>	<u>"dry"</u>	<u>"wet"</u>	<u>"moist"</u>	<u>"dry"</u>
	<u>pre-tillage</u>						
5	42.1 ± 12.7	3.8 ± 0.6	3.4 ± 0.7	4.0 ± 1.3	7.3 ± 4.2	7.8 ± 3.9	14.4 ± 5.2
15	88.8 ± 23.7	8.0 ± 2.1	7.8 ± 1.7	9.6 ± 4.0	19.2 ± 5.9	18.2 ± 6.1	36.3 ± 9.4
25	114.1 ± 21.8	12.9 ± 3.5	14.3 ± 4.7	21.7 ± 8.6	43.6 ± 7.4	45.3 ± 9.2	69.4 ± 21.6
35	110.2 ± 22.4	-	117.4 ± 31.4	-	-	112.8 ± 31.4	-
45	134.6 ± 13.9	-	138.2 ± 14.7	-	-	133.6 ± 19.8	-

¹ "Wet" is approximately soil moisture content at "field capacity",
"moist" varied from "air-dryness" at the surface to field capacity at
40 cm depth and "air dryness" from air dry water content to about
-500 kPa matric water potential.

Table 8: Bulk density as a function of water content and tillage implement (field 5).

Depth [cm]	<u>Cultivator</u>			<u>Rototiller</u>		
	Bulk density [mg.m-3]					
	"wet"	"moist"	"dry"	"wet"	"moist"	"dry"
0- 5	0.785±0.085	0.820±0.031	0.665±0.59	0.957±0.59	0.937±.157	0.985±.051
5-10	0.747±0.139	0.913±0.036	0.767±0.053	1.049±0.057	1.024±.139	1.081±.058
10-15	0.751±0.042	0.978±0.018	0.834±0.045	1.261±0.127	1.117±.119	1.279±.065

Table 9: Aggregate size distribution parameters: weighted-volume median diameter ϕ_m , weighted volume median diameter ϕ_{sv} as affected by tillage implement and water content. ϕ values are given in the following order $\phi_m(\pm s.d.)$; $\sigma(\pm s.d.)$ and ϕ_{sv}

	"Dry"	"Moist"	"Wet"
(a) $\phi_m((\pm s.d.); \sigma_g(\pm s.d.))$			
Pretillage	46.0 $\pm$ ($\pm$ 7.0);47.4($\pm$ 9.4)	-	-
Cultivator	12.0($\pm$ 3.7);9.3($\pm$ 2.1)	4.6($\pm$ 1.7);9.0($\pm$ 3.7)	9.6($\pm$ 2.4);8.0($\pm$ 3.1)
Rototiller	9.1($\pm$ 1.3);3.8($\pm$ 2.1)	4.2($\pm$ 1.2);3.5($\pm$ 0.8)	5.6($\pm$ 2.1);3.3($\pm$ 1.4)
(b) $\phi_{s.v.}$			
Pretillage	0.21	--	--
Cultivator	1.0	0.41	1.12
Rototiller	3.72	1.93	2.82

Table 10: Mean tensile strength at failure $\bar{\sigma}$ [kPa], characteristic strength σ_o [kPa] and characteristic volume element V_o [$m^{-3} \cdot 10^8$] for aggregates obtained prior to and after tilling the soil at different water contents.

a) Mean Tensile Strength $\bar{\sigma}$ [kPa]

Moisture State	Aggregate size classes [mm]					
	2.0-4.0	4.0-5.6	5.6-8.0	8.0-11.2	11.2-16.0	16.0-25.4
Tillage						
Prior	Air dryness 583.5±543.6	265.7±215.4	180.5±95.9	149.1±170.6	113.5±83.9	62.2±57.6
						40.9±53.2
						37.5±34.1
Cultivator	Air dryness 263.4±214.8	265.9±196.2	224.4±206.8	142.9±179.0	123.4±114.3	42.2±30.4
	Moist 415.4±476.4	231.9±243.9	235.8±124.1	151.3±87.5	124.5±68.4	86.6±69.1
	Wet 520.4±351.5	296.6±198.9	209.1±198.2	158.3±180.5	124.9±81.5	72.1±54.7
						34.4±27.5
						13.9±8.6
Rototiller	Air dryness 373.0±424.9	166.3±96.0	205.5±193.5	113.3±79.9	49.6±32.0	48.6±30.8
	Moist 320.4±192.8	205.5±213.5	165.3±95.9	45.6±33.4	48.5±30.8	49.9±24.1
	Wet 584.1±759.6	266.8±248.9	175.6±115.6	40.4±29.3	32.1±21.3	47.9±48.4
						43.2±26.8
						13.9±10.7
						11.8±7.6
						19.1±7.1

b) Mean Volume Characteristic Strength σ_o [kPa]

Prior	Air dryness 564.8	441.3	184.3	166.9	138.8	131.5	70.1	58.0
Cultivator	Air dryness 347.4	378.8	327.3	212.4	163.7	48.8	41.8	19.7
	Moist 583.4	341.2	301.0	180.5	167.2	113.3	55.4	26.6
	Wet 899.3	564.3	399.3	204.8	125.7	95.4	65.9	19.3
Rotofiller	Air dryness 433.7	236.6	233.4	125.5	88.3	74.9	57.7	10.9
	Moist 787.2	258.9	288.8	72.3	121.6	73.4	63.5	13.8
	Wet 982.9	433.9	228.3	46.1	38.2	72.0	70.7	23.5

c) Mean Characteristic Volume Element V_o [$m^{-3} \times 10^8$]

Mean Volume $\bar{V} \cdot 10^8$	1.2	5.5	15.7	33.4	100.1	323.0	2504.0	-
Prior	Air dryness 0.74	0.66	6.28	12.69	40.0	71.1	1452.3	-
Cultivator	Air dryness 0.55	0.38	7.24	9.02	25.72	129.3	386.6	-
	Moist 0.18	1.93	7.31	10.32	36.05	106.7	641.5	-
	Wet 0.26	0.99	2.67	12.03	50.13	119.5	500.4	-
Rototiller	Air dryness 0.47	1.67	6.96	15.50	20.33	85.9	489.4	-
	Moist 0.12	0.92	3.82	2.44	21.07	125.3	750.3	-
	Wet 0.36	1.59	5.65	14.70	42.04	113.1	626.0	-

Table 11a,b,c Coefficients of the regression equation (eq. [eq.2])

$\ln \bar{\sigma} = A + B \ln \bar{V}$ and $\ln \sigma_0 = A' + B' \ln \bar{V}$ for the mean tensile strength. $\bar{V}$ - geometric mean volume of an aggregate size class.

a) $\ln \bar{\sigma}$ vs $\ln \bar{V}$					
Prior	air dry	=	$\begin{matrix} A & B \\ -0.0617 & -0.34267 \end{matrix} \ln V$	$r^2 = 0.9793$	$\begin{matrix} A \text{ kPa} \\ 0.940 \end{matrix}$
Cultivator	air dry	=	$\begin{matrix} 0.6191 & -0.28495 \end{matrix} \ln V$	$r^2 = \begin{matrix} 0.860 \\ 0.9765 \\ 0.9962 \end{matrix}$	1.943
	moist	=	$\begin{matrix} 0.6191 & -0.29791 \end{matrix} \ln V$		1.8572
	wet	=	$\begin{matrix} -0.13653 & -0.35044 \end{matrix} \ln V$		0.8723
Rototiller	air dry	=	$\begin{matrix} -0.32337 & -0.33675 \end{matrix} \ln V$	0.9104	0.7237
	moist	=	$\begin{matrix} 0.52674 & -0.273199 \end{matrix} \ln V$	0.7054	1.6934
	wet	=	$\begin{matrix} -0.837555 & -0.36863 \end{matrix} \ln V$	0.6981	0.4349
b) $\ln \sigma_0$ vs. $\ln \bar{V}$					
Prior	air dry	=	$\begin{matrix} A' & B' \\ 1.302804 & -0.269943 \end{matrix} \ln V$	$r^2 = 0.9133$	3.6796
Cultivator	air dry		$\begin{matrix} 0.20871 & -0.3332 \end{matrix} \ln V$	0.8555	1.23097
	moist		$\begin{matrix} 0.8751 & -0.301527 \end{matrix} \ln V$	0.9898	2.3991
	wet		$\begin{matrix} -0.0005 & -0.37071 \end{matrix} \ln V$	0.9538	1.000
Rototiller	air dry		$\begin{matrix} 0.9145 & -0.275591 \end{matrix} \ln V$	0.9319	2.496
	moist		$\begin{matrix} 0.376345 & -0.3188557 \end{matrix} \ln V$	0.771	1.4569
	wet		$\begin{matrix} -0.560169 & -0.375457 \end{matrix} \ln V$	0.6021	0.5711
c) $\ln \sigma_0$ vs. $\ln V_0$					
Prior	air dry	=	$1.20797 - 0.2578 \ln V_0$	0.9266	3.347
Cultivator	air dry		$\begin{matrix} 0.2266 & -0.33173 \end{matrix} \ln V_0$	0.8548	0.497
	moist		$\begin{matrix} 0.8042 & -0.2826 \end{matrix} \ln V_0$	0.9586	2.2348
	wet		$\begin{matrix} -0.2870 & -0.3576 \end{matrix} \ln V_0$	0.9950	0.7505
Rototiller	air dry		$\begin{matrix} 0.6099 & -0.27247 \end{matrix} \ln V_0$	0.8781	1.8402
	moist		$\begin{matrix} 0.83183 & -0.258 \end{matrix} \ln V_0$	0.684	2.298
	wet		$\begin{matrix} -1.2950 & -0.3940 \end{matrix} \ln V_0$	0.6806	0.274

(a)

(b)

(c)

(d)

Picture 1: Cracks on air-dry soil (No.4) plots prior to tillage (a) 25/11/2011

Fig. 1 :_ Weighted aggregate sizwe distribution for soil (No.2), tilled at air-dryness and when moist (1st year).

Fig. 2 : Cohesive strength, [kPa] before and after tillage on moist and air-dry plots (soil No.2) - 1st year.

Fig. 3 : Cohesive strength [kPa] before and after tillage on moist and wet plots (soil No.2) - 2nd year.

Fig. 4 : Weighted aggregate size distribution for soil No. 3 tilled at air dryness and when moist (1st year).

Fig. 4a: Energy for tillage [$\text{Nm} \cdot \text{m}^{-2}$] as function of soil moisture content [%w/w basis].

Fig. 5 : Cohesive strength (kPa) before and after tillage soil No. 3.

Fig. 6 : Cohesive strength [kPa] before and after tillage soil No. 3.

Gan Shmuel

	Dry	Wet
k =	-0.14 308 ± 0.0732	-0.11307 ± 0.0693
A ₂ =	54.4 ± 9.4 [KPa]	72.2 ± 8.7 [KPa]
r ² =	0.799	0.791

Fig. 7 : Tensile strength of aggregate samples of different diameter size groups from plots tilled when air-dry or moist. Friability [k], bulk soil estimates [A kPa] and coefficient of correlation squared r^2 are given (Soil No.3).

Fig. 8 : Tensile strength of aggregate samples of different diameter size groups from plots tilled when air-dry or moist (Soil No. 1). Friability factor [k], bulk soil estimate A [kPa] and coefficient of correlation squared r^2 are given.

Loess

	Dry	Moist
k =	-0.1674±	-0.1912
A ₂ =	8.5 ± 1.1	3.2±0.8
r ² =	0.874	0.929

Fig. 9 : Weighted aggregate size distribution for soil No.1 tilled at a "dry" and moist state.

Fig. 10: Weighted aggregate size distribution for soil No. 4 tilled under "dry" conditions.

Fig. 11: Weighted aggregate size distribution for soil No. 4 tilled when wet.

Fig. 12: Weighted aggregate size distributions for soil No. 5 for the whole tilled layer (Δ), surface layer ($\blacksquare$) and bottom layer ($\circ$).

Fig. 13: Weighted aggregate size distributions for the top soil layer for natural pore tillage conditons (●), Cultivator (○) and Rototiller (▲) for soil No. 5.

Fig. 14: Logarithm of natural tensile strength, $\log S$ vs. $\log$ of probability of aggregate "survival" relationships for pretilled aggregate samples of 2 to 4 mm ($\square$) and 25.4 to 50.8 mm ($\circ$) mm diameters (Soil No.5).

CHAPTER 2

LABORATORY AND FIELD CHARACTERIZATION OF SOIL FRAGMENTATION
IN TILLAGE. II. PUERTO RICO

Research conducted by Victor A. Snyder and Miguel A. Vázquez

Abstract

The development of simple laboratory tests capable of predicting soil friability under tillage has proven difficult. One of the reasons for these difficulties is that most tests are load- or energy- controlled, whereas tillage is primarily a displacement-controlled process. It is by subjecting soil to a specified displacement process that tillage provides the energy inputs required for crack propagation and at the same time directs the displacement path in such a way that fracture is most effectively induced. Any realistic laboratory simulation test must do the same. In addition, it is necessary that the amount of fracture occurring should be measured, rather than only load at failure as in many tests.

A simple test has been developed which meets these requirements, and in addition is capable of measuring forces and energy expenditures during the fracture process. Comparison with tillage studies in the field suggests that the test is capable of comparing relative friabilities of different soils, and predicting the optimum moisture content for tillage.

An unexpected finding was that for a given loading geometry in displacement-controlled fracture, the relative size distributions of fragments, as indicated by the log standard deviation, was

practically constant across all soils and moisture contents. This seems to imply a universal scaling property of crack populations induced by a specific load-path geometry. An unresolved question is whether this scaling invariance requires constant size-scale of the load-path, or whether the load-path itself can be scaled (maintaining constant geometry) without affecting the relative distribution of fragments.

Introduction:

During tillage, soils are subjected to complex patterns of deformation and fracture.

The primary objective of tillage is to induce fracture. Most tillage implements achieve this by first weakening the soil in shear and compression, then causing it to flow in a bending and twisting pattern before falling back to the ground. Paul (1967) has shown that combined compression and shear in brittle materials tends to induce stable cracks, i.e. relatively small cracks that turn as they propagate in such a direction that conditions become unfavorable for further crack growth. Thus, even though the compression/shear stage of tillage is characterized by the formation of large, catastrophic shear-failure planes (Gill and Vanden Berg, 1968), by the time these planes appear it is likely that an additional network of smaller cracks has formed throughout the soil mass. When, after the initial compression/shear phase, the sheared, intensively cracked soil is forced to flow in a bending mode and fall to the ground, a state of tension is produced over much of the soil volume by the bending moments and stress waves

resulting from the impact of falling. These stresses are relatively small compared to those in the compression/shear stage, but crack growth under generalized tension is very unstable, i.e. once it begins it tends to proceed in a catastrophic manner (Paul, 1967; Snyder and Miller, 1989). Hence, the bending and falling processes in tillage can be very effective in achieving final fragmentation of the intensively cracked soil produced during the initial compression/shear stage.

As was recognized at an early stage by Griffith (1924), the growth of cracks implies the formation of new surface area and a corresponding amount of surface energy. Thus, fracture of materials requires not only applying the stresses necessary to initiate fracture, but also the energy input (work) necessary to allow propagation of these cracks. Work implies not only stresses but also displacements. Thus, in order to achieve effective soil breakup, tillage implements must take the soil through a displacement process rather than merely stressing it until fracture initiation.

Ideally, most of the energy input in tillage is utilized in producing new fracture surfaces. However, fracture is also accompanied by variable amounts of soil deformation, particularly during the early compression/shear stage. This deformation can be either elastic or plastic. If a large fraction of the tillage input energy is used in elastic deformation of the soil, and this energy in turn is not released efficiently in the form of propagating cracks (partly a function of tillage implement design), then the unutilized input energy will simply be lost as elastic rebound

after passage of the tillage implement. Such appears to often be the case in very dry clay soils, which although requiring high energy inputs for tillage, tend to break up into large clods having low specific surface area. On the other hand at the wet end of the soil moisture spectrum, the plastic yield stress of the soil is very low. Here the tillage input energy is mostly dissipated as irreversible plastic flow rather than formation of crack surfaces. The optimum condition for tillage is usually at some intermediate moisture content where plastic deformation is small and crack propagation is maximized.

The question of course is; "what is the optimum moisture content for tillage of a given soil?". One would like to answer this question without having to conduct a field experiment every time a new soil is encountered. The ability to theoretically predict the fracture behavior of elastic-plastic materials for even simple loading systems and material properties is very difficult. In the case of tillage the problem beomes almost intractable because of the complexity of loading conditions and the extreme variability of soil constitutive properties. As a result, the tendency has been to develop semi-empirical soil testing procedures which attempt to simulate conditions encountered in tillage, and observe the resulting soil behavior.

Existing test procedures vary widely in terms of the soil properties which are measured. Many simply measure soil strength under prescribed loading conditions. Others measure the amount of fracture or plasticity as a function of energy input or prescribed boundary displacements. The detail of measurement varies from

highly quantitative to simple visual observation. No general consensus seems to exist regarding the relative merits of these tests for predicting soil behavior in tillage.

It is the purpose of this paper to review the most common tests and highlight their major strengths and limitations. A simple plane strain device, operating on the same principle as the classic Atterberg plastic limit test, is then described, which combines the strengths and minimizes the limitations of most current procedures. Preliminary experimental results obtained with the proposed method are presented.

Common tests for simulating soil behavior in tillage:

A simple procedure is the drop shatter method (Marshall and Quirck, 1950; Gupta and Larson, 1982). Soil fragments are dropped repeatedly to a hard surface from a specified height, usually about 1 m. Fracture is induced by rapidly propagating stress waves originating at the point of impact between a fragment and the hard surface. The magnitude of these stress waves depends on the elastic properties of the soil, and on the impact velocity which in turn depends on drop height. The amount of fracture surface formed - estimated from the size distribution of fragments - is plotted as a function of cumulative impact energy. It is tacitly assumed that, independently of the height of each drop, stresses high enough to initiate and propagate fracture at all or most of the relevant points of weakness within the sample are induced. As long as this minimum stress condition is met, the amount of fracture in elastic brittle materials will depend primarily on cumulative drop height (i.e. energy input) rather than on the height of each drop.

However, if drop height is too low to achieve the minimum stress required for fracture, then both drop height and cumulative energy input will influence the amount of fracture.

Care must be exercised in interpreting the drop-shatter test whenever extensive plastic deformation or rate-dependent stresses occur. These conditions may be encountered when the drop height is high enough to induce stresses exceeding the plastic yield strength of the material, or results in sufficiently fast impact velocities to cause rate-dependent (viscous) stresses. In either case a significant fraction of the impact energy may be dissipated as irreversible deformation. Hadas and Wolf (1984) observed the problem in dry, brittle soils once the drop height exceeded about 2 m. Similar difficulties could be expected at even lower drop heights for very soft material such as moist or loose soil. The effect of variable drop height under the latter conditions does not appear to be well established.

Another test of soil fracture behavior is the Utomo and Dexter friability index (Utomo and Dexter, 1981). This test uses the fact that the probability of fracture below a given stress level increases with aggregate size, due to the increased probability of large cracks. The rate at which the logarithm of aggregate crushing strength decreases with the logarithm of aggregate size is taken as an index of the tendency of soil to fracture into smaller fragments under loading. The procedure assumes brittle (elastic) fracture to be the failure mode, which may place restrictions on its use in highly plastic soils. Unlike in the drop-shatter test, the actual amount of fracture is not measured because the specimen is only

tested up to the point of incipient failure. Other limitations of the friability method were discussed in Chapter 1 of this report.

Christensen (1930) proposed a friability index based on unconfined axial compression tests of cylindrical soil samples. Friability was defined as the ratio of axial strain at shear failure to the energy required to reach this point. Friability defined as such was observed experimentally to increase monotonically with moisture content even up to quite wet conditions. It thus did not seem to differentiate well between what is normally considered a loose, "friable" soil and one which is simply in a weak, plastic state. Furthermore, the actual amount of fracture occurring was not measured, again because testing was only conducted until failure initiation.

Chandler and Stafford (1987) proposed a similar procedure, except that the method of representing results was different. They plotted the crushing strength of a soil clod vs. the corresponding strain prior to failure, and characterized the suitability for tillage of a soil based on its position in this plot. Very weak soils exhibiting large displacement prior to failure were characterized as too plastic and thus unsuitable. Very strong soils with low displacements, although exhibiting brittle behavior, were also qualified as unsuitable for tillage because of high energy requirements and because of the tendency of very hard soils to break into large clods. The most suitable soils were characterized as those failing at both low displacements and low stresses. Since loading was only applied until incipient failure, the actual amount of fracture occurring during the test process was not measured.

The fact that all of the above tests, with exception of the drop-shatter method, only measure stresses and/or energy up to the point of initial failure or maximum load is very important. As discussed previously, much of the crack growth in soils probably occurs after such failure points. Although fracture initiation depends on the magnitude of the applied stresses - usually the point of maximum stress where most tests stop - the extensive growth of fracture surfaces requires that the stresses be accompanied by displacement processes.

