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Soil Aggregates and Their Influence on Soil Compaction and Swelling

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Prediction of the characteristics and properties of compacted fine-grained soils is much aided by a physical soil mechanism or model. This model should, as nearly as possible, fit the observed soil conditions during and after compaction. This paper describes an extension to existing soil compaction models and uses it to explain the behavior of kaolinite compacted in the laboratory by static pressures under conditions of no lateral strain. The experimental investigation included an examination of the kaolinite aggregations at the compaction moisture content but before compaction. This was followed by the determination of the relationship between the net energy input during compaction and the compacted unit weight. Finally, constant-volume swelling-pressure measurements were made on selected compacted samples. The swelling pressures were monitored continuously after giving the samples access to water; the results are presented as swelling pressure versus time relationships. The experimental results confirm the appropriateness of a deformable-aggregate soil model to explain the compaction of kaolinite as prepared in the laboratory and then compacted statically. The model is also appropriate for understanding the constant-volume swelling-pressure pattern that develops on wetting the compacted soil.

The objective of the research described in this paper was to develop a model and a mechanism that can adequately explain the achievement of compacted unit weight for kaolinite statically compacted in the laboratory. Such an explanation should be complete enough to explain the condition of the soil before compaction, the interactions within the soil mass during compaction, and the observed behavior of the compacted soil.

The model hypothesized was one in which the soil is made up of macroscopic aggregations of clay particles. During compaction, it is the interactions of these aggregates, their deformation characteristics, and their ability to fit together in a compact mass that determine the end result unit weight for a given type of compaction and amount of effort. It is this same compacted macrostructure—an assemblage of aggregates—that, to some extent, determines the engineering behavior of the compacted soil.

The experimental approach was to study certain of the properties of the soil aggregates before compaction, monitor the compaction effort, and then subject the compacted samples to a test significant to engineering practice. The achievement of compaction was examined by calculating the energy required to densify the soil continuously from a low unit weight to the final unit weight achieved for each sample. The correctness of the model was determined by analysis of existing results, especially stress-strain and volumetric swelling, and from the results of constant-volume swelling-pressure determinations on the compacted samples. [The relevant literature is reviewed elsewhere (1).]

COMPACTION-MECHANISM HYPOTHESIS

This paper attempts to explain the compaction of fine-grained soils in terms of soil structure and changes in it. The explanation is based in part on the results of previous investigators, principally Lambe (2, 3) and Olson (4), and on tests on a single commercial clay (Edgar Plastic Kaolin) in a single compaction mode (static). The hypothesis states the necessity of recognizing (a) that there are a number of fabric levels and (b) that the stress history before compaction can strongly influence the compaction result. The postulated explanation for laboratory static compaction of kaolinite, from dry soil to the as-compacted state, is described below.

Initial Soil Structure

Before the addition of water to kaolinite, the layers of adsorbed water present on the particles are no more than a few molecules thick. In this state, many of the particles, each being many unit layers thick, are flocculated in face-to-face fashion due to Van der Waals' forces and, probably, hydrogen bonds (which had developed as the Van der Waals' forces brought the particles closer together). Each group of face-to-face particles can be thought of as a domain. Depending on the amount of hydrogen bonding, the adsorbed water will more likely be found on the domain surface rather than being uniformly distributed around each flocculated particle in the packet. Figure 1 presents an idealized representation of the effects of adding water to the soil. In Figure 1a, there are three air-dry domains. The relative spacing between the domains is quite variable and, for the mixing technique used in this research, is constantly changing, but the orientation and spacing of the particles within each domain is relatively constant. Each particle within a domain is many units thick (not shown), and these repeating units are permanently fixed with respect to each other in the particle.

The usual procedure is to add water to the air-dry soil, either in increments or continuously, while agitating or mixing so that the moisture is, in a gross sense, evenly distributed throughout the soil. This can be done manually or, as was done in this research, by using a mechanical blender.

As the soil domains come into contact with the water mist, external double layers begin to form and, depending on the degree of bonding within the domain, water is adsorbed between the face-to-face particles and a slight swelling of the domain may occur (see Figure 1b).

