

# FUNDAMENTALS OF SOIL COMPACTION AND PERFORMANCE

Kamran Majidzadeh and Hani R. Guirguis, Ohio State University

The overall objectives of this research study were to develop methods of predicting pavement performance under severe service conditions; to arrive at a more rapid, accurate means of controlling the quality of the compaction process; to predict, from a limited laboratory study, the quality of sub-grade soil required for the anticipated service conditions; and to ensure that the desired material properties are attained during construction. The research was carried out primarily to study the correlation between engineering and physical properties of field- and laboratory-compacted specimens. Undisturbed field specimens were obtained after regular sheepsfoot-roller compaction, regular proof rolling, and extra proof rolling. The parameters evaluated were physical characteristic, primary response, and ultimate response. The combined effects of compactive effort, type of compaction, moisture content at compaction, and moisture gain after compaction on the physical parameters and engineering properties were noted and compared for field- and laboratory-prepared soil. This study shows that the physical soil parameters describing the interrelation among moisture, density, compaction energy, and method of compaction can be used for the correlation of laboratory and field specimens. Comparing shear strength characteristics of laboratory and field specimens at various moisture contents and compaction conditions indicated that the laboratory gyratory method and the 40-blow drop-hammer method are more representative of the field-compaction process. The creep modulus, dynamic modulus, and resilient modulus can be used to interrelate properties of field- and laboratory-compacted specimens. The relations presented in this report indicate that maximum moduli occur at approximately the same moisture content as the optimum dry density and shear strength.

•THE ABILITY of any rational design method to predict the stress-strain-environment-time response of the pavement system depends on the knowledge of material properties that best measure performance. The subgrade soil is considered to be an important component of the pavement system, and its contribution to the performance level depends on its in situ characteristics. In many engineering analyses, the physical characteristics of soils such as moisture and density are used and in turn are translated into the efficiency of the compaction process. The mechanics of the compaction process, field and laboratory methods of soil densification, and methods of evaluation of properties of compacted soil have been discussed in great length in the published literature (1, 2, 3, 4).

It has been recognized (5, 6, 7, 8) that the soil compaction process affects a variety of properties that can be broadly categorized into 2 interrelated types: physical and engineering properties. The physical properties, i.e., the moisture and density, can be used to analyze the efficiency of the compaction process. However, engineers are interested not only in density per se but also in its effect on the engineering properties describing the soil-support conditions.

It has been demonstrated that soils compacted to a given density and moisture content may exhibit different engineering properties depending on the method of compaction used and the soil structure developed by the application of compaction energy. Be-

cause physical properties cannot fully satisfy all requirements of pavement design and performance evaluation, there is a question of what limiting engineering properties need to be selected.

In the search for suitable engineering properties to be used in the evaluation of laboratory- and field-compacted specimens, many researchers have investigated rheological and strength-deformation characteristics of subgrade soils as affected by the compaction process and environmental conditions (9, 10, 11, 12, 13, 14). The nature of the material response to changes in climatic and loading conditions has been extensively used as a basis for the characterization of soils within the framework of elastic, viscous, viscoelastic, and elastic-plastic theories. The analysis of soil behavior presented in this study can be placed into 2 categories: (a) properties describing the non-failure-state response (primary response) and (b) properties describing the failure-state response (ultimate response). The non-failure-state response can be characterized by creep, relaxation, dynamic modulus, and resilient modulus. The failure-state response includes both unconfined compressive or shear strength and failure under repeated loading.

The rate process theory and accumulated plastic deformation concepts are used to explain the process of deformation and failure under repeated loading. In this study, special attention is given to the following parameters, which are suggested to be a measure of subgrade performance: (a) resilience modulus and response under repeated loading; (b) shear strength; and (c) rutting deformation, resultant densification, and cumulative damage under repeated loading. Other parameters, such as soil suction-moisture relation and frost susceptibility, are also considered as important performance parameters, but they are not discussed in this paper.

#### TESTING PROCEDURES AND MATERIAL CHARACTERISTICS

The material used in this investigation is a silty clay soil obtained from Allen County, Ohio (US-30, Project 433-1969). Two types of specimens were used: undisturbed field cores and laboratory-compacted specimens. The undisturbed samples were taken from the subgrade embankment by means of a 3-in. internal diameter Shelby tube; the dry boring method was used. The specimens were taken after varying amounts of compaction effort had been applied to the soil. The compaction levels at which undisturbed soil samples were obtained are as follows: (a) regular sheepfoot rolling, (b) proof rolling at regular speed, (c) 3 passes of proof rolling at regular speed, and (d) 3 passes of proof rolling at regular speed and 1 pass at double speed.