The drop-shatter method compensates for this problem somewhat by providing a continuous supply of energy for crack propagation after the point of initial fracture. However, a possible limitation is that the independent variable against which fracture is compared is energy input, whereas in tillage the independent variable is primarily displacement, i.e. the boundary displacement path which the soil is forced to follow by the geometry of the tillage implement. In tillage, quantities such as stresses, deformation of the soil material, fracture and associated energy expenditures are all dependent variables which depend on the imposed boundary displacements and soil constitutive properties. It is only during the final stage of the tillage process - where bending moments induced by gravity and the free falling of soil fragments are allowed to occur - that stress or energy controlled situations occur. As mentioned earlier, stresses and energy expenditures imposed during this stage are probably small compared to those in the initial displacement-controlled stages, with their major role being the final separation of fragments corresponding to opposite

sides of cracks formed in the initial stages.

It would seem then, that rather than using maximum load or energy as the dependent variable, a laboratory test of soil behavior in tillage should consist of a given boundary displacement of the test specimen, which simulates the major features of the boundary displacements produced by tillage. This process should be accompanied by measurement of the associated (dependent) phenomena of interest such as fracture, plastic deformation, required boundary stresses and associated energy expenditures. The idea of using imposed boundary conditions as a starting point for analyzing soil behavior has found useful applications in other areas of soil mechanics as well (Baligh, 1985).

Another important consideration is the geometry of loading. It has been shown that the relative magnitudes of principal stresses play a major role in the conditions for fracture initiation (Paul, 1968; Hettiaratchi and O'Callaghan, 1984) and on the fraction of the input energy which actually results in fracture (Sih, 1973).

We note within this context that all of the tests discussed previously are essentially unidirectional compression tests in which allowable compression/shear ratios are severely limited by system constraints. In diametric loading of spherical or cylindrical samples such as in the Brazilian, aggregate crushing and drop-shatter tests, tensile stresses perpendicular to the direction of loading are quite large relative to other stresses. This increases the probability that tensile fracture will occur prior to significant compressive or shear deformation, and is precisely the reason why diametric loading tests are useful for

measuring tensile strength. However, the same property may limit applicability of these tests as simulators of fracture under the relatively high compression/shear ratios typical of tillage. As was mentioned earlier, loading in a compression-shearing mode allows the formation of many small, stable cracks in a soil, which will enhance failure at many points once catastrophic failure by direct tension is induced. However, if the soil is loaded in tension before these cracks have formed, failure is likely to occur at only a few locations since only one crack is sufficient for catastrophic failure under direct tension. Thus, tests which rely primarily on tensile stresses may underestimate the amount of fracture occurring in tillage where a preliminary compression/shear stage is usually present.

On the other hand, axial loading systems, such as triaxial or unconfined compression tests, tend to err in the opposite direction in that the compression/shear ratio is higher than for most tillage situations. This is because all boundary stresses in these tests are principal stresses which are equal to or greater than zero. As a result, normal stresses in the test specimen are nowhere less than zero, i.e. they are everywhere compressive. In this case soil compression during loading could conceivably interfere with the crack formation process in highly compactible soils. The problem can in principle be eliminated by allowing at least one of the principal stresses in a test to be negative (tensile). This has been achieved under triaxial conditions (Bishop and Garga, 1969), but procedures are time consuming and require great care to avoid experimental artifacts. Thus, methods which load the sample with

principal stresses are of limited applicability for routine testing in situations where low compression/shear ratios must be achieved.

Due to the importance of compression/shear ratio on soil behavior, tillage simulation tests should be capable of producing compression/shear ratios which are typical of tillage implements. Furthermore, since tillage implements vary in the compression/shear ratios which are produced, a test should in principle be capable of varying the ratio correspondingly.

A realistic compression/shear ratio is one of the great strengths of the simple hand-crushing test which is commonly used by farmers and field personnel for characterizing soil consistency and suitability for tillage. The test consists of crushing the soil by hand in a shearing-compression mode, along a displacement path which is controlled by the fingers. Note that not only a realistic compression/shear ratio can be achieved, but also the previously discussed requirement of taking the soil through a displacement process. By simple visual observation and the "feel" of soil reaction against the hand, semi-quantitative information is obtained on two important aspects: 1) the relative amounts of plastic deformation and fracture which take place for a given boundary displacement; and 2) the stresses and energy required in the process. Thus, in spite of its crudeness, the technique is quite complete in terms of the parameters which are evaluated.

The well-known Atterberg Plastic Limit test is similar in action to the hand-crushing test, and has been used extensively in conjunction with tillage studies. It is basically a two-dimensional, plane strain version of the hand crushing technique.

A cylindrical soil sample is placed between two parallel surfaces (usually the human hand and a hard glazed surface) which move both parallel and perpendicularly toward each other in such a way that the sample is forced to roll and compress radially at the same time. The radial compression force induces tensile stresses perpendicular to its line of action, similar to those in uniaxial diametric crushing tests. However, shearing is also induced due to the fact that rolling is forced to occur simultaneously with the compression. A more detailed analysis of the force systems involved will be presented later. For the moment, we simply note that the Atterberg Plastic Limit test is essentially a diametric crushing test with shear stresses superimposed on it.

In spite of its capacity to simulate soil behavior under combined compression and shearing, the relevance of the Plastic Limit test to soil behavior in tillage is limited due to the way the test is currently conducted. The only parameter measured is the lowest moisture content (the so-called plastic limit) at which a remoulded soil sample can be rolled to a specified diameter without fracturing. The plastic limit so obtained is a soil "constant" which is independent of transient soil variables such as structure and field water content, since these variables are eliminated during sample preparation. This makes the plastic limit a meaningful parameter when effects of constant soil properties such as texture and clay mineralogy wish to be evaluated. But the transient variables which are eliminated during sample preparation are crucial in determining soil reaction to tillage in the field. Furthermore, the plastic limit test in its current form is only

conducted until the point of incipient fracture, which limits its use in characterizing the fracture process itself.

However, with little effort, the principle of the plastic limit procedure can be used in such a way that these limitations are removed. Furthermore, quantitative information similar to that provided by other methods such as compression, drop-shatter and hand-crushing tests can be obtained.

The remainder of this paper describes a simple method for achieving this effect. Data illustrating the ability of the method to predict field soil fragmentation during tillage are presented.

During the course of analyzing the data, a remarkably uniform scaling relation in the size distribution of aggregates became evident. A brief discussion on this phenomenon and its possible implications is given.

Principle of the Method:

The experimental apparatus is shown schematically in Fig. (1). It basically consists of an upright triangular frame on frictionless rollers, which is free to move back and forth inside a larger triangular frame which is geometrically similar to it. As the small triangle moves back and forth any distance L along the base of the large triangle, the perpendicular distance S between the hypotenuses of the inner and outer triangles will change by the amount $L \sin \theta$. If a cylindrical or spherical object is placed in the space between the two hypotenuses and the lower frame is moved forward toward the vertex of the angle θ , the cylindrical sample will be rotated and diametrically compressed simultaneously as discussed previously for the plastic limit and hand-crushing

techniques. The rate of change dS in specimen diameter per unit forward displacement dL of the inner triangle is given by

$$dS/dL = \sin \theta. \quad (1)$$

Likewise, the tangential displacement dT is related to dL by

$$dT/dL = \cos \theta \quad (2)$$

It is convenient to reduce the variables S , T and L of Eq. (1) in terms of the initial specimen diameter D_0 , which allows writing Eqs. (1) and (2) as

$$dS^*/dL^* = \sin \theta \quad (3)$$

and

$$dT^*/dL^* = \cos \theta \quad (4)$$

where

$$S^* = S/D_0 \quad (5)$$

$$T^* = T/D_0 \quad (6)$$

and

$$L^* = L/D_0 \quad (7).$$

The reason for writing S , T and L in reduced form is that now all displacements can be interpreted as fractions of the original specimen diameter. A given horizontal displacement dL^* will always result in the same diametric strain S^* of the sample independently of sample size. Furthermore, all samples subjected to the same reduced displacement dL^* will have rolled over a constant reduced tangential distance dT^* . Note that the number of revolutions N of the cylinder around its axis of rotation is given by

$$N = dT/(\pi D_0) = (1/\pi) dT^* = (1/\pi) dL^* \cos \theta \quad (8)$$

which implies a constant number of revolutions for constant dT^* or

dL*.

Thus, by reducing the variables in Eq. (3) to (8) to dimensionless form we have obtained rules which guarantee geometrical similarity of motion for cylinders of any diameter D_0 .

Since displacement dL* of the the inner triangle over the outer frame occurs on (near) frictionless wheels, frictional resistance to the displacement is negligible. Thus any force F pushing the inner triangle toward the vertex of θ will be resisted entirely by the cylindrical sample. From trigonometric relations of Fig. (1) it can be established that F is related to the compressive force F_c and the tangential force F_t acting on the sample by

$$F_c = F \sin \theta \quad (9)$$

and

$$F_t = F \cos \theta \quad (10)$$

The compression/shear ratio is obtained by dividing Eq. (9) into (10):

$$F_c/F_t = \tan \theta \quad (11)$$

which shows that the ratio of normal to shear forces acting on the sample depends on the angle θ . Thus, we might expect variable behavior of samples which are forced to deform in systems with different θ . This of course immediately suggests a source of variability in plastic limit tests where the ratio of compressive to tangential forces during the rolling out of samples is quite arbitrary. On the other hand, if θ is a well controlled parameter - as it is in the proposed system - one can use the result Eq. (11) to study soil behavior under different compression-shear ratios. Note that for $\theta = 90$ degrees, we simply have a uniaxial compression

system.

It can be seen from Eqs. (9) and (10) that if the force F on the lower triangle can be measured and the rolling friction of the triangle is negligible, then the forces F_c and F_o can be calculated. Thus F can be considered the independent force variable of the system. Note also that if no slippage (only rolling) occurs at the contacts between the soil cylinder and the hypotenuses of the inner and outer triangles of Fig. (1), no frictional work will occur at these points. For these conditions, the principle of virtual work (Goldstein, 1980) guarantees that the work done by the independent force F during its corresponding displacement (a measurable quantity) will be equal to the work done in deforming/fracturing the sample by the dependent forces F_c and F_r .

The amount of fracture occurring during a prescribed boundary displacement of the system can be measured by a number of means. The crudest technique, but also the simplest, is simply to pass the deformed and fractured sample through a nest of sieves, and estimate new surface area from the size distribution of aggregates. This is the approach used in the drop-shatter method. A limitation of the approach is that it only measures "free" crack surface, i.e. cracks that have propagated far enough to allow complete separation of soil fragments. This may be all that one is interested in from the point of view of tillage. However, if crack surfaces must be measured more precisely in samples that are only partially fractured, a technique such as digital image analysis of sample cross sections could perhaps be adapted.

Experimental Apparatus

Following is a more detailed description of the design of the apparatus shown in Fig. (1).

Two sides on each of the inner and outer triangular frames were constructed in wood, using rugged boards with a 5 cm x 20 cm cross-section. The two sides were joined with heavy-duty steel hinges which allowed varying the angle θ anywhere between zero and 90 degrees. The hinge system of the outer triangular frame was constructed in such a way that the hinged end of the hypotenuse board could be raised and lowered as necessary relative to the base board of the frame. This allowed adjusting the height of the frame to accomodate larger or smaller samples without having to change the length of the frame.

The third side of each frame consisted of adjustable supports the length of which were varied as necessary to achieve the desired value of θ .

The supports for the outer frame consisted of upright aluminum rods bolted to a wooden base under the frame. Aluminum cross-bars were clamped at the desired heights on the uprights to hold the hypotenuse board of the frame in place. Two supports were used; one at the end of the frame and another about midway along the frame. The end support held the hypotenuse board at the proper angle θ by means of cross-bars fixed both above and below the board. The middle support had a cross-bar above the hypotenuse board, to prevent upward bending of the board when very hard samples were being tested.

The end-support for the inner frame consisted of any strong

object which could be placed between the base and hypotenuse boards of the frame such that the angle θ at the hinge joint was maintained at the proper value.

In order to insure "frictionless" movement of the inner frame, the frame was mounted on rugged roller-bearing wheels such that it could roll freely over the base of the outer frame. The front wheels were of the pivoting type to insure that the frame would always move in the direction of least rolling resistance of the sample.

Displacement of the inner frame was achieved with a screw-type jack capable of producing loads as high as 0.5 KN. Two of these were placed in series when large displacements were required. The force (F) produced by the jack was measured with a proving ring fastened between the jack and the frame and fitted with a sensitive LVDT transducer. Linear displacement of the entire frame was monitored with a large LVDT transducer which had a displacement range of ± 37 cm. The analog signals from the transducers were fed to a Linseis Model L4000 chart recorder, which produced a graph of results and also relayed the signals in digital form to a computer for further processing.

When tests are conducted at high values of θ , the inclination of the triangle surfaces could cause fragments to reorient or slide along the surfaces and thus influence results. This problem can be eliminated by simply orienting the triangles in space such that the hypotenuse surfaces are horizontal. In this case gravitational effects would have to be subtracted from the forces and energy involved in moving the inner triangle against gravity.

Experimental studies:

In order to evaluate the method, two types of experiments were conducted. The first experiment evaluated the effects of variable compression/shear ratio (i.e. variable θ) imposed by the experimental apparatus on the degree of soil fracture obtained in materials of different plasticity and friability. In a second series of experiments, laboratory fracture patterns and load-displacement curves obtained on "undisturbed" field samples were compared with measured aggregate size distributions obtained by tilling the soil immediately after sampling.

Experiment 1: effect of variable compression-shear ratio.

Three soil materials of clayey, loamy and sandy texture, respectively, were chosen for analysis. The three materials exhibited very different fracture properties as preliminarily assessed by hand-crushing. The clay samples were obtained from the upper B horizon of the Rio Piedras clay series (Typic Tropudults, clayey, kaolinitic, isohyperthermic). The other two materials were taken from deposits of fill material near the Soils Laboratory of the Agricultural Experiment Station, Río Piedras, Puerto Rico.

Cylindrical soil samples were taken from each site with a hand-held, push type soil probe which collected samples 1.9 cm in diameter and several cm long. The samples were trimmed to 3 cm lengths for analysis. Sampling was conducted several times at each site to obtain a range of moisture contents.

In a typical measurement, a sample was placed between the two frames of the crushing apparatus, with the longitudinal axis of the sample perpendicular to the long axis of the apparatus. The sample

was crushed by displacing the inner frame forward until the distance between it and the outer frame was equal to half the original diameter of the sample. This was equivalent to a sample displacement $dS^* = 0.5$, for S^* defined as in Eq. (5). The load-displacement pattern was monitored electronically as described previously.

After crushing, the sample was passed through a sieve nest, and the fragments retained on each sieve were weighed. Subsequently all size fractions were combined and oven-dried at 105°C for moisture content determination. The geometric mean diameter and log standard deviation of the fragments was determined following standard methods (Larson and Gupta, 1982).

Measurements were conducted at four different values of the frame angle θ for each soil and sampling date. All measurements were made in triplicate.

Experiment II.

Field tillage experiments were established at four sites, corresponding to soils with variable texture and clay mineralogy. The soils and locations were as follows:

- 1) Coloso silty clay loam (Aeric Tropic Fluvaquents, fine, mixed, nonacid, isohyperthermic) near Gurabo, Puerto Rico. Clay mineralogy is primarily 2:1 and 1:1 silicate clay, with minimum amounts of free oxides.
- 2) Fraternidad clay (Udic Chromusterts, very fine, montmorillonitic, isohyperthermic), located near Lajas, Puerto Rico. The predominant clay in this soil is montmorillonite.

- 3) Corozal clay (Aquic Tropudults, clayey, mixed, isohyperthermic), located near Corozal, Puerto Rico. Clay mineralogy includes significant amounts of 2:1 and 1:1 silicate clays, quartz and sesquioxides.
- 4) Coto clay (Typic Eutruxox, clayey, kaolinitic, isohyperthermic), located near Isabela, Puerto Rico. Besides kaolinite, clay minerals present in significant amounts are quartz and sesquioxides.

Three of the soils (Fraternidad, Corozal and Coto) were clayey in texture, with 50 percent clay content or more. The essential difference between these soils was the relative abundance of high and low activity clays. A medium textured soil (Coloso) was also included in order to include textural variation in the experimental data set.

Each soil was tilled with a disc plow on at least three different dates, when the soil was judged to be in plastic, friable and hard states, respectively. On each date, tillage was conducted on four randomly chosen plots within the field. Each plot was tilled once, and half of the plot was immediately tilled a second time. The same disc plow was used on the Coloso, Fraternidad and Coto soils. A different plow had to be used at Corozal.

Immediately prior to tillage, 9 to 10 cylindrical soil samples were taken from the plow layer of each plot with a push type soil probe, as described in Experiment I. Large undisturbed soil cores were also taken by driving sharpened 10 cm sections of 10 cm diameter (ID) PVC pipe into the ground in the middle of each plot. A minimum of two cone penetrometer measurements were taken within

a 20 cm radius of each core, using a recording cone penetrometer pushed by hand into the ground at a rate of approximately 5 cm/sec. A 30° cone with a 1.25 cm² projected cross-sectional area was used. The soil cores were then extracted by pick and shovel, trimmed and taken to the laboratory for measurements of bulk density, water content and soil water matric potential. Water content and bulk density were determined by standard procedures of weighing field and oven dry samples and measuring the inner volume of the sampling cylinders. Matric potential was determined by equilibrating the moist soil in contact with 7 cm discs of Whatman no. 42 for at least 48 h, and comparing the equilibrium water content of the filter paper with a calibration curve determined from pressure plate and vapor pressure experiments (Hamblin, 1982). Tillage operations were carried out after sampling. After tillage, an 0.5 x 0.5m area was chosen at random in each half-plot receiving one tillage operation, and similar areas were chosen for the half plots which had been tilled twice. All soil fragments down to the bottom of the tilled layer within the 0.5 x 0.5 m areas were carefully removed and passed through a nest of sieves. The mass of soil retained on each sieve was weighed, and from this data the geometric mean diameter and log standard deviation of the aggregate size distribution were calculated.

In the laboratory, the small cylindrical soil samples taken with the push-probe prior to tillage were analyzed by the crushing procedure described in Experiment I. An instrument angle of $\theta = \tan^{-1}(0.22)$ was used for the Coloso, Corozal and Coto samples. This angle was found to be too high for the highly plastic Fraternidad

samples, so the angle was reduced to $\tan^{-1}(0.11)$. The reason we express the instrument angle in terms of an inverse tangent is that the argument of this function is equal to the compression/shear ratio defined in Eq. (11).

Results and Discussion:

Experiment I.

The aggregate geometric mean diameters obtained by testing the various soil samples at different compression/shear ratios is shown in Fig. 2.

As might have been expected, the geometric mean aggregate diameter tended to decrease with an decrease in compression/shear ratio, corresponding to more fracture and less plastic deformation. Notice that the difference between curves tended to decrease at lower compression/shear ratio, but the curves never crossed each other except at one point at very low compression/shear ratio. The curves in general were so close to each other in this zone that the single crossover point observed may have been due to experimental error. Thus, a decrease in compression/shear ratio decreases sensitivity but does not appear to alter the order of friability index obtained for different soils.

On the other hand if the compression/shear ratio is increased too much, very plastic soils will not fracture at all but will simply flatten out as a single unit. Plastic soils which are tested within this range will all flatten out and no difference between them will be observable even though this difference may exist in the field.

These preliminary data would seem to indicate that the method

apparently can be used to compare relative friabilities between soils without too much concern regarding the specific compression/shear ratio, as long as the latter is held constant. In general the ratio chosen is a compromise between the need for high sensitivity (requiring relatively high compression/shear ratio) and the need to obtain at least some fracture in all samples in order to compare them. Our preliminary experience indicates that an angle $\theta = \tan^{-1}(0.22)$ is appropriate for soils of medium friability such as loams or clays of low to medium colloidal activity. For highly plastic soils an angle of $\tan^{-1}(0.1)$ or smaller may be necessary.

Experiment II.

General soil characteristics.

Values of cone index, bulk density and degree of pore saturation measured from the large undisturbed soil cores are plotted as a function of soil water matric potential in Figs. (10), (11) and (12).

Bulk density values (Fig. 10) were similar for all soils. A tendency toward a sharp decrease in bulk density at matric potentials > -0.5 bars, corresponding to swelling effects, is strongly noticeable in the Corozal soil and slightly noticeable in the Coloso soil. As might have been expected, this tendency was not observed in the Coto oxisol. Rather surprisingly, the Fraternidad vertisol did not exhibit a bulk density change with drying. However, this may have been due to the fact that samples with a matric potential greater than about -0.6 bars were not taken for Fraternidad.

Moisture release curves are shown in Fig. (11). A tendency

toward increasingly sharp moisture release patterns is apparent in the order Fraternidad, Corozal, Coloso and Coto.

The relation between cone index and matric potential is given in Fig. (12). With exception of the Coto soil, the relation between cone index and matric potential is very strong and quite similar between soils. The Fraternidad and Corozal soils in particular are very similar, showing a parabolic relation with a maximum cone index of about 25 Kg/ cm² at matric potentials between -5 to -8 bars. The increase in cone index is sharper for the relatively coarse-textured Coloso and very sharp for the highly weathered Coto soil. However, the maximum values reached are similar for all soils. The data suggest that the slopes of the cone index and moisture release curves are inversely related to each other. This is possibly due to an inverse relation between plasticity and the slope of the moisture release curve, with greater plasticity being the real cause of a slower increase in cone index with matric potential.