Figure 1c shows the condition as more water is added. The external double layers continue to develop, and the domains are held to one another in edge-to-face arrangement to form aggregates as they collide. This is principally due to Van der Waals' attraction and, to some extent, mutual desire for the water on the surfaces of the domains, because there is a large net water deficiency in the system. As the agitation continues, the flocculated groups of domains grow in size and decrease in number and, as the addition of water continues, the water will inevitably be unevenly distributed among the many aggregates and domains. Hence, domain-aggregate growth continues because the water-poor domains are held at the water-rich sites on the surfaces of the aggregates.

At some point during the mixing operation, the quantity of water on the surface of an aggregate may be large enough so that this water will exhibit, at least temporarily, bulk water properties. Collisions and contacts among aggregates in this state will cause the aggregates to fuse due to capillary pressures caused by the menisci developed near their points of contact; these fused aggregates will be called apparent aggregates.

By this process, the apparent aggregates continue to grow as the water content increases. However, because of the shear strains developed within these relatively large apparent aggregates as they collide with each other and the components of the mixture, they will in time lose their individual identity. Thus, the effect of the shear strains and their accompanying moisture redistribution is to cause a more-dispersed structure to occur within and among the domains.

This characterization of the soil fabric is similar to that described by Yong and Warkentin (5), who recognize the arrangement of clay particles as domains, the grouping of domains as clusters, and the arrangement of clusters in peds. Many more fabric features, primarily in naturally occurring soils, have been reported and are reviewed by Mitchell (6). However, the soil model used for this study appears to be the appropriate one for this laboratory-compacted soil.

Thus, at the end of the mixing operation, the soil batch consists of aggregates and apparent aggregates over a large size range, having a considerable range of water content and a water state that varies from almost bulk to tightly held in a thickness of only a few molecules. The water distribution within each aggregate (we will no longer differentiate between aggregates and apparent aggregates) is not at equilibrium and can be expected to change with time and the pore water pressure is negative, yet the degree of saturation of the aggregate is quite high.

Compaction Mechanism

It has been postulated that the preparation of a clay soil before compaction causes the appearance of macroscopic balls or aggregates whose intra-aggregate moisture content is at a pressure less than atmospheric but having a relatively high degree of saturation that is significantly higher than that of the uncompacted mass as a whole.

Compaction is accomplished by the reduction of the void space at a constant moisture content. That is, no water leaves the system. The increase in density is due to the reduction of air voids only.

During compaction, it is of secondary importance that the aggregates are made up of clay particles, because very little total aggregate-volume change will occur (the volume of air within the aggregates is small). Densification can occur by rearrangement of the aggregates into a denser packing even if they are individually rigid. However, at most compaction water contents, the aggregates act in a plastic manner and can deform under the compactive load to conform to the available interaggregate void space.

Schematically, this densification is illustrated in Figure 2. Figures 2a and 2b show the decrease in interaggregate void ratio (densification) that can occur with only translation and rotation. The aggregates have not changed shape or volume; they have merely been rearranged. Figure 2c shows the further decreases in interaggregate void space that can occur if the aggregates deform to flatten at points of contact and to conform to the shapes of available voids or if two of the aggregates experience unit-weight increases due to aggregate-volume decreases. Thus, volume a > volume b > volume c.

Laboratory compaction can be achieved by a number of different procedures—including impact, kneading, and static type loadings and partial to full coverage of the surface of the soil. The procedure used in this investigation was a nonimpact, rapid-rise-time, controlled-loading-rate type of loading with full coverage of the soil surface. At no time during compaction is the external compressive load released. It builds up rapidly to some preselected peak, is maintained at this value for a time considerably longer than that required to reach the peak, and is finally released to conclude the densification of the soil.

Figure 1. Domains and formation of aggregates: idealized representation.