In this study, laboratory specimens were prepared by drop-hammer and gyratory compaction. For the drop-hammer compaction, the compaction energy varied by the total number of blows imparted to the specimen. The number of blows ranged from 25 to 70; 40 blows correspond to the modified AASHTO compaction energy input. For gyratory compaction, the angle of gyration and the number of gyrations were kept at 2 deg and 15 gyrations. The axial static pressure was 700 and 100 lb. The physical characteristics of the soil used in this study are as follows:

<u>Characteristic</u>	<u>Value</u>
Liquid limit	37
Plastic limit	19
Specific gravity	2.78
Unified classification	CL
AASHTO classification	A-6

The moisture-density relation of drop-hammer- and gyratory-compacted specimens is shown in Figures 1 and 2. In Figure 1, the moisture-density relation of undisturbed soil samples is superimposed on the moisture-density data for drop-hammer compaction. It appears that field data are primarily on the wet side of optimum.

Figure 1. Field moisture content versus density with drop-hammer compaction.

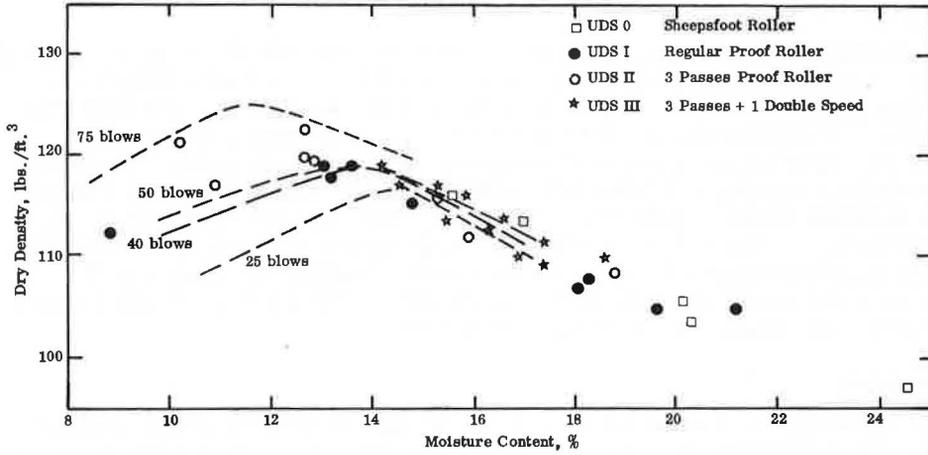
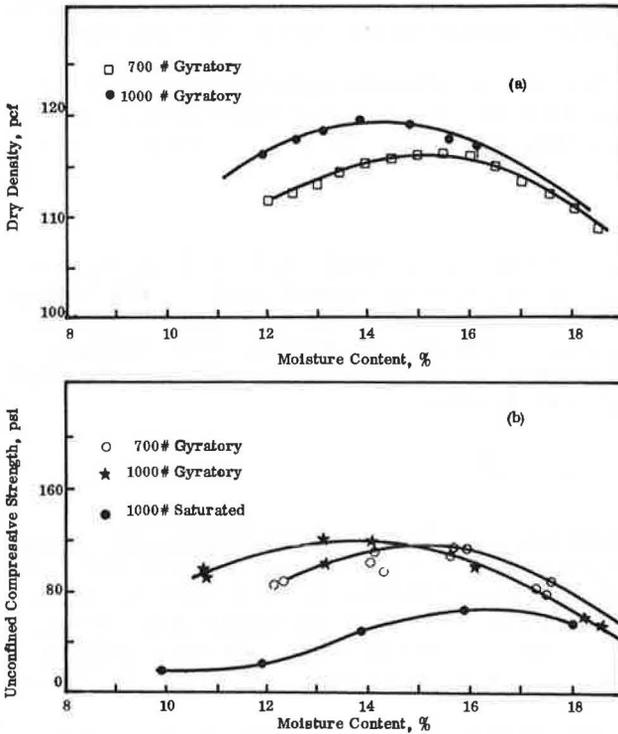


Figure 2. Dry density and unconfined compressive strength versus moisture content with gyratory compaction.



## PRIMARY RESPONSE

Creep Function

The deformation-time response of a material body under constant stress, i.e., creep, is extensively used for characterizing engineering systems. The creep compliance  $J(t)$ , defined as strain per unit stress, or its inverse creep modulus  $E_c$ , has been used in the characterization of subgrade soils as well as other pavement materials.

So that deformation characteristics of compacted soils can be compared at various moisture contents, compactive efforts, and methods of compaction, the creep modulus  $E_c$  has been determined at  $t = 1,500$  sec. The  $E_c$ -moisture content relation exhibits a similar trend to moisture-density and shear strength-moisture content relations (Fig. 3). The optimum moisture content for  $E_c$ -moisture content relation for drop-hammer- and gyratory-compacted specimens agrees closely with the corresponding optimum moisture content for moisture-density-shear strength relations.

Dynamic Modulus

The dynamic response of material systems is characterized by dynamic modulus  $/E^*$ , which is considered to be one of the major primary response mode parameters. The dynamic modulus can be determined either directly from the dynamic experiments or from the transformation of creep compliance data into the frequency domain.

Figure 4 shows the dynamic moduli of compacted laboratory and field specimens at various moisture contents with various compaction levels. The curves are similar to those shown for the creep modulus-moisture content relations. Similarly, the optimum moisture content agrees well with the optimum moisture content for density, shear strength, and creep modulus.