Soil fragmentation under tillage

Geometric mean diameters for aggregate populations obtained in the field tillage experiments are plotted as a function of soil water matric potential in Fig. (3). In spite of the considerable scatter in the data, the trend toward greater soil fragmentation at lower matric potentials is apparent for all soils in the plots receiving only one tillage operation. In the loamy Fluvaquent (Coloso) and the low-activity clayey Oxisol (Coto) the amount of fragmentation reaches a stable maximum value at matric potentials close to -1 bar. However, the more plastic Fraternidad and Corozal

soils require drying to -4 bars or less before reaching maximum fragmentation.

Another important difference between the two groups of soils is that the effect of matric potential in Coloso and Coto is almost obliterated when a second tillage operation is conducted immediately following the first. The effect of secondary tillage is much less noticeable in the more plastic Fraternidad and Corozal clays.

Thus, tillage alone appears to be quite effective for preparing good seedbeds in low to medium activity soils. However, in the more plastic soils, achievement of good tilth requires additional inputs such as mellowing associated with wetting and drying cycles. As mentioned earlier, we were unable to fully evaluate the tilth mellowing effect in this study due to the unexpected need to divert efforts toward development of methodology for studying the fracture behavior of plastic soils. However, we did conduct a simple experiment where undisturbed rectangular blocks from a number of soils (including the soils in the tillage experiments) were subjected to wetting and drying cycles. Wetting was effected by misting rather than by raindrop impact. After three cycles, only the highly expandible Fraternidad and a very sandy mollisol had significantly broken apart. This suggests that in addition to wetting and drying, effective tilth mellowing requires kinetic energy associated with raindrop impact, and/or bending moments induced by gravity in randomly stacked irregularly shaped clods rather than the uniform normal stress existing in rectangular blocks lying on a flat base.

Comparison between field and laboratory behavior

Geometric mean aggregate diameters (GMD) obtained with the laboratory crushing test for the different soils are given in Fig. 4. In order to allow direct comparison with field fragmentation results, GMD values were plotted as a function of soil water matric potential. Ideally, the simplest approach would have been to plot field GMD's against the corresponding laboratory GMD's. However, for some soils a problem was encountered in that the plastic bags used to transport the crush-test samples to the laboratory were not as vapor tight as assumed. Consequently some drying occurred and direct comparison between field and laboratory samples was no longer feasible. The problem was discovered too late to repeat the field experiments. To circumvent the problem, matric potential was directly measured by the filter paper method on laboratory samples immediately prior to testing. This allowed constructing an accurate curve of laboratory GMD's as a function of matric potential, which could then be compared against the corresponding curve for GMD's measured in the field.

In general, it can be observed by comparing Figs. 3 and 4 that the laboratory GMD's followed similar trends as those in the field. This is particularly so for the Coloso, Coto and Corozal soils. Note that the point of minimum GMD is reached at approximately the same matric potential for both field and laboratory samples.

In the case of the Fraternidad soil the agreement is not so good. Two possible reasons for this exist. The first is that the laboratory samples had dried to the point that the highest matric potential in the sample population was about -4 bars, which is close

to the point of minimum GMD obtained in the field. Thus, the laboratory curve effectively missed the important region of the field curve. In addition, even though the angle θ of the crushing apparatus was reduced from $\tan^{-1}(0.22)$ to $\tan^{-1}(0.1)$ to test the highly plastic Fraternidad soil, the angle was still apparently too high. As a result significant fracture of laboratory samples did not begin to be observed until matric potentials considerably lower than those corresponding to maximum fracture in the field.

These results, in general, indicate that the crushing technique offers promise for obtaining an indication of the most appropriate soil water matric potential for tillage.

Further indication that this is so is obtained from preliminary analysis of the load-displacement curves obtained on the laboratory samples. A typical curve is shown in Fig. 5. As deformation of a sample progresses, the load generally increases continuously and then drops sharply corresponding to formation of a tensile fracture crack down the middle of the specimen. Multiple peaks and breaks occur subsequently, corresponding to a series of secondary fractures as the sample continues to be deformed.

The decrease in load at first fracture, expressed as a percentage of the maximum load at fracture or "percent rebound" (F_f/R from Fig. 5), can be used as an indicator of the degree of elasticity of the soil. In Fig. (6) the amount of rebound at first fracture is plotted as a function of matric potential for the Coloso soil. It can be noted that rebound increases sharply with drying until a matric potential of approximately -0.6 bars, at which point it suddenly levels off. Comparison of this curve

with the field and laboratory GMD curves for the Coloso soil in Figs. 3 and 4 shows that the point of maximum rebound coincides with the point of maximum fracture. Thus, the crushing method also appears to offer potential for determining the point of "safe" tillage beyond which plastic deformation of soil structure is minimized.

Information on energy efficiency of fracture can also be obtained with the technique. The energy required to fracture a sample, calculated from the load-displacement curves, was divided by the surface area estimated from aggregate size distributions to obtain a "specific fracture energy". This index is plotted against matric potential for the Coloso soil in Fig. 7. Note that a minimum value (corresponding to maximum energy efficiency of fracture) seems evident at about -0.6 , with a dramatic increase on the wet side of this minimum and a more gradual increase on the dry side. Again, the minimum corresponds very closely to the point of minimum GMD in field and laboratory fragmentation tests.

Scaling of aggregate size distributions

A rather striking result was obtained when the log standard deviations (LSD) for aggregate populations obtained by tillage and laboratory crushing tests were plotted against matric potential in (Figs. 8 and 9). In all cases the LSD was almost completely independent of matric potential, i.e. it was nearly constant for a given soil and loading condition. This is in spite of the fact that the geometric mean diameter varied considerably. In the field soils, LSD was also independent of number of tillage operations, and in fact had very similar values for all soils. The Corozal soil

is not included in Fig. 8 because data had not been completely analyzed as of this writing; however, preliminary LSD values averaged about 0.6, which lies in the same range of values of the other soils. The LSD's for the laboratory crush test were significantly smaller than for the field tests, but again values were quite similar accross soils. The implication of these results seems to be that variation in LSD is to a large extent a function of loading geometry, with relatively small variations attributed to soil type and moisture content.

The log standard deviation is essentially a scaling factor which indicates the relative size ratios of different fractile intervals of the population. A constant value of LSD indicates similarity of populations. Thus, for a given geometrical configuration of boundary displacements imposed on a soil system, similar populations of cracks are apparently being induced independently of soil moisture content and to a large extent even soil type. This is a rather remarkeable result given the wide variety of soils in this study and the low level of control on conditions in field tillage experiments. It strongly suggests the existence of fundamental (chaotic ?) physical principles governing the spatial distribution of cracks formed during displacement-controlled fracture.

An important question is the extentent to which LSD is invariant upon macroscopic scaling of boundary conditions; i.e. whether only the shape of the boundary displacement controls LSD, or whether size scale is also important. Preliminary analysis (Snyder, 1991) would seem to predict scaling invariance. However,

the proof of the pudding is of course experimental evidence, and this is not available in sufficient detail in the present experiments to allow reaching conclusions. Should scaling invariance be established, it would have important applications in scale model tests of tillage implements.

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Figure 1. Diagram of Soil Crushing Device for Friability Testing.

Figure 2. - Aggregate geometric mean diameters obtained at different compression/shear ratios ($\tan \theta$) for materials of varying friability.

Figure 3. - Aggregate geometrical mean diameters after one and two tillage operations in four soils.

Figure 4. - Aggregate geometrical mean diameters obtained with the friability testing device.

Figure 5. - Typical load-displacement curve
curve obtained with friability
testing device.

Figure 6. - Percent rebound at first fracture for Coloso soil samples tested on friability device.

Figure 7. Specific surface energy for Coloso soil samples tested on friability device.

Figure 8. - Log stand deviations for aggregate populations obtained by one and two tillage operations.

log standard deviation

Soil water matrix potential (bars)

Figure 9. - Log standard deviations for aggregate populations produced by friability testing device.

Figure 10.- Bulk density as a function of soil water matric potential for the soil studied.

Soil water matric potential (bars)

Figure 11. - Degree of pore saturation as a function of soil water matric potential for the soils studied.

Soil water matric potential (bars)

Figure 12. - Soil mechanical impedance, as measured by cone index, at different soil water matric potentials in the soils studied.

CHAPTER 3

SPECTRAL ANALYSIS OF CONE PENETROMETER DATA FOR
DETECTING SPATIAL RRANGEMENT OF SOIL CLODS

CHAPTER 4

ENERGY RELATIONS OF PROPAGATING CRACKS IN VARIABLY PLASTIC SOILS

Research conducted by V.A. Snyder and M.A. Vázquez - Puerto Rico

Abstract

A fundamental limitation in tillage studies has been lack of physically based measurements for characterizing the relative tendency of a soil to deform plastically or fracture. This problem has been largely due to lack of a theoretical and experimental framework for studying fracture in highly plastic soils. With the advent of J-integral analysis in the late 1960's and early 1970's, major progress has been made in the general theory and experimental analysis of fracture of elastic-plastic materials during the past 25 years. These developments have found extensive application in many materials science disciplines, but do not appear to be well known in the soil science community. It is the purpose of this paper to summarize the salient features of J-analysis and provide illustrative experimental data for an Oxisol and a Vertisol at various water contents.

Introduction

When a soil is subjected to intense shear and/or tensile stresses such as in tillage, the tendency is for it to undergo a combination of plastic deformation and fracture. The goal of tillage, of course, is to maximize fracture with a minimum of plastic deformation.

It is well known that different soils (or even the same soil

at different water contents and/or void ratios) can be subjected to identical loading systems and yet behave drastically different in terms of the predominance of fracture or plastic deformation which results. The need exists to be able to characterize these differences through easily measured yet physically based soil parameters. A number of empirical procedures have been devised for this purpose, the most popular of which still seems to be the classic Atterberg limit test. In Chapter 2 of this report we discuss some of the limitations of the Atterberg and other tests, and describe a procedure which provides more complete information on the relative brittleness or plasticity of soils. However, even this procedure is largely empirical in nature. The problem with all of these tests is that they integrate rather than separate the fundamental soil properties which determine whether a soil will fracture or yield plastically.

A major reason for this state-of-the-art is that, while the theory of soil plasticity has been reasonably well developed (Hettiaratchi and O'Callaghan, 1984), the same cannot be said regarding soil fracture mechanics. Until quite recently, the general theory of fracture mechanics was restricted to materials which exhibited elastic or only slightly plastic behavior (Anderson, 1991). As a result, most published theoretical work on fracture of agricultural soils is based on linear elastic fracture mechanics. These results are of limited applicability in moist soils which exhibit significant plasticity.

It has only been during the last 25 years that Rice (1968 a,b), Begley and Landes (1972) and others developed the theoretical

and experimental framework for extending fracture mechanics into the fully plastic regime. These seminal results and subsequent developments have found extensive applications in many of the materials science disciplines (Anderson, 1991), but do not appear to have made their way into the soil mechanics community.

With this situation in mind, it is the purpose of the present paper to summarize recent advances in the theory of fracture of elastic/plastic materials and associated experimental techniques. Results obtainable by some of these techniques are presented for two clay soils, a Vertisol and an Oxisol, at varying water contents.

Theory

When an elastic-plastic material such as soil is loaded, the initial deformation increments tend to be elastic in nature (Hettiaratchi and O'Callaghan, 1984). However, elasticity involves a continuing and often very sharp increase in stored mechanical energy with increased deformation. In many materials, this can only occur to a limited extent before an energy releasing process such as plastic yield or fracture will occur.

In plastic yield, mechanical work done on a system is essentially converted into frictional heat, which under closed isothermal conditions is then lost to the environment. The material retains its macroscopic nature as a continuum, although in many cases with altered specific volume and constitutive properties. In fracture, on the other hand, material continuity is disrupted by the formation and propagation of surfaces of discontinuity, i.e.

crack surfaces. These surfaces imply storage of energy in the form of surface energy or "surface tension". Thus, thermodynamically, fracture differs from plastic deformation in that work done on the material is converted into surface energy rather than lost as heat to the environment.

These considerations suggest that the criteria which determine tendency towards fracture or plastic deformation of a material are fundamentally thermodynamic in nature. This is indeed the case, as will become evident from the following discussion. For the sake of clarity, we first consider fracture in elastic materials, and then extend discussion to the elastic/plastic case.

When an elastic material with a pre-existing crack is loaded, a zone of stress concentration and associated stored (elastic) mechanical energy develops in the vicinity of the crack tip. If the crack were to grow, stress relaxation would occur in the surrounding material, resulting in release of some of the stored mechanical energy. The total amount of energy in a cracked body which would be released per unit extension of a crack of unit width is commonly termed the "strain energy release rate" J and is defined as

$$J = dE/da \quad (1)$$

where E is the strain energy and a is crack length.

It is important to note that in general, J represents a potential situation rather than an actual occurrence; it indicates the amount of elastic energy which would be released by material surrounding a crack if the crack were to grow by an amount da under current loading conditions. Under this concept, J should be

considered as energy available for crack extension. The question of under what conditions the crack will actually begin to grow is discussed later.

The available energy J is a function of the type and magnitude of loading, and of material properties such as elastic moduli and crack size. Three main types of loading with respect to the orientation of a crack plane are generally distinguished (Figure 1): mode I (pure tension), mode II (in-plane shear) and mode III (out-of plane shear or "tearing"). For plane strain conditions, it can be shown (Anderson, 1991) that

$$J = C \sigma^2 a / E \quad (\text{for mode I or II loading}) \quad (2a)$$

or

$$J = C \sigma^2 a / \mu \quad (\text{for mode III loading}) \quad (2b)$$

where σ is the stress (tensile or shear, depending on loading mode) in the material at an "infinite" distance from the crack, a is half the crack length, E is the elastic modulus and μ is Poisson's ratio. The proportionality constant C is a function of the loading mode. For small-strain linear elastic conditions, the value of J for a mixed loading situation involving modes I, II and III is simply the sum of individual components given by Eqs. (2).

In order for Eqs. (2) to apply, a two-dimensional plane-strain situation must exist. This condition has been shown to exist to a good approximation in the vicinity of flat "penny shaped" cracks embedded in three-dimensional elastic bodies (Anderson, 1991) and in flat plates traversed by cracks such as in Fig. 1. The shape of most cracks lies somewhere between these two configurations, thus rendering Eqs. (2) quite general in applicability.

As discussed previously, an essential feature of fracture is the formation of new surface area corresponding to the boundaries of growing cracks. This involves absorption of energy in the form of surface tension. The amount of surface energy A absorbed per unit crack area (or unit length for a crack of unit width) can be written in a similar manner as the strain energy release rate

$$A = dE_s/da \quad (3)$$

where E_s denotes surface energy, and a is crack length as before. Note that A is equal to twice the surface tension of the material, since two units of crack surface (one on each side of the crack) are produced for each unit of crack length.

It is important to note that the specific surface energy A is a material property, whereas J depends on loading mode and magnitude, as well as on material properties such as crack length and elastic moduli.

In his classic theory of fracture, Griffith (1924) realized that crack growth (fracture) can only occur if the strain energy J released by crack growth exceeds the surface energy A required to form the corresponding new crack surface. In other words, the fundamental thermodynamic requirement for fracture of an elastic material is

$$J > A \quad (4)$$

If the releasable energy is less than the required surface energy ($J < A$), fracture will not occur.

Plasticity plays an important modifying role in the above relationships, in that it places an upper limit on the strain energy release rate J that can be developed in a cracked material.

If a material has a high plastic yield stress, it can be loaded to high stresses and still behave elastically. This allows very intense accumulations of stored elastic energy, with correspondingly high values of J . If the specific surface energy A of the material is low, J can easily be made to exceed A and fracture will readily occur. This constitutes a brittle material. On the other hand, a material with very low plastic yield strength will undergo plastic yield before stresses can reach values high enough for J to exceed A . Fracture in such a material cannot occur, unless strain hardening increases the plastic yield stress (and hence the capacity to store elastic energy) to the point where J can be made to exceed A . Materials where J is always less than A , or where considerable strain hardening must occur before $J > A$, are said to be tough.

Materials such as molding clay or very wet soil are infinitely tough, since the plastic yield strength is always too low for J to exceed A . However as a soil dries, its plastic yield strength and rigidity ($1/E$) tend to increase, with a corresponding capacity for developing large J values. In general, one might also expect the surface energy A to increase with drying, but not so fast as the plastic yield stress, as indicated by the tendency toward greater brittleness of drier soils. Data on A values discussed later on in this paper tend to verify this statement.

Rice (1968) suggested a simple technique for measuring J and A in linear or non-linear elastic materials. Consider two elastic bodies, identical in all respects except that the crack length a in one of them (Fig. 2a) is shorter than in the other body (Fig. 2b)

by an amount Δa . Both bodies are loaded at the same point by either a fixed external force P (with displacement a dependent variable), or by imposing a fixed displacement s (with force the dependent variable). In either case, the corresponding work W on the body required to achieve the imposed force or displacement is determined from the area beneath the experimental load-displacement curve for the load point. Note that in thermodynamic terms, W for a force-controlled loading situation would correspond to the change in Gibbs free energy of the loaded body, whereas W for displacement-controlled loading would correspond to the change in Helmholtz free energy (Malvern, 1969). Loads or displacements are kept at values less than those causing crack growth initiation, such that the initial crack length in each body is not altered during the course of loading. Loading may be of type I, II or III.

Because of the different crack lengths, the loading work W will in general be different for the two bodies. Rice realized that for elastic materials, this difference (ΔW) is exactly equal to the strain energy ΔE which would be released if the shorter crack in the body of Fig. (2a) were to extend by an amount Δa , such that the final crack length and boundary conditions (load and displacement) conditions were equal to that in Fig. (2b). This is an application of the principle of path-independence of an elastic potential function: the change in elastic potential distribution -and the related total energy - upon a change of state of a loaded body is independent of the path through which this change takes place. It is tacitly assumed that a given material element under the same boundary conditions will always have the same spatial distribution

of elastic potential within the material, i.e. the state of the element is uniquely specified by the conditions imposed at its boundary. This is obviously true for linear elastic materials since the potential distribution satisfies Laplace's equation, which by virtue of its linearity guarantees uniqueness of potential distribution for given boundary conditions. Non-linear elastic materials with a unique stress-strain relation can in principle be linearized by effecting Kirchhoff transformations on the potentials. Since these transformed potentials satisfy Laplace's equation, uniqueness between boundary conditions and state of the material element is again established.

If the experimental difference Δa in crack length between the two bodies in Fig. (2) is small enough that $\Delta a \rightarrow da$, then from Eq. (1)

$$\Delta E / \Delta a \approx dE/da = dW/da = J \quad (5)$$

Begley and Landes (1972) were the first to experimentally determine J by use of Eq. (5). They simply measured load displacement work (W) for various crack lengths (a) and then calculated J as the slope of this relation.

The surface tension A of an elastic material can in principle be determined by measuring J at the point of incipient crack growth and applying the identity Eq. (4). However, the point of incipient fracture is often difficult to determine exactly. A common procedure is to measure J at gradually increasing loads or displacements for a given pair of crack lengths a and $a+da$, and infer fracture initiation from the point where a "significant" change in the slope $\Delta J / \Delta P$ or $\Delta J / \Delta s$ occurs (Anderson, 1991).

Rice (1968) realized that J- analysis could also be extended to plastic materials. He noted that the two-body loading procedure described above will give exactly the same values of ΔW for plastic and elastic materials as long as the stress-strain relation during loading is the same for both materials. This assumes no unloading of the plastic material, because then its stress-strain relation would change. Such a constraint does not appear to be restrictive so long as large loading-unloading cycles are not involved (Paris, 1977).

Even though the same value of ΔW is obtained for elastic and plastic materials of identical stress-strain relations, the physical meaning of ΔW (and hence J) is different in each case. For elastic bodies, ΔW is equal to the stored elastic energy which would be released in its entirety as rebound energy were the crack to extend by an amount Δa . However for elastic-plastic bodies, ΔW represents the sum of releasable stored elastic energy (ΔW_E) plus all the strain-hardening plastic dissipation energy (ΔW_p) required to harden the material enough to store the elastic energy ΔW_E . Thus, for elastic/plastic materials, the experimentally measured ΔW is given by

$$\Delta W = \Delta W_E + \Delta W_p \quad (6)$$

so that

$$J = dW/da = d(W_E + W_p)/da \quad (7)$$

Since dW_p is lost to the environment as heat, only dW_E is available to form new surface energy.

Equation (7) shows that experimentally measured values of J for elastic/plastic materials represent the sum of elastic energy

actually available for crack growth, plus the amount of plastic dissipation energy which must be "wasted" per unit crack extension in order to harden the material sufficiently for the available elastic energy to accumulate.