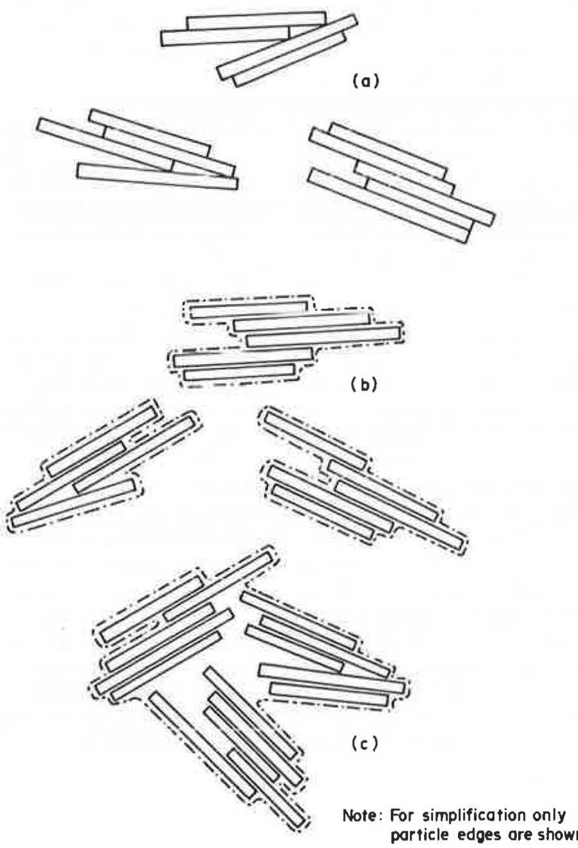
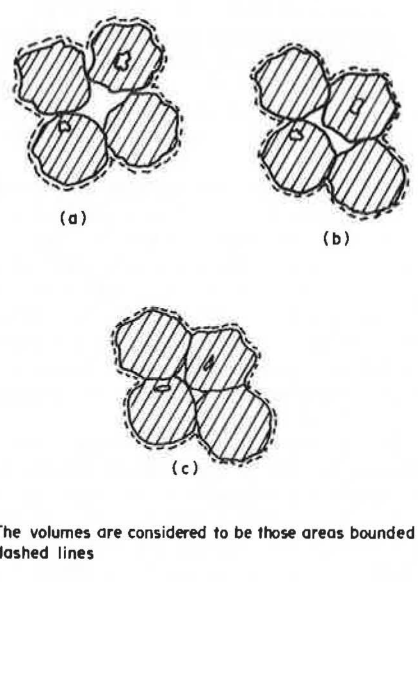


Figure 2. Aggregate compaction: idealized representation.



Note: The volumes are considered to be those areas bounded by the dashed lines

The densification proceeds at an ever decreasing rate from the outset of loading, due to (a) aggregate rearrangements not requiring deformation, (b) aggregate rearrangements initiated by slight yielding, (c) void-space filling as aggregates are deformed and literally flow into the necessary shape, and (d) the reduction of possible intra-aggregate air voids.

As the density increases, discrete aggregates become less and less apparent until finally they lose their individuality almost completely.

The microstructure or fabric of an individual aggregate also may change. If change does occur, it will be toward a more directional arrangement of particles, parallel to the plane of major stress increase, and be a result of intra-aggregate straining and surface smearing. Fabric reorientation increases as density increases and as the aggregate water content increases.

DESCRIPTION OF EXPERIMENTAL PROGRAM

Soil

The single raw material used for this study was a naturally occurring Florida kaolin mined commercially by the Edgar Plastic Kaolin Company and known as EPK Airfloated Kaolin.

The classification properties of this soil are given below:

Property	Value (%)
Liquid limit	58.5
Plastic limit	36.5
Specific gravity of solids	2.6
Clay fraction (< 0.002 mm)	81.0

Sample Compaction

The samples were compacted in a specially designed mold, and the effort was applied by an MTS servo-hydraulic tester. This combination, along with a Sanborn model 321 dual-channel carrier amplifier-recorder and a high-speed Brush model 16-2300 oscillograph, made it possible to monitor the sample densification in detail.

In the full-face-coverage compaction process (generally termed static), it is possible to control the rate of loading, monitor the axial deformation or compression, measure the input load, and measure the load transferred to the bottom of the mold by the sample, all as functions of time.

An extensive pilot study was made of the load-displacement characteristics of the machine-soil system. A comparison of the limitations of the system and the requirements of the research led to the following standard test procedure: a single application of a full-coverage compaction foot applied at a loading rate of 13.35 kN/s (3000 lbf/s) to a load of either 4.45 kN (1000 lbf) or 6.67 kN (1500 lbf) using effort as a variable. The maximum load of 4.45 or 6.67 kN was then maintained on the sample to allow further densification

Figure 3. Effect of moisture content on relationship between wet unit weight and energy: P6-R8 at 4:45-kN level.