It is useful to be able to separate the elastic strain energy release rate dW_e/da from the plastic dissipation component dW_p/da in Eq. (7). An experimental solution to the problem is to load, unload and then reload the specimens to the same previous load or displacement. The first loading gives J as defined by Eq. (7), i.e. it includes both elastic and plastic energy components. The second loading gives only the elastic component dW_e/da , since all plastic energy was lost in the first load-unload cycle. The difference between the first and second J values is the plastic component dW_p/da . Note that if the load-unload cycles are conducted at successively larger loads until fracture initiation, the elastic component dW_e/da corresponding to the fracture point will in principle be equal to the surface tension A .

Substituting the fracture condition $dW_e/da = A$ into Eq. (7) gives J at fracture initiation as

$$J_f = A + dW_p/da \quad (8)$$

where the subscript f denotes the value of J at fracture. As mentioned previously, A is a material constant. However, the value of dW_p/da in plastic materials is not in general constant, but rather increases with increasing crack length. This is because stresses at a given distance from a crack tip increase with crack size (Anderson, 1991). As a result, the amount of plastic deformation in the zone surrounding a moving crack tip increases as

the crack grows, requiring increasing amounts of plastic energy with each successive increment in crack length. Thus, the experimental value of J_f in Eq. (8) is not in general a material constant as it would be if only A contributed to J_f . Rather it is a function of crack size.

Further insight into the meaning of the parameter J in elastic/plastic materials was obtained by Rice (1968 a,b) in a now classic analysis. We summarize his results here, drawing primarily on reviews by Paris (1977) and Anderson (1991).

Consider the state of stress and deformation in the vicinity of the crack as shown in Fig. 3a. For purposes of mathematical analysis, the deformation in this zone is allowed to be either plastic or elastic, as long as the condition of "small-strain" ($\leq 10\%$) deformation exists. In practice, a zone of very intense "large strain" deformation may exist in the immediate vicinity of the crack tip (Paris, 1977). It is generally assumed that this zone is small relative to other dimensions of the system.

Rice (1968) showed that within the material domain where the small-strain assumption applies, a certain line integral has some useful properties. This line integral, known as the J -integral in the fracture mechanics literature, is given by

$$J' = \int_{\Gamma} [W dy - T_i (\delta u_i / \delta x) ds] = 0 \quad (9)$$

where

W = energy of deformation (elastic and/or plastic) in an infinitesimally small volume element along the

integration contour

Γ = contour path of the integral

ds = increment of distance along the contour

T_i = normal traction (force) acting on the contour (if the latter were cut out as a free body)

u_i = material displacement in the direction of T_i , and

x, y = rectangular coordinates as noted in Fig. (3).

The fact that J' is always zero shows that it is path-independent, i.e. the same value of zero will be obtained no matter how the closed contour Γ is chosen. Thus, a contour can be chosen as in Fig. 3a, where part of the path lies along the material surface on both sides of a crack. The contour is divided into 4 sections $\Gamma_1, \Gamma_2, \Gamma_3$ and Γ_4 , with corresponding values of J' along these sections denoted as J_1', J_2', J_3' and J_4' . From Eq. (9), we know that

$$J_1' + J_2' + J_3' + J_4' = 0 \quad (10)$$

For the J -integral sections J_2' and J_4' , corresponding to the contour sections along the crack borders, the values of dy in Eq. (9) are approximately zero for a very flat (hairline) crack. Furthermore, the values of T_i are zero everywhere along the edges of a crack except at the singularity corresponding to the crack tip (where an infinitely small radius of curvature exists). Introducing $dy = 0$ and $T_i = 0$ into Eq. (9) for these sections gives that $J_2' = J_4' = 0$. Substitution of these conditions into Eq. (10) yields that

$$J_1' = -J_3' \quad (11)$$

The negative sign in Eq. (11) arises because the two corresponding integration path sections Γ_1 and Γ_3 run in opposite

directions. For both paths in the same direction, we would have that $J_1' = J_3'$. Since the contour sections Γ_1' and Γ_3' are completely arbitrary, the implication is that J'_1 (or J'_3) can be evaluated along any contour section which begins anywhere on one boundary of a crack and ends anywhere on the opposite boundary (without crossing the crack), such that its value is always the same.

So far, J' may simply appear to be an interesting integral with no obvious physical significance. However, the J -integral is in fact identical to the parameter J defined in Eqs. (1) and (7). To see this, we first consider the J -integral in an elastic material.

Suppose we somehow measure J' on contour Γ' around the tip of a crack of length a . The crack is then allowed to extend by an infinitesimal amount da ($= dx$), carrying the same contour Γ' along with it as shown in Fig. 3b. If J' is again measured along Γ' , after crack extension, its value would be expected to be slightly different than originally because crack extension changes the state of stress in a material. However, since the amount of crack extension is infinitesimal, the values of J' are close enough that they can be represented by a single average value $\bar{J}'$. Let us multiply $\bar{J}'$ by da . From Eq. (9),

$$J'da = J'dx = \int_{\Gamma} W \, dy \, dx + \int_{\Gamma} T_i (\delta u_i / \delta x) \, dx \, ds = \int_{\Gamma} W \, dy \, dx + \int_{\Gamma} (T_i \, du_i) \, ds \quad (12)$$

Inspection of terms on the right hand side of Eq. (12) shows that

$$\int_{\Gamma} W \, dy \, dx = \text{energy of deformation (gained or lost) of} \\ \text{a volume element crossed by the contour as it}$$

moves from one position to the next,

and

$$\int_{\Gamma} (T_i du_i) ds = \text{work done by tractions on the contour in moving it.}$$

Thus, $\bar{J}' da$ represents the energy gained or lost in the material zone swept out by the contour as it moves laterally a distance da equal to the amount of crack growth, and $\bar{J}'$ is the amount of energy per unit crack extension. Because of the path-independence of $\bar{J}'$, the integration contour can be made arbitrarily small until it just surrounds the crack tip. At this point $\bar{J}'$ can only be interpreted as the amount of energy made available at the crack tip surface per unit crack extension. In other words, $\bar{J}'$ is identical to the strain energy release rate J of Eq. (1).

We note that, for elastic and plastic materials with the same stress-strain relations, the values of J' must be equal since they are calculated from identical variables. Thus, J' for elastic-plastic materials has the same physical meaning as J .

It is important to note that the identity between J and J' is valid only for materials where the small-strain condition holds all the way to the crack tip, or where the large-strain zone, if it exists, is negligible in size relative to the small-strain zone. If a significant large-strain zone exists near the crack tip, then the integration path of J' cannot be taken infinitely close to the crack tip and the analogy between J and J' falls apart. However, note that it is only the (analytical) J' contour integral of Eq. (9) which is subject to the small-strain constraint. It is very

possible that the experimentally measured J of Eqs. (1) and (7) is valid for either small or large strains. However, this is contingent on being able to demonstrate that: "given 1) a unique stress-strain relation (either large or small strain), and 2) the known physical principle that the total elastic energy in a body will tend toward a minimum, then a given boundary condition will guarantee a unique potential distribution within the body". We are not sure of this point at present. Note that the arguments for uniqueness given previously on the basis of Laplace's equation only apply for the small-strain condition where the spatial coordinate of each material element is effectively unchanged by the strains.

The path-independence of the J -integral has important implications for the distribution of stresses and energy in the vicinity of the crack tip. Inspection of Eq. (9) shows that, in order for J' to be path-independent (i.e. constant for all contours), the strain and traction components W and $T_i(\delta u_i/\delta x)$ must in general vary inversely with length of the integration contour. In other words, these variables must become increasingly large as the contour shrinks corresponding to smaller and smaller radii of integration around the crack tip. Since both W and T_i are directly related to stresses in the material, stresses must also be inversely related to contour length. The proportionality constant for this inverse relation is given by J' . The greater the value of J' , the greater the stress and energy concentration effect as the crack tip is approached. Thus, J' (and its experimentally measured analogue J) is in effect a field intensity parameter which characterizes the stress and strain distribution near a crack tip.

Hutchinson (1968) and Rice and Rosengren (1968) showed that J' is related to the stress and strain distribution in a cracked power law hardening material by

$$\sigma_{ij}(r, \theta) = k_1 f_1(\theta, n) (J'/r)^{1/(n+1)} \quad (13)$$

and

$$\epsilon_{ij}(r, \theta) = k_2 f_2(\theta, n) (J'/r)^{1/(n+1)} \quad (14)$$

where

$\sigma_{ij}(r, \theta)$ = ij^{th} stress tensor component at a given value
of r and θ relative to the crack tip (Fig. 5)

$\epsilon_{ij}(r, \theta)$ = ij^{th} strain tensor component at a given (r, θ)

k_1, k_2 = proportionality constants

f_1, f_2 = dimensionless functions of θ and n

n = exponent of a strain-hardening rule of type

$$\sigma/\sigma_0 = (\epsilon/\epsilon_0)^n,$$

where σ_0 and ϵ_0 are stress and strain at plastic yield point.

For linearly elastic materials, $n=1$, resulting in the familiar $r^{-1/2}$ variation of stress and strains in linearly elastic materials (Anderson, 1991). Note that Eqs. (13) and (14) predict infinite stresses and strains at $r=0$, a physical impossibility. This error is due primarily to breakdown of the small-strain assumption of J -integral analysis at small r .

The original measurements of J obtained by Begley and Landes (1972) required conducting load-displacement curves on several samples of increasing crack length. Such a procedure can be laborious, and introduces errors due to sample variability. However, Rice et al. (1973) showed that for certain loading configurations, among them three-point bending, J could be measured

using only a single sample. The three-point bending test is particularly suited to many soils laboratories since it can be run using the same apparatus devised by Richards (1953) for measuring modulus of rupture on soil briquettes. A brief discussion of the underlying principles follows.

Consider a deeply notched (cracked) rectangular plate (Fig.4) subjected to bending moment M . The total angular displacement Ω of the plate due to the bending moment can be treated as consisting of two parts: 1) a displacement Ω_{nc} which the plate would experience if no crack were present; and 2) an additional displacement Ω_c due to the presence of the crack. Thus the total angular displacement is

$$\Omega = \Omega_{nc} + \Omega_c \quad (15)$$

The work done on the plate during bending is

$$W = \int M d\Omega \quad (16)$$

or, for load as the independent variable,

$$W = \int \Omega dM \quad (17)$$

From Eq. (5) we have

$$J = \delta W / \delta a \Big|_M = \int (\delta \Omega / \delta a) dM = \int \delta (\Omega_c + \Omega_{nc}) / \delta a dM = - \int \delta (\Omega_c + \Omega_{nc}) / \delta b dM \quad (18)$$

where the last identity arises because from Fig. (5), $a = h - b$, where h is constant so that $da = -db$. By definition, Ω_{nc} is independent of crack size (i.e. $\delta \Omega_{nc} / \delta b = 0$), so that Eq. (18) reduces to

$$J = - \int \delta \Omega_c / \delta a \, dM \quad (19)$$

Rice et al (1973) next applied dimensional analysis and found that, for very deep notches, Ω_c is primarily a function of M/b^2 . Thus,

$$\Omega_c = \Omega_c(M/b^2) \quad (20)$$

To give an idea of the reasoning behind this result, we note that relevant system variables are

E = elastic modulus (the crack is assumed to so weaken the plate that stresses "on the average" will not exceed the plastic yield stress before plate failure)

$(a/p)^{1/2}$ = dimensionless crack length parameter known to determine stress concentration, where a is crack length and p is radius of curvature at the crack tip (Anderson, 1991)

b = width of uncracked zone

M = bending moment

Ω_c = angular displacement (dimensionless) of cracked plate

Of these variables, Ω_c and $(a/p)^{1/2}$ are already dimensionless. The remaining variables E , b and M can be grouped into the dimensionless variable M/Eb^2 . For very large cracks (as in the deeply notched bar which we are considering) and small p (sharp cracks), the square root function $(a/p)^{1/2}$ is very large and is thus insensitive to further changes in crack length a , allowing the function to be considered as a constant. Since E is also constant

for a given material, the only remaining variables are Ω_c and M/b^2 , which leads to Eq. (20).

Differentiation of Eq. (20) with respect to b and applying the chain rule gives

$$\delta\Omega_c/\delta b = \delta\Omega_c/\delta(M/b^2) \cdot \delta(M/b^2)/\delta b = 2M/b^3 \delta\Omega_c/\delta(M/b^2) \quad (21)$$

Similar differentiation with respect to M gives

$$\delta\Omega_c/\delta M = 1/b^2 \delta\Omega_c/\delta(M/b^2) \quad (22)$$

Combining Eqs. (21) and (22),

$$\delta\Omega_c/\delta b = 2M/b \delta\Omega_c/\delta M \quad (23)$$

which substituted into Eq. (19) yields the expression for J

$$J = 2/b \int M d\Omega_c \quad (24)$$

Eq. (24) is very simple, but recall that we cannot directly measure Ω_c . Only the total angular displacement Ω of the cracked body and the value of Ω_{nc} of a non-cracked similar plate can be measured. From Eq. (15), we know that $\Omega_c = \Omega - \Omega_{nc}$, which substituted into Eq. (24) gives finally

$$J = 2/b \int_{\Omega_{nc}}^{\Omega} M d\Omega \quad (25)$$

Eq. (25) is technically the correct expression to use. However, if the crack is deep enough that $\Omega_c \gg \Omega_{nc}$, then for practical purposes the total displacement Ω can be substituted for Ω_c in Eq. (24) and measurements on an uncracked specimen are not necessary.

With three-point bending, the work $M d\Omega$ is induced by a point force F (actually a line force) displaced a linear distance ds . From the principle of virtual work we know that both must be

equivalent, i.e.

$$M d\Omega = F ds. \quad (26)$$

Thus one can write Eq. (25) as

$$J = 2/b \int F ds \quad (27)$$

Finally, we note that Eqs. (25) and (27) are derived for a plate of unit thickness. For any thickness c different from unity, results should be divided by c .

Methods and Materials:

J- analysis was performed on two soils of very different clay mineralogy; an oxisol and a vertisol. The oxisol was Coto clay (Tropoctic eutrustox, clayey, kaolinitic, isohyperthermic) and the vertisol was Fraternidad clay (Udic chromustert, very fine, montmorillonitic, isohyperthermic). Chemical and physical data for the two soils are given in Table 1.

The soils were collected in the field, air-dried and passed through a 1.8 mm sieve. Batches of approximately 1 Kg of soil were gradually wetted by spraying and mixing, until the water content was such that pore saturations of 0.3, 0.4, 0.5, 0.6 and 0.7 were obtained when the samples were compressed to a bulk density of 1.5 g/cm³. The moistened samples were allowed to equilibrate in vapor-tight plastic bags for 24 h, and were then compressed into special briquettes for J- analysis.

Two sets of briquettes were formed, with dimensions as shown in Fig. 5. The two types of briquettes were essentially identical to each other except that one set was formed with a very deep notch while the other un-notched. The briquettes were formed by placing

the desired amount of moist soil in wooden moulds, and compressing the soil with a Carver laboratory press until a bulk density of 1.5 g/cm^3 was reached. Three notched samples and two un-notched samples were formed for each soil and pore saturation. After compression, the samples were allowed to equilibrate in plastic bags inside styrofoam boxes for 36 h prior to J-testing.

For J- testing, a sample was removed from its plastic bag, and the V-shaped notch was made sharp enough to simulate a deep crack by pressing a razor blade into the material at the bottom of the notch. This procedure has been found acceptable for J-testing of soft materials such as plastics (Anderson, 1991, p. 496). The sample was then placed in a three-point bending position on a beam balance apparatus as shown in Fig. 6. A constant rate of loading was applied by allowing water at constant head to pour into a container placed at one end of the beam balance. The loading rate on the sample was about 0.2 KN/min . Load was measured by means of a sensitive LVDT fitted to a proving ring, and displacement of the load point was monitored with another LVDT fitted to the weighing platform of the beam balance. Transducer output was recorded on a two-channel Model L-4000 Linseis chart recorder. A load-controlled rather than displacement-controlled mode of loading was chosen because of the more unstable nature of fracture under controlled loading (Anderson, 1991). In unstable situations, catastrophic failure rapidly ensues once fracture initiates, thus allowing easier detection of the point of fracture initiation.

The main objective of the present analysis was to obtain values of the specific surface energy A . Thus testing needed to be

done in a way which allowed estimating the reversible (elastic) component of J at fracture. This was achieved by loading the specimen until approximately $1/3$ to $1/2$ of the load required for fracture (curve-section a in Fig. 7a). The sample was unloaded (curve b), then reloaded until fracture (curve c). The initial linear part of curve c corresponded to reloading in the region which had previously been loaded and subsequently unloaded in curves a and b. Thus its slope was taken to be the elastic component of the load-displacement curve. By extrapolating this section to the failure point (curve d in Fig. 7a), an estimate of the elastic component of the load-displacement curve was obtained.

Once the runs had been conducted for the notched specimens, the un-notched specimens were also loaded, unloaded and re-loaded (curves a, b and c, respectively in Fig. 8b). The load in both the first and final loading was increased until it exceeded the maximum (failure) load encountered for the notched specimens. The slope of the elastic loading component (curve c) was then subtracted from the slope of the elastic loading curves for the notched samples (curve d in Fig. 8a). This procedure corrected the notched sample curves for both system elasticity (a significant linear load-displacement relation was observed for the balance itself) and un-notched sample deformation (i.e. Ω_{nc}). The area under the final corrected curve yielded the energy integral $\int F ds$ of Eq. (27), which upon multiplication by $2/bc$ yielded J .

After all specimens had been tested, they were allowed to equilibrate for 48 h in close contact with 7.0 cm discs of Whatman no. 42 filter paper. The filter paper was then removed and its

gravimetric water content was determined. Comparison of this water content with a pre-determined calibration curve for the filter paper (Hamblin, 1981) yielded the matric potential of the soil.

Results and Discussion

Values for J at failure ($\approx A$) and matric potential of the different samples studied are given in Table 2. It can be seen that repeatability between replicate samples is fair at best, indicating the need to run at least 4 or 5 samples to obtain good averages.

Sample deformation during testing was very small, with cumulative load-point displacement (corrected for balance distortion) at onset of fracture being typically on the order of 10^{-4} m. Thus the small-strain requirement seemed reasonably well satisfied.

Average values of A are plotted as a function of degree of pore saturation and matric potential in Fig. 9. In general, A values for the vertisol are considerably larger than those for the oxisol. This property, combined with the relatively low plastic yield stress of vertisols, is consistent with the general tendency of these soils to deform rather than fracture over a wide range in moisture contents. Similarly, the low values of A for the oxisol are consistent with the generally friable behavior of oxisols.

Apparent trends in the data of Fig. 9 suggest that values of A may be similar for the two soils at either very low or very high water contents. However, testing at these water contents was impossible using present sample preparation and measurement techniques. Samples of very dry soil compressed to a density of 1.5 g/cm^3 tended to crumble very easily, and very wet samples fractured

under their own weight. Problems with very dry soils could perhaps be overcome by first saturating and then gradually drying either laboratory-prepared or field samples. Testing of very weak specimens could be achieved by placing and loading the samples in a horizontal direction.

It is important to keep in mind that the measurements reported here are for Mode-I (tensile) loading. Through the incremental crack length procedure discussed earlier, J- analysis can also be applied to mode II and III load configurations which involve shear. These loading modes are particularly relevant to tillage conditions. Of interest would be seeing whether or not estimated values of A are similar for all three loading modes. Similarity would indicate path independence, which in turn would add credibility to the notion that A is equivalent to (twice) the thermodynamic surface tension.

Conclusions:

The results of this study are preliminary and mainly illustrative in nature. The major objective has been to increase awareness of very useful theoretical and experimental tools which are available for studying the phenomenon of fracture in variably plastic soils. A seemingly fruitful avenue of research would be establishing a better linkage between principles of fracture mechanics and plasticity theory as embodied in critical state soil mechanics.

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Table 1

Physical and mineralogical data for the Fraternidad and Coto
Soils

	<u>Coto</u>	<u>Fraternidad</u>
Texture		
% sand	28	18
% silt	10	30
% clay	62	52
Particle density (g/cm ³)	2.74	2.71
Mineralogy	50% kaolinite 30% quartz 15% goethite	primarily montmorillonite, small amounts of kaolinite
Organic carbon (%)	2.5	1.6
Exchange complex	< 5% exch. Na	< 5% exch. Na

Table 2

J values at fracture for Coto and Fraternidad soils **

<u>Soil</u>	<u>Pore saturation</u>	<u>Matric potential</u> -MPa-	<u>J</u> -N/m-
Coto	0.3	-7.4	118
			112
			-
	0.4	-0.7	107
			45
			34
	0.5	-0.05	67
			40
			-
Fraternidad	0.4	-2.1	162
			107
			91
	0.5	-0.65	260
			108
			226
	0.6	-0.32	202
			175
			-
	0.7	-0.07	99
			158
			164

** Bulk density of all samples was 1.5 g/cm^3 , corresponding to a total porosity of about 0.45.

Figure 1. Three modes of loading a crack: I) opening or "tensile", II) in-plane shear; and III) out-of-plane shear or "tearing".

Figure 2. Two-body loading procedure for determining J .

Figure 3. a) Components of the J-integral around a stationary crack tip.
 b) J-integral contour moving with propagating crack tip.

Figure 4. Deeply notched plate in pure bending.

Figure 5. Dimensions of soil briquettes used for J-analysis.

Figure 6. Beam balance system for loading samples.

Figure 7. Loading history of notched and un-notched samples.

J Values at Fracture vs Pore Saturation

J Values at Fracture vs Matric Potential

Figure 8. J-values at fracture obtained for an oxisol and a vertisol at various pore saturations and matric potentials.

RESEARCH CONDUCTED UNDER OBJECTIVE 2

CHAPTER 5

Structured soil behavior under compressive and shear stresses

Research conducted by A. Hadas - Israel

1. Introduction

A tillage implement applies stresses to the soil volume it tills. The resultant soil fragmentation and spatial repositio-ning of clods or aggregates depend on existing, pre-tillage soil structure and the bulk soil response to stresses applied. The active and passive behavioral response of soil to these forces is reflected in the degrees of fragmentation attained. In a sense, when tillage operations or traffic-induced soil compaction are discussed, one has to divide soil response into soil motion (e.g. reallocation of soil structural units) and soil deformation (e.g. shear, volume change densification, dilation). Soil deformation does not follow the same laws as gases (a compressible fluid) since we deal with a medium made of solid granulae and lubricants e.g. clay and clay-soil solution mixtures.

When an implement is used by a farmer to till a soil in a given state, the farmer attempts to reach a given goal, namely create a certain new soil environment favorable to the development of the intended crop. This is done by applying stresses and exchanging momentum through tillage implements with the soil. The soil responds by either shear, tension, compression, and flow or reallocation of material. At times one or two response modes may be distinguished, and sometimes all four are involved. Yet, there

are cases where none of these "simple" responses can be pointed out or defined, nor can the extent of the applied forces which cause soil to fail and reach the final tilth which we describe in the field (Shaffer and Johnson, 1982; Mckeys, 1985, 1989).

Of these soil responses, shear is commonly described in terms of the Coulomb friction model (Yong, Warkentin, 1975; McKeys, 1989). Shear strength at failure is given by

$$\tau = C_{sh} + \sigma \tan \phi \quad [1]$$

where τ and σ are the shear and normal stress acting on the soil, C_{sh} is soil cohesion and ϕ the angle of internal friction. This form of failure occurs along existing ruptured, cracked surfaces and along interfaces between the soil matrix and implement surfaces.

Shear strength has been found to vary with soil constituents, water content and loading history (McKeys, 1989; Koolen and Kuipers, 1983; Yong and Warkentin, 1975). Furthermore, measurements to determine it show a dependence of the resultant data on the devices and measuring procedures used (Bailey and Weber, 1965; Yong and Warkentin, 1975). Hence the shear model parameters, namely the cohesion C_{sh} and apparent soil friction angle ϕ , should be determined experimentally for each soil environment and stress field, since there is still no proven model that correlates soil state variables (e.g. texture, density, moisture content, structure and particle size distribution) to its behavioral characteristics.

Commonly, soil structure has been and still is overlooked when

shear strength is measured. The stresses applied or measured are usually given as the forces measured or imposed for a given supposedly uniform surface area. If soil is the material undergoing shear, some inference must be given to the fact that it is granulated and the contact area is not uniform unless polished.

Soil failure by compression is usually associated in a simple manner to volume change (unidimensional sinkage or compression). Commonly used models apply the Boussinesq model (1885) for elastic, semi infinite solids as adapted to soil systems by Frohlich (1934) and Soehne (1953, 1958), or their more recent derivatives (e.g. Gupta and Larson, 1982). Soil compression can be measured by a unidirectional compression procedure and results expressed as (Gupta and Larson (1982)).

$$D = D_0 + C_{com} \log_e P \quad [2]$$

where D and D_0 are dry soil bulk density under stress P and at zero stress history, respectively. Alternately, triaxial compression can be used and results expressed by (Bailey and Grisco (1984)

$$\log_e D = \log_e D_0 - (A + B \sigma_h) [1 - \exp (- C \sigma_h)] \quad [3]$$

where σ_h = hydrostatic confining stress, and A , B and C are empirical coefficients.

When prediction of tillage implement - soil interaction is considered, Coulombic shear-plane failure and compression are usually taken to be the major forms of soil failure. Yet, failure by tension or plastic flow at times are also major forms of soil failure (e.g. in tillage of air dry and wet soils, respectively).

In both these cases, mechanisms are not well defined (Schafer and Johnson, 1982).

All of the above modes of failure are influenced in as yet incompletely understood ways by the degree of granulation of the soil. Thus, laboratory tests were carried out in order to study in some detail the interactions between soil structure and stresses under variable loading conditions. These tests included:

- a) direct shear and compression tests on selected whole soils which were initially considered to be used for field experiments;
- b) analysis of compression and shear strength of soil aggregate beds;
- c) analysis of influence of initial water content and load level on moisture release characteristics of compacted aggregate beds; and
- d) distribution of bulk density and inter-intra aggregate porosities under simulated traffic induced compactive load.

2. Materials and Methods

2.1 Shear and compression tests on whole soil samples.

Soil samples from eight locations (Table 1.1) were brought to the laboratory, air-dried, crushed, sieved through a 2 mm sieve and stored. Small samples were mixed with varying amounts of water to achieve pre-determined moisture contents and stored in vapour tight bags. They were then placed in an air conditioned room at a constant temperature of 24°C and left to equilibrate for a week. After equilibration, the samples were packed into a 2" by 2" cell in a direct shear unit ("Soil test" model U-125-8). The packed

samples were then preloaded at different normal loads (50, 100, 200 and 400 KPa) for 10 min or until sinkage rate decreased below 0.01 [mm/min]. The load was then relieved and the compressed samples were placed in the stress apparatus and sheared at a rate of 1 mm/min. The maximum shear stress measured or the shear stress value at 20% displacement were used to calculate the shear characteristics: namely, cohesion C_{sh} and angle of internal friction ϕ .

Two of the soils were also used for consolidation testing of soil samples at different moisture contents. The samples were packed into cylinders having a diameter of 2" and height of 2", then the samples were loaded stepwise in a consolidation unit up to loads of 20,00 kPa in order to obtain the compressibility coefficients C_{com} .

2.2 Analysis of compression and shear strength of soil aggregate beds

Large samples of Loessial soil (No. 3, Table 1.1) were brought from the field, air dried and sieved to various aggregate size fractions. Subsamples were wetted to varying amounts of water on shallow trays by spraying fine water spray and then sealed to avoid evaporation and to allow attainment of equilibrium. Subsamples were then packed carefully into square shear cells (2" x 2"), compressed and sheared. Changes in soil volume, shear strength and shear displacement were monitored and degrees of further compression or dilation during shear were determined along with the

soil shear characteristics.

2.3 Influence of initial water content and load level on moisture release characteristics of compacted aggregate beds.

Soil samples composed of mixtures of different aggregate sizes were brought to different initial water contents in the same way as described above, packed into cells of 89 mm diameter and 25.0 mm height, and compressed under different stresses. The compacted samples were then brought to saturation and subjected to drainage on a pressure plate at different equivalent water matric potentials to determine changes in pore size distributions.

2.4 Distribution of bulk density and inter-intra aggregate porosities under simulated traffic induced compactive load.

Aggregates in the 5.6 to 8 mm size class at an initial water content of 11.8 ± 1.16 were packed into cells 80.0 mm diameter by 50 mm high and compressed at different loads ranging from 1 to 750 KPa. The internal structures of the compacted aggregate beds were examined visually using structure descriptors (Dexter, 1976, 1978) to assess internal changes in porosity and sample uniformity.

3. Results and Discussion

3.1. Shear and compression tests of whole soil

a) Direct Shear

Calculated shear strength characteristics, C_{sh} and ϕ for the various soils are given in Table 1.2. As expected, changing water

content caused changes in C_{sh} , most likely through effects on the curvature of soil water minisci and resulting changes in effective stress acting on the soil. Mean cohesion values were also influenced by clay content, showing a general increase with % Clay (Table 1.2).

On the other hand, the friction angle ϕ was barely affected by soil water content. There were changes in ϕ with soil texture, although the changes were small. When one compares soils Nos. 2, 6 and 8, consisting of 22.6, 41.0 and 69.3% clay, respectively, the measured ϕ values ranged between 23-27°. Higher ϕ values than the average range were observed for soil Nos. 1, 5 and 7 where silt and sand fraction percentages were rather high. The higher internal friction in these soils was probably due to particle interlocking effects and lack of "sliding clay packets", which add to improved lubrication, during shear.

The above results agree qualitatively with predictions from scaling analysis of granular similar media governed by Coulombic friction and the principle of effective stress (Snyder and Miller, 1985; Snyder, 1990). According to this analysis, the tensile strength ($\approx$ cohesion intercept) of granular media at approximately equal water contents and void ratios should vary inversely with particle size, which is the general trend observed in the present experiments. Scaling theory (Snyder, 1990) also predicts that variations in water content and particle size should have negligible effects on macroscopic friction angle ϕ as long as structural similarity is maintained and the friction coefficient μ

at interparticle contact points is constant.

The inverse relation between cohesion and particle size predicted by scaling theory is based on a capillary model where pore-water pressures are inversely related to particle size (Miller and Miller, 1956). The capillary assumption quite likely breaks down at low water contents in soils with high clay contents as in the soils studied here. However, particle size may play a role in determining the pore-water pressures induced when adjacent soil particles are displaced relative to each other, even in purely colloidal systems where capillary theory breaks down (Snyder and Miller, 1989). In such systems, particle size would be a better state variable than measured values of matric potential. This needs to be further investigated.

The slight observed variation in friction angle with texture was likely due to different degrees of particle interlocking and different amounts of lubrication associated with varying water film thicknesses in sliding clay packets (which in effect changes the material coefficient of friction μ). Both of these factors represent deviations from similarity requirements.

Following McKeys (1985), when a cutting tool is breaking the soil and forcing the soil forward and upward (Fig. 2.9, McKeys, 1985) the tangential component (horizontal force) must be much larger than the vertical component and the mode of failure is of shear. Taking the universal earth moving equation (Reece, 1965, in McKeys, 1985), the total tool force for breaking and moving a soil of density D at a depth d and working width of w under a soil load

stress q is given by eq. [4]

$$p = [(D \cdot g \cdot d^2)N + (S_{sh} \cdot d) N_{csh} + (q \cdot d)N_q]W \quad [4]$$

where g is the gravitational constant and N , N_{csh} , N_q are functional factors which depend on acceleration, soil shear strength, tool geometry and tool-soil strength interrelations, respectively. Since the shearing angle ϕ of the soils listed in table (1.2) does not vary much (e.g. soils No. 2, 6, and 8) differences in tillage resistance of these soils would seem to be less related to internal friction than to soil cohesion, which does vary considerably with texture and water content.

However, interesting results are obtained when laboratory cohesion values are compared to the measured draft forces required to till the loessial soil (soil No. 3) and vertic soil (soil No. 6) under field conditions (see Chapter 1 of this Report). The specific resistance to ploughing the loessial soil at mean moisture contents of 3-6% (w/w) and 8-14% varied from 117 to 108 kPa, respectively. For the vertic soil at moisture contents of 12, 16, and 27-28%, the specific resistance was 114, 86 and 76 kPa, respectively. Note that, whereas cohesion (Table 1.2) for the vertic soil No. 6 is almost twice as high as that for loessial soil No. 3, the specific resistance to tillage is generally lower. These discrepancies stress the point raised in discussing tillage operations in the field, that tillage and soil fragmentation does not necessarily follow only Coulombic cohesion or shear but may be soil cracking-pattern dependent even if the soil is moist. Further evidence for this can be obtained by examining Figure 2.9 in McKeys (1985),

which shows that a soil cut will follow the lowest slip line in the moved soil. However, shear stresses along this same plane are in principle negative (McKeys, 1985), where by convention the shear stress acts in a direction opposite to the implement movement direction.

These are only scanty observations rather than clear-cut proofs, yet they give rise to a number of questions:

- a) How do fracture surfaces propagate in a porous, cracked medium under tillage, and to what extent is Coulombic shear active ?
- b) To what degree do laboratory measurements correspond to field measurements, let alone predictions of the expected field situation after tillage operations have been carried out?

b) Compression of Soil Samples

In Figure (1-1) results of the compression tests for loessial and vertic soils (soils No. 1-3 and 1-6, respectively) are given. Calculated coefficients of compression for the two soils are given in Table 1-3. For both soils, soil compressibility was lower for the high and low initial water matric potentials and higher for the intermediate values.

The reason for low compressibility values at high water contents, where interparticle friction is lowest, was likely due to low hydraulic conductivities at high soil densities. Higher rates of compression, typical of compaction under field conditions, may

have caused even lower compressibilities than those reported. At low water potentials, the low compressibilities observed were probably due to an increase in interparticle friction. At the intermediate moisture range, interparticle friction was still rather low and hydraulic conductivities were also still low but the soil was not saturated and densification could occur through air compression and/or escape. It appears that the most susceptible water content range for soil compaction is somewhere around the plastic limit or above (-1500 to -1100 KPa).

3.2 Shear and compression tests of aggregate beds

Shear strength characteristics and bulk densities of different aggregate size beds having different water contents are given in Table (1-4). For high water contents, bulk densities and angles of internal friction were the same for both coarse and fine aggregates. At low moisture contents, the coarser aggregates exhibited lower bulk density for a given stress, and for lower preshear stress a slight increase in the internal friction angle ϕ with increasing aggregate size was observed. Practically no variation in cohesion with aggregate size was apparent.

Calculated bulk densities (P_{98}) and soil compressibility values (C_{com}) under a stress of 98.1 KPa for the different aggregate size classes at different water contents are given in Table 1-5. For all water contents, P_{98} tended to diminish as aggregates size increased. However, the trend was very small at high water contents. Compressibility seemed to decrease slightly with

increasing aggregate size at high moisture content, with the opposite trend being apparent at low water content. At both water contents the variation of compressibility with aggregate size was quite erratic, with a range of values on the order of 0.02. The significance of these differences is perhaps questionable.

Changes in soil bulk density with shear are given for these samples in Table 1-6. There was a very slight tendency for an increase in dilation (lower final bulk density) with increased aggregate size for the high water content. This same tendency was much stronger at low water content.

A question arising from these experiments is why, particularly at low water contents, the beds of larger aggregates exhibited less compressibility and greater dilatancy than smaller aggregates. The question is particularly intriguing in view of the common observation (Braunack et al., 1979) that the unconfined compression strength of larger aggregates tends to be less than that of small ones. Thus, one would expect beds of larger aggregates to be more compressible than beds of smaller aggregates, provided the primary mode of failure is tension which it probably is at low water contents.

The independence of compressibility and shear dilatancy with respect to aggregate size, observed at higher moisture contents, is what one would expect from scaling theory (Snyder, 1990). According to this theory, effective stresses at any point within similarly loaded granular similar media are linear combinations of σ and $1/n_m$ for very shallow samples where gravitational forces can be

neglected. Here σ is the applied stress and n_m is the characteristic particle (or pore) length of the soil matrix. For aggregated soils, an additional aggregate characteristic length (n_a) would need to be considered. Thus, for aggregated similar media, effective stresses should be considered functions of σ , $1/n_m$ and $1/n_a$. In the present experiments, the soil matrix characteristic length n_m was constant since aggregates all came from the same soil. The parameter $1/n_m$ can thus be eliminated as a variable. Furthermore, since inter-aggregate porosity drains at matric potentials very near zero (Sharma and Uehara, 1968; Yule and Ritchie, 1980), the contribution of inter-aggregate water (influenced by n_a) to effective stresses in even quite moist soil is probably small. Thus, the parameter $1/n_a$ can also be eliminated as a variable. This leaves only the applied stress σ as a scale variable influencing effective stresses, with no effects due to aggregate size.

3.3 Compression patterns in stressed and partially failed aggregated beds

In Figure (1.2a-d) the general shapes and contact points of compressed aggregates are traced for aggregate beds at relative volumes (V/V_{initial}) of 1.0, 0.89, 0.72 and 0.57 corresponding to applied stresses of 0, 49, 98 and 725 KPa. It can be seen that the denser the beds become, more contact areas are formed and the stress is borne by a larger area.

When individual aggregates were compressed in an unconfined

mode, the mode of failure was tensile fracture. Tensile strength of the soil aggregates of different sizes and water contents are given in Table (1.7). The data show a decrease in aggregate tensile strength with increases in either aggregate diameter, its water content or with both.

The precursor method suggested by Dexter (1976) was used to evaluate the changes in inter- and intra-aggregate porosity for the particular sections given in Fig. (1.2a-d). Results are given in Table 1-8. Although the total porosity of the aggregate beds decreased continuously with increasing stress, no change in intra-aggregate porosity was observed until stresses reached values somewhere between 98 and 198 KPa. These results indicate that at stresses below about 98 KPa, any volume change of the aggregate beds occurred through mechanisms which involved no plastic deformation, such as rearrangement and/or fracture of aggregates. However, at some point between 98 and 196 KPa plastic deformation began to play a role, as indicated by changes in intra-aggregate porosity.

The compression stress vs. bulk density relationships obtained in previous experiments for crushed whole samples of the loessial soil at approximately the same initial water contents as for the aggregate samples are

$$P_b = 1.294 + 0.194 \ln(P/98.1) \quad P_w = [9.8 \pm 1.2\%]$$

and

$$P_b = 1.392 + 0.149 \ln(P/98.1) \quad P_w = 12.3 \pm 0.4\%$$

When aggregate bulk densities were calculated from these

relationships, assuming the aggregates to follow the same compression pattern as for crushed soil samples, the calculated ranges of internal bulk densities for aggregates in the 9.8 - 12.3 percent moisture content range were 1.428 to 1.495 $\text{Mg}\cdot\text{m}^{-3}$ for a 198 KPa load and 1.682 to 1.692 $\text{Mg}\cdot\text{m}^{-3}$ for a load of 725 KPa. These values are lower than the average aggregate densities estimated by Dexter's (1976) method for the samples in Fig. (1-2). Near the sample edges the estimated aggregate bulk density was even higher than the average value.

If the initial aggregate bulk densities are assumed to be caused by precompressional stresses obeying the compression equations above, then calculated precompression stresses for the aggregates are 340 and 420 KPa for moisture contents of 12.3 and 9.8 percent, respectively. For the final measured aggregate densities of 1.644 and 1.743 $\text{Mg}\cdot\text{m}^{-3}$, corresponding to applied loads of 198 and 725 KPa, the compressional stresses calculated from the compression equations will vary between 530 to 600 KPa for a final bulk density of 1.644 $\text{Mg}\cdot\text{m}^{-3}$, and between 990 to 1040 KPa for a final density of 1.743 $\text{Mg}\cdot\text{m}^{-3}$. These data suggest that either the aggregates responded to or "sensed" not the average applied stress (load divided by the cell cross section area) but rather the "actual stress" (load divided by average soil-filled cross section, i.e. the cell cross sectional area multiplied by [1-intra aggregates porosity]), or that their internal strength was less than that of a crushed soil. When the nominal applied average stresses of 1, 49, 98, 196 and 725 kPa are corrected for the actual

soil-filled cross-section, the new calculated stresses are 16, 69, 98, 220 and 760 kPa or greater, respectively. These stresses are still far lower than those required to produce the same bulk density in the whole soil as described above. It thus appears that compressibility of individual aggregates in a bed of uniform aggregate size is higher than in a whole soil with a wide range of aggregate sizes. This is possibly due to lesser confinement of aggregates in uniform-sized aggregate beds, giving them more freedom for contractive shear deformation. It could also be due to the fact that a whole soil contains many small aggregates in addition to the large aggregates used in this study. As discussed previously, small aggregates tend to be stronger than large aggregates, which would increase the average resistance of the bed to compression.

Summing up, changes in aggregate densities suggest that the actual stresses acting on the aggregates are higher than the "nominal" or "apparent" ones and that aggregate internal compression/shear strength is smaller in the context of aggregate beds than in a whole soil. This study also points out the importance of establishing the loading range within which partially supported aggregates undergo brittle fracture and the stress beyond which plastic deformation becomes important.

3.4 Compression of two aggregated beds of "similar"

aggregate size distributions and their water retention characteristics

In this set of experiments the possibility of attaining a better insight as to the failure modes of an aggregated bed was evaluated by using two mixtures of sorted aggregates (A and C) where the median weighted aggregate diameters are different but their geometrical standard deviation is the same.

The two aggregate size distributions are shown in Figure (I-3). The corresponding mean weight diameters ϕ and geometric standard deviations σ_g for the two mixtures A and C are $3.1 \pm .3$ and 10.4 ± 0.4 mm, and 3.2 ± 0.39 and 3.12 ± 0.19 , respectively.