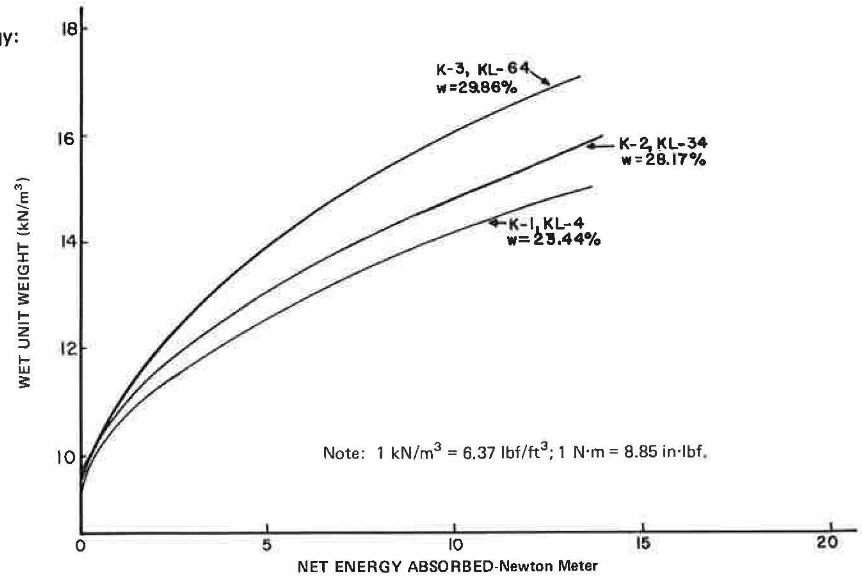
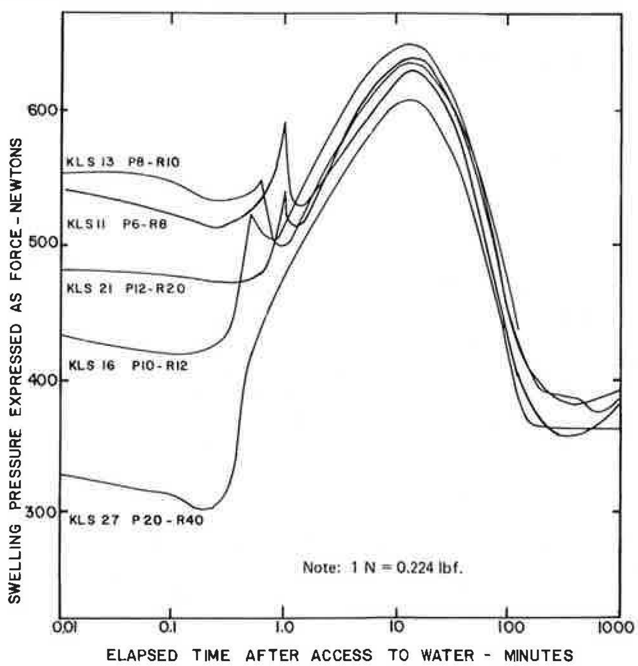


Figure 4. Results of swelling-pressure tests: K-1 at 4.45-kN level.



to occur for a period of 2 min.

The rapid rise time followed by the relatively long-term continuation at constant load made it possible to infer the aggregate-aggregate interaction. That there was no further compression (or increase in unit weight) of the sample after the initial load buildup indicates that there was an immediate rearrangement of the aggregates into a denser packing by aggregate movement, aggregate fracture, and elastic straining. Long-term compression, on the other hand, suggests plastic behavior of the aggregates.

Swelling-Pressure Measurements on Compacted Samples

As an aid to the determination of the macrostructure after compaction and its response to the forces generated by the addition of water, swelling pressure as a function of time was monitored for selected compacted samples. No gross volume change was permitted, and the temperature was held constant by making water available to the sample.

The apparatus used for these measurements consisted of a cell equipped with lower and upper bronze stones and an upper cap to eliminate vertical swelling, a Sanborn model 321 strip-chart recorder to monitor the vertical total stress by means of a Statham pressure transducer, and a constant temperature chamber (±0.2°C). Water was supplied to the cell from an inclined burette through a Tygon tube.

The cell was placed in the constant-temperature [29.5°C (84°F)] chamber and allowed to equilibrate. The pressure head of the water supply at the bottom stone was essentially constant and of small magnitude. The moisture movement through the soil was upward, which allowed the air to escape through the upper porous stone and perforated lucite spacer. The relationship between the constant-volume swelling pressure and time was recorded until an equilibrium pressure was reached (usually 24-48 h).