In Table (I-9), computed values of bulk densities obtained under compressive stresses of 1 and 98.1 kPa, compressibility, and the corresponding regression coefficients are given. As expected, the lower the initial matric water potential, the lower the compressibility coefficients and density under a stress of 98.1 [kPa], for each of the mixtures. When the two mixtures were compared, the coarser mixtures (C was coarser than A) exhibited higher compressibility coefficients and lower bulk density at a stress of 98.1 [kPa]. However, the differences observed are quite small in the 0 - 1471 KPa matric potential range. Only in the very dry aggregates, at a matric potential of -20000 KPa, is compressibility of the larger aggregates notoriously larger than for the smaller ones. This moisture-dependence of the relative compressibilities of different sized aggregates is similar to

results reported earlier in Section 3.2.

Moisture release curves for aggregate mixtures which had been compressed to varying bulk densities while at different water contents are shown in Fig. (1.4). The corresponding data points are given in Table (1.10). The differences between water contents at a given matric potential for similar bulk densities are not statistically significant, even though there is a tendency toward lower water content for a given bulk density attained by compressing an initially "drier" sample, or a "finer" mixture (e.g. A vs C). This observation tends to validate the assumption adopted for this project, namely, that for a given soil, its density, soil water content and matric potential are valid state parameters which are at least independent on the load and moisture content path followed to reach a given bulk density. It is important to keep in mind that the uniqueness between bulk density and pore-size distribution reported here was encountered for the specific case of drainage of an initially saturated soil. Uniqueness of hysteresis loops in the moisture retention characteristic was not investigated.

Contrary to what might at first be expected from similar media theory, no significant effect of aggregate size on moisture release curves is apparent from Table 1.10 or Fig. (1.4). The mixtures of aggregates used had the same size distribution shape but differed in the size of aggregates by a mean factor of 3.15 (the ratio between the aggregate geometric mean diameters). According to Klute and Wilkinson, (1958); Wilkinson and Klute (1959) and Miller

and Miller (1958 a,b) these mixtures should follow the rules of similar media with respect to their macroscopic size ratio, namely, a shift toward lower water contents for given matric potential and bulk density (Klute and Wilkinson, 1958).

However, in analyzing this apparent contradiction, it is important to note that in these experiments "similar media" principles apply only with respect to the interaggregate porosity (influenced by aggregate size). The intra-aggregate pore size distribution should remain invariant with respect to aggregate size, provided all aggregates of different sizes have same internal spatial arrangement and same density. The data shown in Fig. (1.4) correspond to matric potentials of - 20 KPa and lower. Thus, most inter-aggregate water was probably absent from the very start of the experiment, since drainage of water from inter-aggregate pores appears to be complete at matric potentials lower than about -30 KPa (Sharma and Uehara, 1968; Yule and Ritchie, 1980; Gupta and Larson, 1982). This would explain why a "similar media" effect of different aggregate sizes on moisture release curves was not observed.

4. General observations and conclusions:

The following general points are evident from this study:

- a) Shear studies of soils with varying texture and initial water content indicated only a small dependence of the Mohr-Coulomb shearing angle on either soil water content or texture. Higher values were encountered in coarser textured

soils where the the possibility of particle interlocking was greater and where lubricating water layers between sliding clay packets were largely absent. The cohesion intercept, on the other hand, tended to show a significant increase with decreasing water content and increasing clay content. These observations are in general agreement with an effective stress-Coulombic friction model of soil strength, which varies with particle size distribution in a way consistent with scaling theory (Snyder, 1990). However, colloidal plasticity effects which are not accounted for in scaling theory also appeared to play a role. The data suggest that differences in friction angle may serve as indicators of "unsimilarity" of soils.

- b) Contrary to predictions of classical tillage mechanics theory, which assumes Coulombic shear to be the dominant mode of soil failure during tillage, specific resistance of some of the above soils to actual tillage operations in the field were not related to laboratory shear strength determinations. This suggests that failure of even moist soils during tillage may be largely due to fracture at pre-existing (i.e. structural) zones of weakness in the soil as suggested by Wolf and Hadas (1987) for very dry soils.
- c) Shear and compressive behavior of aggregate beds appeared to be relatively independent of aggregate size in relatively moist soils. This behavior follows predictions of scaling analysis (Snyder, 1990), provided inter-aggregate pore water

contributes negligibly to effective stresses, and intra-aggregate composition and structure are independent of aggregate size.

However, for relatively dry soils, resistance to volume reduction both under direct compression and shear tended to increase with an increase in aggregate size. The reasons for these results, and whether or not they apply to many soils or are soil-specific, are not clear at present.

- d) When aggregate beds of different aggregate size and initial moisture content were compressed to a given final bulk density, resulting intra-aggregate pore-size distributions determined by pressure plate extraction were practically identical. These results tend to agree with one of the working hypotheses of this Project, namely that the relation between void ratio, water content and matric potential can be treated as a state function of a soil as long as water content is varied along a monotonically decreasing matric potential path beginning with a saturated soil.
- e) Aggregate beds of a given aggregate size and initial water content were compressed under different loads, and internal structure was analyzed. Initial macroscopic volume reductions associated with increasing load were not accompanied by a corresponding decrease in intra-aggregate porosity, suggesting aggregate fracture as a major mode of soil failure. However, beyond a certain stress level a steady reduction in intra-ped porosity occurred, indicating the

involvement of plastic deformation in the volume reduction process.

Analysis of inter-aggregate contact points after compression, and comparison of final aggregate density with average whole soil density under similar loading conditions, suggests that aggregates in beds with narrow aggregate-size distributions are subjected to greater stress than in a medium with a wide size distribution. This is apparently a result of smaller solid cross-sectional area under narrow size distributions, which causes higher stress on individual aggregates. Furthermore, the greater porosity in unimodal aggregate beds results in less confinement of aggregates which facilitates both shear and tensile failure.

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Table I.1: Characteristics of soils used, for shear and consolidation tests.

Soil No.	Taxonomic designation	----- (w/w) % -----						
		Clay	Silt	Sand	Organic matter	CaCO ₃	Plastic Limit	Liquid Limit
1	Typic Xerochrept	3.9	3.3	92.8	0.4	-	-	-
2	Typic Rhodoxeralf	22.6	6.2	71.2	0.3	Traces	19.8	29.9
3	Calcic Haploxeralf	18.1	28.3	51.7	2.6	11.3	21.5	34.3
4	Entic Haploxerol	22.7	62.4	14.9	5.2	64.7	34.9	48.3
5	Typic Haplogrid	29.3	46.0	23.2	-	22.5	19.2	25.1
6	Vertic Palexeralf	41.0	10.5	48.5	-	16.9	34.1	42.4
7	Typic Salortid	12.2	15.9	71.9	1.2	30.5	23.8	-
8	Typic Rhodoxeralf	69.3	24.9	5.8	-	Traces	22.8	45.4

Proctor
bulk density
mg.m ⁻³ /%(w/w)
1.870/11.5
1.840/16.2
1.775/16.3
1.420/31.3
1.785/22.1
-
-
-

Table 1.2: Shear strength characteristics for the soils given in Table 1.

Soil Number	1	2	3	4														
Moisture content (w/w)%	2.0	4.0	6.0	10.0	12.0	5.6	9.1	11.4	17.2	8.0	13.0	18.0	28.0	6.8	10.8	15.8	21.2	25.2
c_{sh} cohesion [kPa]	2.9	14.8	35.5	16.7	15.8	36.2	27.2	19.9	14.2	20.4	22.6	31.8	14.6	5.7	8.6	23.8	32.2	34.2
Angle of internal friction ϕ ($^{\circ}$)	42.3	35.6	334.8	33.4	32.3	25.7	26.4	27.3	27.0	21.3	27.0	23.5	27.6	23.3	25.0	36.7	35.9	35.6
	5					6						7				8		
	9.5	14.3	17.8	20.7	22.8	24.5	16.4	17.8	23.8	29.4	8.8	12.6	16.8	20.5	24.4	25.6	30.1	33.2
	11.9	14.2	25.6	32.7	26.3	23.6	54.4	40.9	39.8	36.0	11.8	12.7	10.9	11.5	10.0	50.1	38.9	33.7
	38.8	39.9	37.6	37.6	36.4	35.2	25.2	27.0	27.0	17.8	34.6	33.0	35.3	35.2	27.3	25.4	23.7	25.7

¹ These data were given by Dr. I. Shmulewitz - Faculty of Ag. Engng. The Technion, Haifa, Israel.

Table I.3: Calculated compressibility coefficients "C" and the corresponding regression coefficients of soils #3 and #6 for four initial matric potential values.

Initial matric potential	kPa							
	<-2000		-981		-180		-98	
Soil No.	"C"	r	"C"	r	"C"	r	"C"	r
3	0.1105	0.981	0.2228	0.998	0.2022	0.996	0.1559	0.991

Soil No.	kPa							
	-1470		-392		-98		-49	
	"C"	r	"C"	r	"C"	r	"C"	r
6	0.1513	0.936	0.2558	0.984	0.2608	0.979	0.2029	0.972

Initial water content bulk densities at different preshear compression and the shear cohesion angle of internal friction and the regression coefficient squared for three aggregate size classes of loessial soil (soil No.3).

Preshear pressure (kPa)	Aggregate size class - (mm)														
	<2					2.0 to 4.0					4.0 to 8.0				
	P_1 (%W/W)	P_2 mg.m ⁻³	C_{sh}^3 (kPa)	ϕ^4 (o)	r^{25}	P_W (%W/W)	P mg.m ⁻³	C_{sh}^6 (kPa)	$\phi^{(o)}$	r^2	P_W (%W/W)	P (mg.m ⁻³)	C_{sh}^6 (kPa)	ϕ (o)	r^2
39		1.22±0.02	12.3	13.3	0.892		1.24±0.02	12.0	13.7	0.964		1.20±0.01	11.5	14.4	0.987
88	18A±0.3	1.37±0.03	19.2	14.2	0.936	17.8±0.4	1.41±0.03	19.0	15.0	0.911	17.7±0.2	1.35±0.02	19.3	16.0	0.905
196		1.57±0.03	27.3	24.9	0.995		1.56±0.03	24.6	22.7	0.948		1.56±0.03	24.3	23.2	0.924
392		1.77±0.02	42.1	30.9	0.984		1.75±0.04	32.8	32.4	0.984		1.72±0.02	33.2	32.1	0.973
39		1.31±0.01	11.1	16.7	0.984										
88	15.9±0.3	1.47±0.02	16.6	21.0	0.993										
196		1.59±0.03	21.7	24.7	0.983										
392		1.75±0.03	27.2	32.3	0.986										
39		1.25±0.02	11.6	19.6	0.960		1.13±0.02	10.7	16.2	0.985		1.06±0.03	12.1	15.6	0.962
88	10.9±0.4	1.37±0.02	17.3	24.7	0.834	11.6±0.3	1.27±0.03	16.0	17.7	0.948	12±0.3	1.22±0.01	17.2	16.0	0.938
196		1.48±0.03	20.7	23.5	0.959		1.40±0.02	21.0	23.5	0.967		1.33±0.03	21.3	24.9	0.964
392		1.63±0.05	28.7	31.2	0.988		1.56±0.03	29.5	32.2	0.991		1.47±0.02	30.3	33.5	0.995

Table I.5: Water content P_w bulk density at compression of 98.1 [kPa] P_{98} compressibility "C" and regression coefficient squared.

Aggregate size class (mm)											
2.0				2.0 to 4.0				4.0 to 8.0			
P_w	P_{98}	"C" Con	r^2	P_w	P_{98}	"C" Con	r^2	P_w	P_{98}	"C" Con	r^2
18.4	1.42	0.239	0.991	17.8	1.43	0.217	0.994	17.7	1.39	0.229	0.995
15.9	1.51	0.186	0.918	-	-	-	-	-	-	-	-
10.9	1.39	0.161	0.989	11.6	1.29	0.183	0.994	12.0	1.22	0.173	0.995

Table 1.6: Changes in bulk density during shear of three aggregate size classes, three initial water contents and four initial preshear compression levels.

Preshear stress (kPa)	Aggregate size classes [mm]											
	<2			2.0 to 4.0			4.0 to 8.0					
	P_{w1}	P_1	P_f	P_{w1}	P_1	P_f	P_{w1}	P_1	P_f			
39		1.22±0.02	1.13 to 1.26		1.24±0.02	1.13 to 1.29		1.20±0.01	1.09 to 1.28			
88	18.4±0.3	1.37±0.03	1.26 to 1.44	17.8±0.4	1.41±0.03	1.30 to 1.48	17.7±0.2	1.35±0.02	1.25 to 1.45			
196		1.57±0.03	1.43 to 1.57		1.56±0.03	1.43 to 1.56		1.56±0.03	1.42 to 1.53			
392		1.77±0.02	1.61 to 1.65		1.75±0.03	1.59 to 1.66		1.72±0.02	1.54 to 1.59			
39		1.31±0.01	1.18 to 1.39									
88	15.9±0.3	1.47±0.02	1.32 to 1.59									
196		1.59±0.03	1.42 to 1.64									
392		1.75±0.03	1.53 to 1.61									
39		1.25±0.02	1.21 to 1.29		1.13±0.02	1.04 to 1.18		1.06±0.03	0.95 to 1.13			
88	10.9±0.4	1.37±0.02	1.26 to 1.54	11.6±0.3	1.27±0.03	1.17 to 1.29	12.0±0.3	1.22±0.01	1.15 to 1.28			
196		1.48±0.03	1.37 to 1.46		1.40±0.02	1.27 to 1.42		1.33±0.03	1.19 to 1.37			
392		1.63±0.05	1.42 to 1.53		1.56±0.03	1.42 to 1.46		1.47±0.02	1.34 to 1.41			

P_{w1} - initial water content [g/g]

P_1 - initial bulk density ($\text{mg}\cdot\text{m}^{-3}$)

Range of final bulk density - after shear ($\text{mg}\cdot\text{m}^{-3}$).

Table I.7: Tensile strength, (kPa), of aggregates of different sizes under different water contents and the regression coefficients for $y = a P_w^b$ relationship (loessial soil No. 3).

Aggregate diameter class (mm)	Initial water content (% w/w basis)					a kPa	b	r^2
	2.7	12.1	17.9	22.3	26.8			
2.0 - 4.0	289	97	90	76	61	597.5	0.682	0.983
4.0 - 5.6	243	--	--	--	--	--	--	--
5.6 - 8.0	216	84	72	59	38	489.2	-0.713	0.956
8.0 -12.7	203	--	--	--	--	--	--	--
12.7 -25.4	162	80	63	32	17	449.6	-0.833	0.786

Table 1.8: Total and aggregate (D_t , D_{ag}) bulk densities ($\text{mg}\cdot\text{m}^{-3}$), total intra and interaggregate porosities (n_t , n_{int} , n_{ag}) ($\text{m}^3\cdot\text{m}^{-3}$) for an (5.6-8.0 mm) aggregate bed at initial water content 11.8% (w/w) under different compressive loads $P(\text{kPa})$ in a compression cell (diameter 150 mm and height 75)

load	P				
	1	49	98	196	725
D_t	0.989	1.178	1.323	1.456	1.662
D_{ag}	1.576	1.576	1.576	1.644	1.743
$\hat{n}_t$	0.643	0.578	0.501	0.445	0.373
$\hat{n}_{int}$	0.373	0.288	0.157	0.099	0.047
$\hat{n}_{ag}$	0.403	0.403	0.403	0.380	0.342

Table I-9: Estimates¹ of bulk densities under stress of 1.0 and 98.1 (kPa), compressibility C and regression coefficient squared for the two aggregate mixtures at different initial water matric potentials.

Aggre- gates	-98				-392				Initial matric water				potential (kPa)				-20000 ²			
	P ₁	P _{98.1}	"C"	r ²	P ₁	P ₉₈	"C"	r ²	P ₁	P ₉₈	"C"	r ²	P ₁	P ₉₈	"C"	r ²	P ₁	P ₉₈	"C"	r ²
I	0.612	1.344	0.159	0.981	0.704	1.320	0.134	0.976	0.962	1.267	0.071	0.947	1.025	1.244	0.050	0.923				
II	0.499	1.356	0.187	0.967	0.608	1.286	0.149	0.965	0.855	1.209	0.075	0.938	0.705	1.104	0.088	0.915				

¹ The relation used was:

$$P = P_1 + C \ln (P/P_{98.1}) \quad [C, P_1, P_{98.1}] = [\text{mg.m}^{-3}]$$

$$[P] = [\text{kPa.kPa}^{-1}]$$

Table I.10: Gravimetric water contents (%w/w) retained at equilibrium with six levels of matric water potential for the two aggregate mixtures at initial equilibrium with four water, matric potentials and brought to varying bulk densities¹ (four replicates).

Soil moisture	Initial water matric potential (kPa)	Attained ² bulk density (mg.m ⁻³)	Matric water potential (kPa)					
			-20	-33	-98	-196	-392	-1471
A	-98	1.24	22.5±0.7	19.0±0.3	14.2±0.3	12.9±0.4	11.0±0.8	8.2±0.3
		1.42	22.9±0.2	19.4±0.2	15.7±0.2	13.4±0.3	12.4±0.4	8.1±0.1
		1.50	23.5±0.7	19.4±0.6	15.7±0.4	13.9±0.3	13.2±0.2	9.0±0.4
		1.59	24.4±1.2	19.9±0.5	16.3±0.1	14.0±0.2	13.4±0.5	9.3±0.4
	-392	1.23	21.7±0.5	18.4±0.4	14.3±0.4	12.6±0.3	11.5±0.4	7.7±0.4
		1.37	22.0±0.3	18.4±0.3	14.2±0.5	13.3±0.2	12.2±0.3	7.8±0.5
		1.45	22.5±0.2	19.0±0.3	14.6±0.3	13.4±0.2	12.8±0.2	8.3±0.2
		1.52	23.9±0.3	19.3±0.4	15.3±0.3	13.7±0.4	13.1±0.3	8.7±0.5
	-1471	1.21	20.4±0.9	17.7±0.5	13.3±0.7	12.2±0.4	11.1±0.3	8.0±0.3
		1.29	20.5±0.8	18.0±0.3	13.3±0.5	12.5±0.3	11.6±0.3	8.5±0.3
		1.34	21.2±0.7	18.3±0.4	14.1±0.3	12.8±0.2	11.7±0.2	8.2±0.2
		1.39	21.5±0.6	18.4±0.3	14.9±0.4	12.7±0.3	12.0±0.3	8.5±0.2
	-20000	1.19	20.8±1.1	17.3±0.5	12.8±0.6	11.4±0.6	10.2±0.8	8.1±0.2
		1.20	20.9±0.6	17.7±0.5	12.6±0.4	12.1±0.3	10.6±0.6	8.3±0.3
		1.29	21.9±0.7	18.9±0.3	13.0±0.3	12.1±0.4	10.8±0.5	8.3±0.2
		1.32	21.8±0.8	18.5±0.3	13.9±0.3	12.4±0.4	10.9±0.5	8.4±0.3
C	-98	1.28	22.8±0.3	19.1±0.4	15.6±0.4	13.3±0.4	11.6±0.8	8.6±0.9
		1.48	23.3±0.5	19.4±0.3	15.7±0.3	13.4±0.6	11.5±0.7	8.7±0.8
		1.56	23.4±0.4	19.7±0.4	15.7±0.3	13.7±0.3	13.1±0.7	8.6±0.8
		1.64	24.1±0.7	19.8±0.3	16.2±0.2	14.3±0.3	13.3±0.5	9.1±0.5
	-392	1.19	21.7±0.4	18.7±0.4	13.9±0.6	12.2±0.4	11.8±0.8	8.2±0.5
		1.36	32.2±0.6	19.1±0.4	14.1±0.5	13.3±0.5	12.0±0.4	8.0±0.6
		1.48	23.1±0.3	19.4±0.5	15.0±0.4	12.9±0.5	12.3±0.4	8.7±0.4
		1.56	23.5±0.6	19.5±0.4	15.7±0.5	13.4±0.4	12.5±0.3	9.3±0.6
	-1471	1.13	20.5±0.5	17.8±0.7	12.7±0.3	11.9±0.6	10.9±0.4	8.1±0.7
		1.22	20.7±0.5	17.9±0.6	12.8±0.4	11.5±0.5	11.0±0.4	8.0±0.5
		1.31	21.1±0.6	18.3±0.3	13.4±0.5	12.4±0.3	11.7±0.3	8.4±0.8
		1.38	21.9±0.3	18.9±0.3	14.0±0.5	12.5±0.3	12.1±0.5	8.6±0.8
	-20000	1.02	20.1±0.9	17.3±0.4	11.7±0.6	-	9.7±0.8	-
		1.14	20.6±0.7	17.9±0.4	12.7±0.3	-	9.6±0.6	-
		1.21	20.6±0.6	18.2±0.3	13.0±0.5	-	10.1±0.7	-
		1.26	21.1±0.5	19.0±0.4	13.3±0.4	-	10.3±0.3	-

¹ The stresses used to compress the moist to wet aggregate mixtures were 49, 147, 245 and 343 (kPa)

² Coefficient of variation in bulk densities was maximal (-6%) for the driest samples (app. -20,000kPa)

Fig. 1-1: Compression stress [kPa] - soil bulk density [$\text{Mg} \cdot \text{m}^{-3}$] relationships for soils No. I-3 and I-6 under initial soil water. Water contents 6.0 ± 0.8 (o); 9.4 ± 0.9 (+); 12.8 ± 0.8 () and 15.3 ± 0.8 (0), (% w/w basis).

Fig. 1-2: Solid and void matrix of an aggregated bed made of 5.6-8.0 mm aggregates at $11.8 \pm 1.2\%$ (w/w) as function of volume changes under compression.

Fig. I-3: Weighted aggregate size distribution of aggregates mixture A, B, and C of median weighted diameter of 3.1, 5.9 and 10.4 mm, respectively.

Fig. 1-4: Soil water matric potential [kPa] - water contents (% w/w water basis) for mixture A (a-c) compressed to similar soil bulk densities [$\text{Mg}\cdot\text{m}^{-3}$] by different compressive loads at different initial water contents.

Fig. 1-4: Soil water matric potential [kPa] - water contents (% w/w water basis) for mixture C (d-f) compressed to similar soil bulk densities [$\text{Mg}\cdot\text{m}^{-3}$] by different compressive loads at different initial water contents.

Fig. 1-5: Comparison of compression to given soil bulk density between Mixture A and C under different loads of samples at given initial soil water matric potential.

CHAPTER 6

I.2.4. - COMPRESSION OF SOIL AGGREGATES

by
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Report on work undertaken during a 6-month Postdoctoral Fellowship at the
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INTRODUCTION

A seedbed is a surface layer of soil tilled in order to create conditions that promote germination and emergence. However, in such an aggregated state the soil is particularly susceptible to compaction - a change in its volume caused by forces from natural (e.g. drying and wetting) or more especially mechanical (e.g. heavy agricultural traffic) sources. In response to vehicular loads the aggregates are either crushed or rearranged. The soil becomes a denser mass, much less favourable to seedling growth due to a reduction in aeration or water infiltration rate, or to impence of root growth. Maintenance of the seedbed in an aggregated condition is therefore necessary, and a knowledge of tilth load-bearing capacity desirable.

The effects of compression of an aggregated soil depend on the size and strength of the individual aggregates of which it is composed. The tensile strength of single aggregates can be calculated from the force required to crush them between flat, parallel plates (Rogowski, 1964; Dexter, 1975; Hadas and Lennard, 1988). This can then be related to the effects of applied stress on the compaction of aggregate beds (Dexter, 1975; Braunack and Dexter, 1978).

The aims of the present study were to:

1. Determine the aggregate size distribution of an air-dry sample of Loess soil and characterise the tensile strengths of individual aggregates in all size ranges.
2. Compress beds of aggregates from three size ranges at a range of water potentials and compare their behaviour with three previously published model equations.

THEORY

Strength of individual brittle aggregates

Measurement of resistance to crushing between flat, parallel plates (Rogowski, 1964) provides a rapid method for determining the strength of individual soil aggregates over a wide range of sizes, with results which can be interpreted in terms of basic mechanical properties of the aggregate material (Dexter, 1975). Rogowski (1964) assumed that when brittle aggregates are loaded mechanically they deform elastically until failure occurs at some critical level of stress. Assuming the aggregates to be spherical, the stress acting on planes containing both poles is tensile in the centre regions of the sphere and is symmetrical about the polar axis. If F is the diametric loading force at failure and Y the tensile yield strength of the material, then

$$Y = f(F/d^2) \quad [1]$$

where d is aggregate diameter and f is a proportionality factor that has commonly been taken to equal 0.576 or 0.821 (see Hadas and Lennard, 1988). However, for natural, irregular aggregates, the relationship between tensile strength and crushing force may differ from that for spheres. Dexter (1988), using synthetic aggregates with the Brazilian test for cylindrical samples as the standard, recalculated an f value of 1.86.

Rogowski (1964) assumed that diametrically loaded elastic aggregates fail by tensile cracking according to the Griffith (1924) theory of crack propagation. The basis of this theory is that the strength of an aggregate is governed by the presence of naturally occurring cracks, the weakest limiting its strength. It shows that the tensile strength of a material is inversely proportional to the square root of the length, l , of the worst oriented crack:

$$Y \propto l^{-0.5} \quad [2]$$

At its longest l would be equal to the aggregate diameter:

$$Y \propto d^{-0.5} \quad [3]$$

Combination of equations [1] and [3] shows that the crushing load of individual aggregates would vary with diameter as

$$F \propto d^b \quad [4]$$

where $b = 1.5$. However, Dexter (1975) and Braunack and Dexter (1978) both obtained a value for b of approximately 2.0, indicating that their aggregates did not contain any cracks of a size approaching that of the smallest aggregates used.

According to Griffith's theory (1924), at a critical tensile stress the most severe crack spreads sufficiently to produce fracture of the aggregate. Yield strength is thus a random quantity, dependent on the statistical distribution of crack occurrence and magnitude. It is also volume dependent since if these cracks are uniformly distributed with respect to volume then a larger aggregate will, in greater probability, contain a crack of greater severity and hence will fracture at a lower applied stress. Aggregate strength analysis should thus be treated by a probabilistic approach, as follows (from Braunack, Hewitt and Dexter, 1979, and Hadas, 1987, after Lee and Ingles, 1968, and Freudenthal, 1968):

The volume of the aggregate is considered to be composed of a number of equal volume elements, each containing a number of cracks, each of which requires a critical tensile stress, s , to propagate it. The distribution of these critical stresses follows the equation

$$P(s) = 1 - \exp[-(s/s_0)^m] \quad [5]$$

where $P(s)$ is the probability of aggregate rupture under a critical tensile stress, and s_0 and m are constants. The parameter s_0 is the critical tensile strength at rupture of an

aggregate unit volume for which $P(s) = 1 - e^{-1} = 0.63$, whilst m is proportional to the scatter of local crack strengths. Assuming the aggregate total volume, V , to be made up of n volume elements, V_0 , then the probability $P_i(s)$ of its strength being equal to or smaller than s is given by the equation

$$P_i(s) = 1 - \exp[-n(s/s_0)^m] = 1 - \exp[-(V/V_0)(s/s_0)^m] \quad [6]$$

Taking logarithms twice this becomes

$$\ln\{-\ln[1 - P_i(s)]\} = \ln(V/V_0) + m \ln(s/s_0) \quad [7]$$

If a set of j observations of fracture strengths of aggregates of the same volume, arranged in ascending order, are taken to represent the distribution of tensile strengths, then the i^{th} value can be given a cumulative frequency of $P_i(s_i) = i/(j+1)$. The tensile yield distribution can then be found by plotting $y = \ln\{-\ln[1 - i/(j+1)]\}$ against $x = \ln(s)$, where the gradient of the line is m . When such sets of data are plotted for several groups of aggregates of differing volumes, the parallel lines so obtained are known as Weibull lines.

Stresses on individual aggregates in a bed

The load on an individual aggregate in a bed is not exerted simply across a polar diameter as in the standard crushing test, but through all points of contact between the aggregate and those surrounding it (Dexter, 1988). Dexter (1975) investigated the loads required to crush aggregates within beds of differing geometrical configuration. Using synthetic brittle aggregates located between strong glass spheres he tested two different stress systems, similar to those that would be encountered in face-centred cubic (FCC) and close-packed hexagonal (CPH) lattices of spherical aggregates. Fracture of aggregates loaded in this way, although complex, still appears to be

composed of brittle, tensile failures. Both configurations resulted in the loading force on the test aggregate being applied through six points of contact. It was found that the crushing force was greater by a factor I than it was for simple polar loading, where I averaged 1.5 for the two configurations (1.4 for CPH and 1.6 for FCC).

Compression of beds of aggregates

The changes in dimensions or density of small beds of aggregates with applied pressure have been studied by a number of workers (see Dexter, 1988), several of whom have derived equations to model these changes. The three equations to be used in this study were derived as follows:

1. Dexter and Tanner (1973) investigated compression of five remoulded soils, each at several moisture contents, when subjected to isotropic stress in a pressure cell. Bulk density, ρ_b , calculated from the change in sample volume at each pressure increment was converted into particle packing density, D , (the proportion of total soil volume occupied by mineral particles) from the equation

$$D = [\rho_b / 2.66] [(100 - OM) / (100 + w)] \quad [8]$$

where OM is the percentage weight of organic matter in the soil, w is the gravimetric moisture content and 2.66 g cm^{-3} (the density of quartz) is an average density for individual soil particles. Packing density rose asymptotically with applied pressure, P , to a maximum limiting value, a pattern which could be described by the equation

$$D = D_0 + B \exp(-kP) + C \exp(-lP) \quad [9]$$

where D_0 is the maximum limiting value of D , and B , C , k , and l are adjustable parameters. At $P = 0$ the initial packing density is given by $(D_0 + B + C)$, where B and C are negative. The exponents k and l are measures of how rapidly D_0 is attained with

increasing pressure. It was found that whereas B was independent of water content, the absolute value of C increased linearly with it. Since wetter samples on remoulding were seen to contain more aggregates, presumably as a result of water film cohesion, Dexter and Tanner (1973) concluded from their data that the first exponential term in this equation (parameters B and k) corresponded to soil particle rearrangement or flow of material, whilst the second term (parameters C and l) referred to aggregate destruction. The ratio l/k took values of between 10 and 40, indicating that destruction of aggregates occurred at much lower pressures than rearrangement of particles.

2. Dexter (1975) constructed ideal brittle tilths from synthetic aggregates and compressed them uniaxially in a loading frame. Changes in volume of the tilth samples were used to calculate changes in packing density, D, with applied pressure, P. Values of P were rendered dimensionless by dividing them by the tensile yield strength, Y, of individual aggregates. It was observed that only half of the aggregates were crushed during compression, and that material from these had flowed plastically into spaces between the remaining intact aggregates. The overall compression process was described by the equation

$$D = D_i - (D_i - D_f)[1 - \exp(-kP)].\Phi(P/Y) \quad [10]$$

where D_i and D_f are initial and final values of D, the exponential term describes the plastic flow of broken aggregates (k is a parameter to be determined), and $\Phi(P/Y)$ is a statistical function describing the proportion of aggregates broken as a function of dimensionless stress, P/Y. Substituting the latter with the equation for the probability of aggregate rupture (equation [5]) then

$$D = D_i - (D_i - D_f)[1 - \exp(-kP)][1 - \exp(-(P/Y)^n)] \quad [11]$$

3. Braunack and Dexter (1978), using natural soil aggregates at a range of water potentials, stressed beds uniaxially, recording their height, H, as a function of applied

pressure. Bed height was rendered dimensionless by dividing H by the initial height, H_i , and the decrease in H/H_i with increasing P/Y could be described by the empirical equation

$$H/H_i = A + B \exp[C(P/Y) - D(P/Y)^{0.5}] \quad [12]$$

where $A = H/H_i$ at $P/Y = \infty$, $B = 1 - A$, and C and D are adjustable parameters describing how rapidly H/H_i decreased with increasing P/Y . They found A to be almost constant at 0.4, and only small changes in C and D occurred with water content and aggregate size for unsaturated soil. They therefore proposed a representative compaction equation

$$H/H_i = 0.4 + 0.6 \exp[0.017(P/Y) - 0.38(P/Y)^{0.5}] \quad [13]$$

to be used as a working equation describing the uniaxial compaction of beds of aggregates of any size and water content as a function of dimensionless uniaxial stress. This was later revised by Dexter (1988) to

$$H/H_i = 0.4 + 0.6 \exp[0.055(P/Y) - 0.683(P/Y)^{0.5}] \quad [14]$$

to account for Y values recalculated using his f value of 1.86 in equation [1]. The value for A of 0.4 derives from the fact that A is approximately equal to the reciprocal of the maximum density of soil particles, 2.66 g cm^{-3} , the density of quartz.

MATERIALS AND METHODS

A sample of air-dry Loess soil (a silty loam) was sieved to produce aggregate fractions within the size ranges >25.0, 16.0 - 25.0, 11.2 - 16.0, 8.0 - 11.2, 5.6 - 8.0, 4.0 - 5.6, 2.8 - 4.0, 2.0 - 2.8, 1.7 - 2.0, 1.0 - 1.7 and <1.0 mm diameter. Each fraction was then weighed, and the aggregate size distribution determined by plotting the percentage of sample smaller than a given diameter on log-probability paper (Gardner, 1956). Geometric mean diameter (GMD) is the size corresponding to 50% of the sample undersize, but was also calculated separately using the equation

$$\text{GMD} = \exp\left\{\frac{\sum_{i=1}^n [W_i \ln(X_i)]}{\sum_{i=1}^n [W_i]}\right\} \quad [15]$$

where W_i is the weight of aggregates in a size fraction of average diameter X_i , and $\sum_{i=1}^n [W_i]$ is the total weight of the sample (Kemper and Chepil, 1965).

Omitting the >25 and <1 mm diameter size ranges, 20 approximately spherical aggregates from each fraction were selected and their diameters measured along three axes using Vernier calipers. A mean diameter for each aggregate was then calculated and a mean for the 20 aggregates within each size range.

Determination of aggregate tensile yield strength

Individual aggregates were placed, flattest side downwards, between the flat parallel plates of an unconfined compression apparatus (Fig. 1(a)). Aggregate diameter, d (cm), was recorded as the height between the plates, and the applied compressive force at fracture, F (kg converted to N), then determined. Tensile yield strength, Y (kPa), was calculated using equation [1] with an f value of 0.576, and a mean for the 20 aggregates within each size range determined. A plot of F against d (log scales)

allowed the value of b in equation [4] to be found.

The gravimetric moisture content of the air-dry aggregates after crushing was also determined.

Compression of aggregate beds

Three aggregate size ranges were selected, 2.8 - 4.0, 8.0 - 11.2 and 16.0 - 25.0 mm diameter, referred to here as small, medium and large aggregates respectively. The air-dry aggregates were poured or placed into steel rings (approximately 80 mm in diameter and 50 mm deep with a gauze base), one ring for each size range. The rings were gently tapped in order to allow the aggregates to settle, and an attempt was made to fill them to the top with as level a surface as possible. The aggregates were then allowed to wet to saturation by capillary action, before being brought to their desired water potential. A sintered glass funnel with a suspended water column was used to obtain potentials of -6 and -10 kPa, whilst for -20 and -50 kPa the rings were placed in pressure cells (water forced out by air pressure). A fifth set of beds contained air-dry aggregates.

Each bed was compressed uniaxially in the compression apparatus (Fig. 1(b)). The initial height, H_i , of the bed was recorded, then a 3.8 kg weight placed on top and the new height, H , recorded. This weight was then compressed into the aggregate bed in increments of applied force, up to a maximum of approximately 250 kPa of pressure on the wetted beds and 500 kPa on the air-dry aggregates. The height of the bed after each increment of pressure was converted into dimensionless bed height, H/H_i , and sample volume. The oven-dry weight of the compressed soil sample was determined at the end, and bulk density after each pressure increment thus calculated. Particle packing density,

D, was calculated using equation [8], in which the percentage weight of organic matter in the soil, OM, was taken to be zero, and the gravimetric moisture content, w , was estimated from previously determined soil moisture characteristic curves, except for air-dry soil where the measured value was used.

Two replicate aggregate beds were compressed at water potentials of -6, -10 and -20 kPa (and mean results from the two calculated) whilst at -50 kPa and air-dryness only one replicate was available. Plots of H/H_i and D against applied pressure, P , were produced for each aggregate size range and water potential. A non-linear regression procedure was then used to fit the data to equations [9], [11] and [12]. For equations [11] and [12], first approximations of Y for wetted aggregates were calculated from previously determined tensile strength relationships, but for air-dry aggregates the measured values were used. Goodness of fit of the experimental data to the fitted equations was assessed and parameter estimates compared with those published.

RESULTS AND DISCUSSION

A log-normal distribution of aggregate size (Fig. 2) appeared to produce a straight line, though with some deviation at the extremes of the diameter range as might be expected (Gardner, 1956). This linearity on log-probability coordinates is in agreement with the majority of published aggregate size distributions (Gardner, 1956). The fitted line shown in Fig. 2 indicated a GMD of approximately 6.0 mm, whilst equation [15] gave a value of 6.8 mm. Mean diameters for the 20 selected aggregates from each size range are given in Table 1, and use of these values in equation [15] gave a slightly higher GMD of 7.4 mm. The latter is probably the more accurate of the three figures quoted since it uses aggregate diameters measured along three axes, whereas a single diameter determined purely by sieving non-spherical aggregates will almost certainly be smaller.

Aggregate tensile yield strength

On compression between flat, parallel plates the majority of the air-dry aggregates fractured perpendicularly into two pieces. Mean aggregate height between the plates (d) and applied compressive force at failure (F), and the tensile yield strength (Y) calculated from the two, are given in Table 1. The relationship between Y and d (Fig. 3) includes 95% confidence intervals showing the wide variation in strength measurements within each size range. The accuracy of the strength measurements for the two smallest size ranges (1.0 - 1.7 and 1.7 - 2.0 mm diameter) is doubtful due to the difficulties involved in determining the point of fracture of such small aggregates. If these two size ranges are therefore omitted, Fig. 3 indicates a clear trend of increasing

gth with decreasing aggregate size. Indeed five out of the seven points lie almost on a straight line; Fig. 3 shows one fitted through all seven points, and those at extremes of this line exhibit significant differences. This increase in tensile strength as aggregate size decreased was also observed by Braunack *et al.* (1979) and Hadas (1987), but Braunack and Dexter (1978) found values of Y to be independent of aggregate diameter. The relationship between F and d (log scales) is shown in Fig. 4. The gradient of the fitted line gave a value for b in equation [4] of 1.8 if all the points were included, or 1.6 if the two smallest aggregate size ranges were again omitted. These values are similar to the 1.7 or 1.8 obtained by Hadas and Lennard (1988) and 1.7 calculated by Braunack and Dexter (1978) from data of Martinson and Olmstead (1949). All these figures are intermediate between the value of 2.0 obtained by Dexter (1975) and Braunack and Dexter (1978) which implies no cracks of a size approaching that of the smallest aggregates used, and the theoretical prediction of 1.5 by Rogowski (1964) which implies large cracks.

Using the probabilistic approach to aggregate strength analysis (equation [7]), Fig. 5 shows a plot of $P(Y)$ against $\ln(Y)$ for the three aggregate size ranges which were selected for compression in beds, 2.8 - 4.0 (small), 8.0 - 11.2 (medium) and 16.0 - 25.0 mm diameter (large). Weibull lines similar to those obtained by Hadas (1987) are apparent, with m values of 2.2, 2.0 and 2.0 respectively. These values are of similar order to those obtained by both Braunack *et al.* (1979) and Hadas (1987). The order of the lines again shows how values of Y diminish as aggregate size increases.

The gravimetric soil moisture content of the air-dry aggregates determined after washing was found to be approximately 3.5%.

particle packing density than medium or large ones.

The relationship between dimensionless bed height (H/H_0) and applied pressure for medium size aggregates at all five water contents is shown in Fig. 8. A similar pattern of curves was found for the small and large aggregates in that -6 and -10 kPa compressed more than -20 and -50 kPa, though the order within each of these pairs was not consistent. A comparison between small and large aggregates at potentials of -6 and -20 kPa and air-dryness is given in Fig. 9. Air-dry aggregates compressed much less than wet ones at all three sizes, with bed height decreasing only slowly (Figs 8 and 9 again show data up to 250 kPa only). However, the form of the curves for wetted aggregate beds follows that recorded by Braunack and Dexter (1978). They observed no significant difference in compression behaviour between beds of different size aggregates, and Fig. 9 shows that there was little difference between aggregate sizes in air-dry aggregate beds. Amongst the wetted beds small aggregates appeared to compress less than medium or large ones but the order of the latter was not consistent.

The data was then fitted to the three model equations:

$$1. \quad D = D_0 + B \exp(-kP) + C \exp(-lP) \quad [9]$$

(Dexter and Tanner, 1973)

In the non-linear regression procedure five parameters were set, D_0 , B , C , k and l , each with boundary limits to prevent unrealistic values (B and C must be negative, k and l positive and $D_0 < 0.8$).

The fit of the equation to the data appeared to be good in all cases (Fig. 10 shows an example of medium size aggregates at -10 kPa). Table 2 gives mean parameter estimates from all water contents within each size range (not including values that reached their upper boundary limits). The figures are generally of a similar order of magnitude to those of Dexter and Tanner (1973) although values for k and the ratio l/k

are higher. In only a few instances did B , C , k and l reach their boundary limits, but D_0 tended to reach its upper limit with the small aggregates, again indicating that the pressure applied was not high enough in order for a natural plateau to be attained. The largest error limits on parameter estimates were also found within this size range. Table 3 gives an example of parameter estimates for medium size aggregates at each water content (means of two replicates where available). The different plateaux evident in Fig. 6 are reflected by the increase in D_0 as water content decreases within the wetter range, but these differences are not significant so do not necessarily disagree with Dexter and Tanner (1973), who observed D_0 to be independent of moisture content. They also observed an increase in the absolute value of C with increasing water content, whilst B was seen to be independent of it. The remaining parameters in Table 3 show no clear trends though. Using the mean estimates of B , C , k and l from all five water contents within each size range, values for the two exponential terms in equation [9] were calculated (Table 2). These give an indication of the relative importance of the two physical processes that occur within beds of compressed aggregates (Dexter and Tanner, 1973). Whereas soil particle rearrangement was more important than aggregate destruction in beds of small and medium size aggregates (first exponential term largest), it had become much less so in beds of large ones. Since the tensile strength of the small and medium size aggregates was higher than that of large ones this might be expected.