RESULTS

Some typical results are shown in Figures 3 and 4. The samples of soil batches are identified by the batch number and the aggregate-size sieve limits. For example, K-2, P8-R10 indicates batch number K-2 and aggregates that passed a 2.36-mm (no. 8) sieve and were retained on a 2.00-mm (no. 10) sieve. The moisture content of the batch increases with the batch number.

Achievement of Compacted Unit Weight

The relationship between load and deformation during the densification process was monitored for each compacted sample. This made it possible to define the relationship between the sample unit weight and the work applied to achieve that unit weight.

Figure 3 shows this relationship for the P6-R8 samples for the 4.45-kN load in terms of wet unit weight (w). The net energy absorbed was calculated by averaging the measured loads at the top and bottom of the sample at the beginning and end of an increment of the test data, averaging the two averages, and multiplying by the net compression of the specimen during the increment. (The averaging of the loads at the top and bottom of the sample represents an attempt to take into account the side friction between the soil and the confining ring.)

In general, the results indicate that the relationship between unit weight and net energy absorbed by the soil during densification is a simple one; that is, it does not appear to be mathematically complex.

Swelling Pressure

Figure 4 shows the relationship between the swelling pressure and time; the results are expressed as a function of aggregate size, with the batch (compaction moisture content) and level of compactive effort as constants.

In general, for the samples compacted at the 4.45-kN level of effort, the maximum swelling pressure increases as the compaction water content increases. A general time versus pressure pattern is also evident. The pressure rapidly reaches a maximum value after the sample has been given access to water and thereafter decreases at a slower rate to some equilibrium value.

The time versus pressure pattern for the samples compacted at the 6.67-kN level showed the same general pattern. However, whereas at the 4.45-kN effort level, for each aggregate size, there was an increase in the maximum swelling pressure as the compaction moisture content increased, at the 6.67-kN effort level, all aggregate sizes, without exception, exhibited increases in their maximum swelling pressure from the low (K-1) to the intermediate (K-2) moisture content and decreases from the intermediate (K-2) to the high (K-3) moisture content.

DISCUSSION OF RESULTS

Achievement of Compacted Unit Weight

In the usual terminology, the samples were compacted at water contents dry of optimum for the static compaction involved. It is commonly known that increasing the compactive-effort input results in a higher unit weight at a constant moisture content and a lower optimum moisture content. That is, as the compactive effort increases, the curve of the moisture content versus unit weight relationship shifts upward and to the left. It is also known that, if the type of compaction and the compactor rating are held constant and only the number of blows per layer or of passes in the procedure are allowed to vary, a condition—a combination of compaction and soil variables—is reached beyond which no further densification occurs and no further net energy is transferred to the mass; i.e., the soil behaves elastically. However, a heavier roller in the field or a heavier hammer in the laboratory will cause the soil to accept additional net energy until a new plateau is reached.

This can be understood in terms of the second-order skeleton. The skeletal strength increases during densification until its shear strength is equal to the compaction shear stresses. If the compaction stresses are increased by a procedural change or a change in equipment rating, further densification will occur until a new equilibrium is reached between the skeletal shear strength and the imposed shear stress system. Beyond a certain combination of stress input and compaction moisture content (or aggregate strength), the skeleton structure is effectively destroyed.

For a given soil and type of compaction, the optimum moisture content is essentially dependent on the degree of saturation. For a given moisture content, the upper limit on the compacted unit weight is imposed by the compacted soil reaching a particular degree of saturation. As shown elsewhere (2), a marked decrease in air permeability occurs at this point, which indicates a rapid decrease in the interconnected or continuous air voids. The moisture content controls the intraggregate structure. The type of compaction and the aggregate size distribution control the degree of saturation at which the air voids become discontinuous. Once the air voids become discontinuous, very little further densification will occur, regardless of the input effort. This is shown by the fact that, as the moisture content is increased wet of optimum, a common water content versus unit weight relationship develops that is independent of input energy.

The curves relating the unit weight achieved (γ_m) to the net energy absorbed (E) (such as Figure 3) are mathematically uncomplicated and can be fitted by regression analysis to quadratic equations that adequately describe the experimental results. A few examples are shown below.

For K-1, P6-R8 at 6.67 kN:

$$\gamma_m = 10.162 + 0.57122E - 0.01378E^2 \quad (1)$$

For K-3, P12-R20 at 6.67 kN:

$$\gamma_m = 10.396 + 0.80701E - 0.02191E^2 \quad (2)$$

(where γ_m is expressed in kilonewtons per cubic meter).