There were considerable differences between techniques used in the work described here and those of Dexter and Tanner (1973), namely that they compressed remoulded soil isotropically whereas in this work natural aggregates were compressed uniaxially. However, such differences have not prevented the data obtained here from being in general agreement with the published equation.

$$2. \quad D = D_i - (D_i - D_o)[1 - \exp(-kP)][1 - \exp(-(P/Y)^m)] \quad [11]$$

(adapted from Dexter, 1975)

The parameters set were k and m (k must be positive and $m > 0.1$) together with Y at all water contents except air-dryness. The value of D_i was taken as that of D_o calculated for equation [9], and was also made a variable in cases where D_o was previously at its limit of 0.8.

The fit of the equation to the data was good in most cases, but poor in a few instances with the small aggregates. Table 4 gives mean estimates of k and m from all water contents within each size range, and Table 5 estimates for medium size aggregates at each water content. The values for k appear to be generally much higher than the 0.006 obtained by Dexter (1975). However, he used dry synthetic brittle aggregates, and the value for medium size air-dry aggregates (Table 5) is much closer to the published figure. Parameters m and Y tended to reach their lower boundary limits with small aggregates. When D_i was made a variable, however, it stayed within its boundary limits. In contrast to k , the values for m were much lower than the figure of approximately 2.0 expected. Even if the values for single air-dry aggregates are divided by $I = 1.5$ to allow for the greater crushing force experienced in beds (Dexter, 1988), $2.0/1.5 = 1.3$ is still much higher than the figure of 0.3 determined here.

$$3. \quad H/H_i = A + B \exp[C(P/Y) - D(P/Y)^{0.5}] \quad [12]$$

(Braunack and Dexter, 1978)

Initially four parameters were set, A , B , C and D (all positive), with Y fixed, but in a second set of regressions Y was included as a fifth variable.

The fit of the equation to the data was generally reasonable, but not as good as with the previous two equations (Fig. 11 shows an example of medium size aggregates at -10 kPa). Several of the fits were improved by making Y a variable, but error limits

on parameter estimates became much larger. Table 6 gives mean parameter estimates (Y fixed) from all water contents within each size range, and Table 7 estimates for medium size aggregates at each water content. The parameter A is the value of H/H_i at $P/Y = \infty$ and thus indicates where the curves shown in Figs 8 and 9 level off. Fig. 9 showed that amongst the wetted beds small aggregates compressed less than large ones, and this is reflected in Table 6 by an apparent decrease in the value of A (and consequent increase in the value of B) as aggregate size increases (within individual replicates $A + B$ approximately equalled 1.0). The value of C often tended to reach its lower boundary limit of zero, although not when Y was made a variable, and also appeared to increase with aggregate size. These differences were not significant, however, and means from all three size ranges gave the general equation

$$H/H_i = 0.35 + 0.60 \exp[0.041(P/Y) - 0.81(P/Y)^{0.5}] \quad [16]$$

The values for A and B in equation [16] are approximately the same as the 0.4 and 0.6 found by Braunack and Dexter (1978), while C and D correspond more closely to the 0.055 and 0.683 calculated by Dexter (1988).

The techniques used in the work described here closely resemble those of Braunack and Dexter (1978), who also uniaxially compressed beds of natural soil aggregates. A similar conclusion appears to have been reached too, namely that there was no significant difference in the compression behaviour of beds of different size aggregates.

CONCLUSIONS

Soil strength is very important in agriculture. A soil must be both weak enough to be easily tilled and strong enough so that its structure can withstand mechanical stresses and those caused by natural forces such as rainfall. In order to determine the optimum conditions of soil strength and structure it is necessary to have accurate measures of soil strength, both for bulk soil and for individual aggregates. The present study has provided a contribution to the requirement expressed by Dexter (1988) for more experimentation on the compression of beds of aggregates of different soil types, water contents and size distributions. It has allowed a comparison of three existing model equations that describe the effect of applied pressure on the behaviour of aggregate beds. Equation [9] fitted the data best, despite the techniques used by Dexter and Tanner (1973) being most dissimilar to those used here. Equation [11] (from Dexter, 1975) was also a good fit, although again there were some differences in techniques. The methods of Braunack and Dexter (1978) were in closest agreement but equation [12] fitted the data least well. The reason for this may be the fact that equation [12] was an empirical one derived simply to fit the data of one soil, whereas equations [9] and [11] have some physical basis to them and as such are more likely to be applicable to a range of soils and conditions. There was a clear effect of water content on the compression behaviour of the aggregate beds, with wetted ones compressing to a much greater degree than air-dry ones. However, no significant difference was found between beds of different size aggregates. Both these observations agree with those previously reported (Dexter and Tanner, 1973; Braunack and Dexter, 1978). The increase in tensile yield strength of individual air-dry aggregates observed with decreasing aggregate size is in agreement with Braunack *et al.* (1979) and Hadas

(1987) but not with Dexter (1975) and Braunack and Dexter (1978). The implication here is that smaller aggregates will have a greater load-bearing capacity, and although compression of beds indicated that this might be true, differences were not significant. If the load-bearing capacity of an aggregate bed is independent of aggregate size, then agricultural tilths could be produced with aggregate sizes that optimize other soil physical properties (Braunack and Dexter, 1978), for example to obtain maximal seed-soil contact. In practice, however, within a bed of aggregates there will be a wide range of aggregate sizes, each having a wide range of strengths. Thus a sudden onset of crushing is unlikely, but rather a more gradual one (Dexter, 1988).

In this study three uniform aggregate size ranges were tested, at water potentials mostly within the wetter region, and an increase in the number of size ranges, water potentials and replicates may have produced a more detailed picture. Ideally tensile yield strength of individual aggregates should have been determined at a range of water potentials, although this is not easily done in the wetter range. In addition, a combination of uniaxial and shear stresses would have more accurately reflected those experienced under vehicular loads. However, this work has provided a useful insight into the effects of compression of soil aggregates and could be used as a starting point for a further study on how the resulting changes in structure of a seedbed, for example with regard to pore size, would affect seedling establishment.

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Table 1. Size and strength of air-dry Loess soil aggregates (means from 20 within each size range).

Size range, mm	Diameter (3 axes), mm	Height between plates (d), mm	Compressive force at failure (F), N	Tensile yield strength (Y), kPa
16.0 - 25.0	21.4	18.5	114.3	194.2
11.2 - 16.0	14.6	12.5	74.6	273.6
8.0 - 11.2	9.9	8.8	32.2	234.0
5.6 - 8.0	7.3	6.5	25.6	357.7
4.0 - 5.6	5.4	4.7	14.3	379.9
2.8 - 4.0	3.8	3.4	9.8	484.4
2.0 - 2.8	2.9	2.5	4.3	406.3
1.7 - 2.0	2.0	1.7	1.7	331.8
1.0 - 1.7	1.5	1.2	0.8	292.3

Table 2. Parameters for equation [9] : means from all water contents within each size range, not including values which reached their upper boundary limit.

Parameter	Size range , mm		
	2.8 - 4.0 (small)	8.0 - 11.2 (medium)	16.0 - 25.0 (large)
D_0	0.670	0.506	0.508
B	-0.276	-0.171	-0.113
C	-0.148	-0.096	-0.121
k	0.0082	0.0086	0.0101
l	0.0638	0.0808	0.0327
l/k	179.4	59.4	11.5
$B \exp(-k)$	-0.27	-0.17	-0.11
$C \exp(-l)$	-0.14	-0.09	-0.12
Ratio	1.9	1.9	0.9

Table 3. Parameters for equation [9] : medium size aggregates at all water contents
(means from two replicates at -6, -10 and -20 kPa except where indicated
*, signifying that one of the replicates had reached its boundary limit and
is therefore not included).

Parameter	Water potential , kPa				
	-6	-10	-20	-50	Air-dry
D_0	0.449	0.466	0.511*	0.623	0.583
B	-0.121	-0.103	-0.278	-0.096	-0.268
C	-0.082	-0.126	-0.070	-0.213	0
k	0.0135	0.0098	0.0106	0.00063	0.00066
l	0.106	0.051	0.037*	0.014	0.200
l/k	7.9	5.2	3.5	21.7	301.2

Table 4. Parameters for equation [11] : means from all water contents within each size range, not including values which reached their boundary limits.

Parameter	Size range , mm		
	2.8 - 4.0	8.0 - 11.2	16.0 - 25.0
k	0.023	0.402	0.453
m	1.134	0.503	0.495

Table 5. Parameters for equation [11] : medium size aggregates at all water contents
(means from two replicates at -6, -10 and -20 kPa except where indicated
*, signifying that one of the replicates had reached its boundary limit and
is therefore not included).

Parameter	Water potential , kPa				
	-6	-10	-20	-50	Air-dry
k	2.027*	0.353	0.0323	0.0144	0.00096
m	0.646	0.710	0.257	<0.1	0.301

Table 6. Parameters for equation [12] : means from all water contents within each size range when Y was a fixed value, not including values which reached their upper boundary limit.

Parameter	Size range , mm		
	2.8 - 4.0	8.0 - 11.2	16.0 - 25.0
A	0.463	0.303	0.279
B	0.546	0.603	0.635
C	0.0115	0.0238	0.0883
D	0.701	0.880	0.847

Table 7. Parameters for equation [12] : medium size aggregates at all water contents
(means from two replicates at -6, -10 and -20 kPa except where indicated
*, signifying that one of the replicates had reached its boundary limit and
is therefore not included).

Parameter	Water potential , kPa				
	-6	-10	-20	-50	Air-dry
A	0.532	0.232	0.223	0.419	0.030
B	0.471	0.543*	0.777	0.581	>1
C	0	0.095	0	0	0
D	1.561	0.969	0.554	0.740	0.135

Fig. 1. SoilTest loading frame as used for
(a) unconfined compression of single aggregates, and
(b) uniaxial compression of aggregate beds.
(Not to scale.)

Fig. 2. Log-probability plot of the aggregate size distribution of Loess soil. (Fitted line.)

Fig. 3. Relationship between tensile yield strength and diameter of air-dry Loess soil aggregates. (Fitted line; open symbols only.)

Fig. 4. Relationship between applied force at fracture and diameter of air-dry Loess soil aggregates. (Fitted line; open symbols only.)

Fig. 5. Plot of $P_t(Y)$ against $\ln(Y)$ for small ($\circ$), medium ($\circ$) and large ($\square$) aggregates. (Fitted lines.)

Fig. 6. Increase in particle packing density with applied pressure for medium size aggregates at five water potentials: -6 (○), -10 (○), -20 (□) and -50 kPa (□) and air-dryness (Δ).

Fig. 7. Increase in particle packing density with applied pressure for small (open symbols) and large aggregates at three water potentials: -6 (o) and -20 kPa (□) and air-dryness (Δ).

Fig. 8. Decrease in dimensionless bed height with applied pressure for medium size aggregates at five water potentials: -6 (o), -10 (o), -20 (o) and -50 kPa (o) and air-dryness (Δ).

Fig. 9. Decrease in dimensionless bed height with applied pressure for small (open symbols) and large aggregates at three water potentials: -6 kPa ($\circ$) and -20 kPa ($\square$) and air-dryness (Δ).

Fig. 10. Fit of equation [9] to data for medium size aggregates at -10 kPa.

Fig. 11. Fit of equation [12] to data for medium size aggregates at -10 kPa.

DESCRIPTION OF COOPERATION

The level of cooperation between the Investigators was in general good. Work in Israel concentrated on field studies of soil fragmentation and evaluating the behavior of aggregate beds. Research in Puerto Rico concentrated on theoretical analysis of fracture and development of a laboratory index of friability which would correlate well with field soil behavior. The efforts in both countries complemented each other well. Some problems were encountered with communication. Although FAX facilities were available at both institutions, it was often very difficult to establish linkage. As a result most communication was conducted by mail.

A visit by Dr. Hadas to Puerto Rico was conducted in the fall of 1990, followed with a visit to Israel by Dr. Snyder in the summer of 1991.

EVALUATION OF RESEARCH ACHIEVEMENTS WITH RESPECT TO THE ORIGINAL RESEARCH PROPOSAL, WITH EMPHASIS ON BENEFITS FOR AGRICULTURE

I. ACHIEVEMENTS WITH RESPECT TO OBJECTIVE 1

As mentioned in the Introduction, Objective 1 stated in the original proposal was modified in lieu of the need to further understand the mechanisms of soil fracture in tillage, and to improve existing field and laboratory methods for characterizing soil friability. Achievements with respect to these new objectives are summarized below.

I.1 Evaluation of fracture mechanisms in tillage:

Field observations of fragmentation in this study tended to confirm previous suggestions by Wolf and Hadas (1987) that the classical Mohr-Coulomb shear failure criterion is often, if not generally, insufficient to explain the stresses and energy expenditures required in tillage and the soil fragmentation patterns which result. Appearance of the characteristic shear failure planes commonly observed in tillage seems to be preceded by a process of crack growth and branching which involves physical principles quite different than simple Coulombic shear plane slippage.

In order to account for such behavior, a simple two-stage model of soil fragmentation has been formulated, based on established principles of fracture mechanics. The first stage of

the model is a period of stable crack growth which allows fracture at many places within the soil mass rather than at only one or two points of extreme weakness. This stage is made possible by a state of combined compression/shear as the soil is approached by the leading edge of a tillage implement. The amount of fracture produced seems to be controlled by the spatial/strength distribution of soil structural flaws and the degree of soil plasticity. The second stage is one of unstable fracture, induced by a state of tension produced when the soil is forced to flow in a divergent manner over a convex surface at the trailing edge of the tillage implement. At this point catastrophic failure occurs around large cracks formed during the first stage, leading to desintegration of the soil mass. An all-important ingredient in the whole process is a constant supply of energy provided by movement of the tillage implement, without which crack surfaces cannot form. The total amount of energy available depends on the stress developed in the soil (a function of soil mechanical strength) and on the dimensions of the tillage implement.

If the above fragmentation theory is correct, it points out the need to consider principles of fracture mechanics, as modified by plasticity, to a much greater extent than in the past in order to fully understand the tillage process. The role of a tillage implement in achieving soil fragmentation may have to be considered in a very different light than in the present. This could lead to significant improvements in tillage implement design criteria, with obvious benefits to agriculture.

I.2 Improvement of Field and Laboratory Methods for Characterizing Soil Friability

I.2.1 Field characterization of soil tilth

The most common methods of characterizing soil tilth usually involve measuring the average distribution of sizes and/or strengths of aggregates in the entire tilled layer. However, results of this study point out that systematic variations of such properties occur within the tilled layer. This variability needs to be characterized.

In response to this need, a promising technique has been developed in which spatial variation in size distribution of aggregates is inferred from spectral and geostatistical analysis of electronically recorded cone penetrometer data measured as a function of depth and horizontal location. The procedure can be automated without too much difficulty, which makes it economical in terms of time and manpower.

I.2.2 Laboratory Characterization of Soil Friability

Field and theoretical investigations in this project have pointed out serious limitations in current laboratory methods for estimating soil friability, as well as general lack of methods for measuring more basic soil properties related to fracture. Significant contributions in these areas were achieved during the course of the project.

A novel yet very simple apparatus has been developed for

simulating soil fragmentation in tillage. The primary fracture-inducing characteristics of tillage are preserved in the method, namely a displacement-controlled loading process and the capability for controlling compression/shear ratio. Differences in fragmentation of undisturbed field samples as a function of variable compression/shear ratio and soil moisture content can be evaluated rapidly either in the laboratory or the field. When suitably instrumented, the apparatus also provides comparisons of energy efficiency of fracture and indices of plastic deformation occurring during the fracture process. Preliminary tests indicate that the point of maximum soil friability estimated by the method correlates well with maximum friability as determined by field tillage experiments.

In response to the need for better understanding of basic physical processes involved in the fracture of soils with variable plasticity, a literature review of recent theoretical and experimental advances in the study of fracture in elastic/plastic materials was conducted. The review disclosed the existence of very useful concepts and techniques which, although well known in the material sciences disciplines, do not appear to be familiar to soil scientists. Preliminary experiments indicated that many of these techniques can with relative ease be used to study the interaction between soil plasticity and its fracture properties under a variety of loading conditions. Such studies could in turn provide valuable information for improved design of tillage implements and determining the proper timing of tillage.

II. ACHIEVEMENTS UNDER OBJECTIVE II.

A considerable number of experiments was conducted on the compression and shear behavior of aggregate beds, with some attention also given to hydraulic properties of the deformed beds. Highlights of results are given below.

II.1 Compression and shear behavior of aggregate beds

Results generally indicated that, at moisture contents corresponding to the 0 -1400 KPa matric potential range, variation in aggregate size had very little effect on compression and shear behavior of aggregate beds, as long as aggregate size distributions were similar in shape and intra-aggregate structure was the same. These results are in general agreement with other results reported in the literature. They are also consistent with predictions of stress scaling analysis, when it is assumed that inter-aggregate pore water and gravity have negligible effects on effective stresses in unsaturated soils. An important implication for agriculture is that decisions on the most appropriate aggregate size to be achieved by tillage need not consider optimization of mechanical behavior. This provides freedom to optimize aggregate size for other factors such as adequacy for plant root growth and transport processes.

Analysis of inter-aggregate contact points in moist aggregate beds after compression, and comparison of final aggregate density with average whole soil density under similar loading conditions, suggested that aggregates in beds with narrow aggregate-size

distributions are subjected to greater stress than in a medium with a wide size distribution. This is apparently a result of smaller solid cross-sectional area under narrow size distributions, which causes higher stress on individual aggregates. Furthermore, the greater porosity in unimodal aggregate beds results in less confinement of aggregates which facilitates both shear and tensile failure. This implies that although variation in aggregate size often does not play a role in mechanical behavior of aggregate beds, changes in shape of the size distribution function do play a role.

The general independence between aggregate size and bed behavior tended to break down in very dry soils, with greater shear-dilatancy and resistance to compression observed in beds of larger aggregates. The particular reasons for these results is not known at present.

Aggregate beds at different initial water content were compressed under different loads, and internal structure was analyzed by visual inspection techniques. Initial macroscopic volume reductions associated with increasing load were not accompanied by a corresponding decrease in intra-aggregate porosity, suggesting aggregate fracture as a major mode of soil failure. However, beyond a stress of about 98 KPa, a steady reduction in intra-bed porosity occurred, indicating the involvement of plastic deformation in the volume reduction process. The point at which plastic deformation begins in aggregates in loaded aggregate beds has been the object of

speculation and needs to be investigated further.

II.2. Shear behavior of whole soils

Shear studies of whole soils with varying texture and initial water content indicated only a small dependence of the Mohr-Coulomb shearing angle on either soil water content or texture. The cohesion intercept, on the other hand, tended to show a significant increase with decreasing water content and increasing clay content. These observations are in general agreement with an effective stress-Coulombic friction model of soil strength which varies with particle size distribution in a way consistent with capillary scaling theory, as postulated in the project proposal. However, colloidal and particle interlocking effects which are not accounted for in scaling theory also appeared to play a role. Higher shear angle values were encountered in coarser textured soils where the possibility of particle interlocking was greater and where lubricating water layers between sliding clay packets were largely absent. The possibility of using a particle size index in addition to, or in lieu of, soil water matric potential as a state variable in the study of colloidal soils is briefly discussed.

II.3. Effects of stress and moisture history on soil state variables.

Soil moisture release characteristics were determined on aggregate beds which had previously been compacted to specific bulk densities along different load and moisture content paths. The shapes of the moisture release curves were dependent only on bulk

density, and were independent of aggregate moisture content at time of compaction or the stress required to reach a given density. These results tend to agree with one of the working hypotheses of this Project, namely that the relation between void ratio, water content and matric potential can be treated as a state function of a soil as long as water content is varied along a monotonically decreasing matric potential path beginning with a saturated soil. This finding needs to be tested under a much wider variety of soil conditions in order for its significance to be evaluated.

III. ACHIEVEMENTS UNDER OBJECTIVE 3.

Relatively little work was conducted on effects of wetting and drying cycles on tilth mellowing. However, some general observations were made which suggest avenues for further research. It appears that shrinking and swelling by itself is only effective in achieving fracture of clay soils with very high colloidal activity, e.g. vertisols. Very sandy soils seem to easily fall apart under their own weight when wet. Other "intermediate" soils, however, seem to require a combination of shrinking and swelling together with additional sources of energy such as raindrop impact and stress anisotropy due to effects of gravity on irregularly shaped aggregates supported at only a few contact points.

LIST OF PUBLICATIONS RESULTING FROM THE PROJECT

Hadas, A. 1990. Directional strength of aggregates as affected by aggregate volume and by a wet/dry cycle. J. Soil Sci. 41:85-93

Hadas, A. and I. Shmulewich. 1990. Spectral analysis of cone penetrometer data for detecting spatial arrangement of soil clods. Soil and Tillage Res. 18:47-62