Grouping the linear terms of these equations at constant moisture contents shows a definite correlation; as the aggregate size decreases, the coefficient of the linear term increases. Grouping these linear terms by aggregate size also shows a definite correlation; in general, as the moisture content increases, the coefficient of the linear term also increases.

Swelling Pressure Tests

The swelling pressure tests show two characteristics that do not appear to have been previously reported in the literature. The first is that the swelling pressure for most samples tested first increased and then decreased as a function of the elapsed time after initial access to water. The second is the temporary decrease of the swelling pressure after an elapsed time of one minute or less.

As shown by Hodek (1), both the short-term (temporary) and the long-term decrease of the swelling pressure can be explained with the aid of the compacted soil model used in this study.

According to Mitchell (6), collapse due to wetting at constant total stress requires an open, partly unstable, partly saturated fabric, a metastable structure for the particular state of stress, and sufficient strength for stability before wetting.

Although collapse is usually not associated with compacted cohesive soils, when these conditions are met, collapse at a constant boundary stress level or swelling-pressure behavior such as shown in Figure 4 at constant gross volume should be expected.

Before being given access to water, the soil skeleton is at equilibrium with the boundary restraint provided by the confining ring and the fixed end cap. On the introduction of additional water to the system, at least three changes can occur. The aggregate skeleton may collapse due to local softening at the aggregate-aggregate contacts. Because the gross volume is fixed, this effect will be manifested as a rapid decrease in the measured swelling pressure. Also, as water becomes available, the aggregations may swell. If the soil skeleton can resist the increase in pressure due to this, there will be an increase in the boundary or confining pressure. Finally, as the moisture content of each aggregate increases due to swelling, its strength will decrease to allow plastic deformation of the soil skeleton into the previously empty skeletal voids. This effect occurring alone would result in a decrease in the measured swelling pressure.

Thus, the effects of water on this system are not additive. Because all three phenomena will occur simultaneously, the observed swelling pressure will be a reflection of the dominant mechanism occurring during some time increment. According to this explanation, the temporary decrease in the swelling pressure exhibited by most of the samples is caused by structural collapse, and the slow decrease from some peak value (as shown in Figure 4) is due to the dominance of plastic structural rearrangement over the swelling of individual aggregates.

SUMMARY AND CONCLUSIONS

In this study, a model or mechanism has been developed to explain the effects observed during the laboratory static compaction of kaolinite. The mechanism includes the influences of the precompaction soil preparation and conditioning as well as those of the soil interactions that occur during compaction.

Many of the swelling-pressure tests exhibited a temporary collapse or at least a decrease in the swelling pressure generated. This effect is also explained by the model. At the lower level of compaction, the maximum swelling pressure increased as the compaction moisture content increased. But at the higher compaction level, the maximum swelling pressure was largest at intermediate compaction moisture content. The final measured swelling pressures have the same type of relationship with the compactive load-moisture content combination.

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Evaluation of the Use of Indirect Tensile Test Results for Characterization of Asphalt-Emulsion-Treated Bases

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The results of a laboratory investigation of cold-mixed asphalt-emulsion-treated mixtures used for black bases are reported. The tensile properties for several mix variables and test temperatures were evaluated by the indirect tensile test. One type of asphalt emulsion and two types of aggregate were used. Two asphalt-emulsion contents and two initially added moisture contents were used. Specimens were cured at two curing conditions to represent early and long-term field curing. The relationships between load and horizontal deformation and between vertical and horizontal deformation were recorded during the test on X-Y recorders. The indirect tensile properties evaluated included indirect tensile strength, Poisson's ratio, indirect tensile stiffness, total strain at failure, and indirect tensile index. The indirect tensile index is represented by the secant modulus of the load versus horizontal deformation curve. Other parameters evaluated included compactability (unit weight after compaction), unit weight at time of testing, and percentages of retained moisture and voids at time of testing. The indirect tensile properties of the mixtures were very sensitive to the test temperature and were also affected by the other factors included in the study. In most cases, the interactions of the different factors had significant effects on the mixture properties. The indirect tensile index provided a high correlation with the asphalt-emulsion content, the type of aggregate, and the test temperature and proved to be a better mixture-characterization factor than the indirect tensile stiffness.

Emulsified-asphalt-treated mixtures are being widely accepted by engineers because of their many ecological-performance and economic advantages. Unlike asphalt cement, emulsified asphalt requires little or no heating