The comparison of data indicates that, at high moisture contents (greater than 16 percent), the agreement is rather poor but that, within the range of optimum moisture content, the field data agree well with either 700 lb of gyratory or 40 blows of drop-hammer compaction.

Resilient Modulus

The sinusoidal loading functions used for the determination of dynamic modulus were an oversimplification of actual loading patterns occurring in pavements. Traffic loading patterns were simulated in the laboratory by experiments that used pulsating dynamic loads of varying frequency and rest periods. The modulus characterizing material response under such dynamic loading is known as the modulus of resilience,  $M_r$ , which was calculated for each load cycle from the relation

$$M_r = \frac{\sigma}{\epsilon_d}$$

where  $\sigma$  is the repeatedly applied stress and  $\epsilon_d$  is the resilient or recoverable strain. In general, 2 types of relations were observed: At stresses greater than the endurance limit,  $M_r$  decreased with increasing  $N$ ; at stresses smaller than the endurance limit,  $M_r$  increased with increasing  $N$ . The first type of behavior was possibly because of work softening, while the latter was a result of work hardening. Above the endurance limit, the structural changes induced are nonreversible and are caused by cumulative damage within the material system.

In general, the relation of  $M_r$  with the number of load repetitions  $N$  can be expressed as an approximation by

$$M_r = A N^{\alpha(\sigma)}$$

where  $\alpha$  depends on the stress level or more precisely on the ratio of applied stress to strength. For stresses below the endurance limit,  $\alpha$  is more than unity; for stresses large enough to result in damage, i.e., greater than endurance limit,  $\alpha$  is less than unity.

Figure 3. Dynamic moduli versus moisture content with various compaction methods.

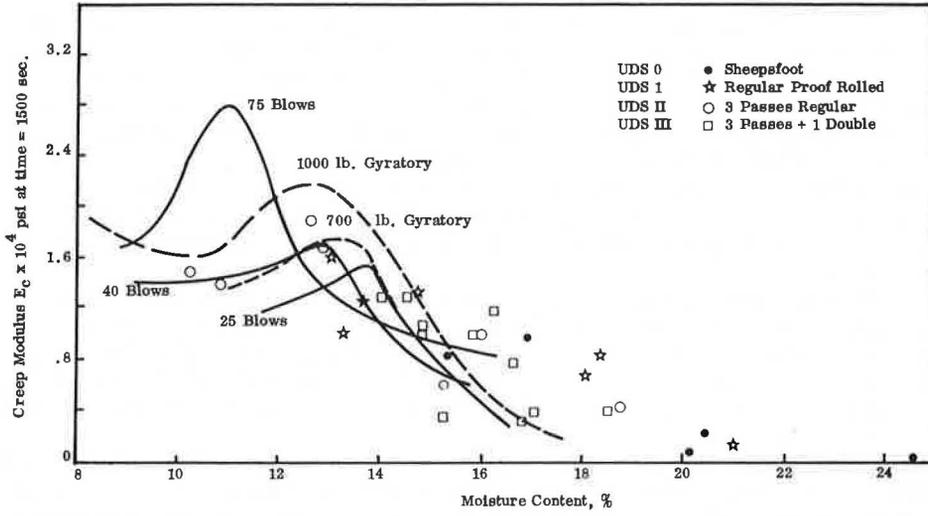
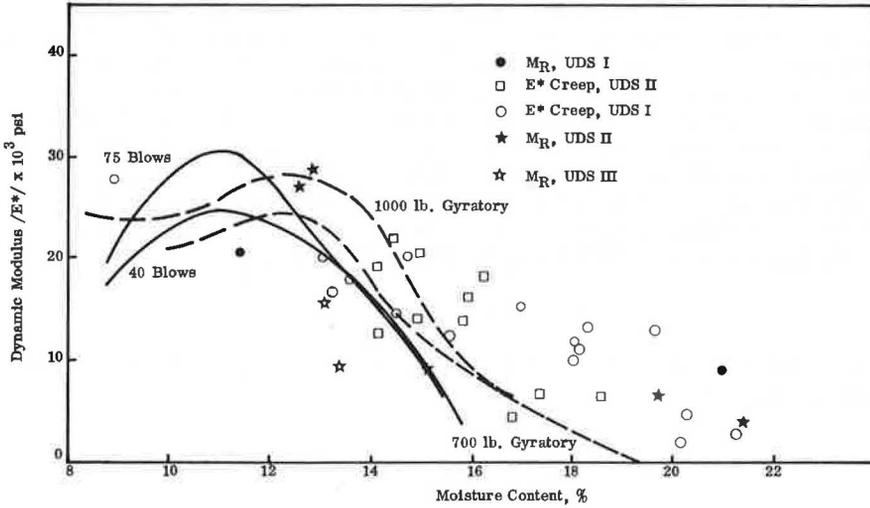


Figure 4. Creep moduli versus moisture content with various compaction methods.



The variations of  $M_r$  with repeated load magnitude and confining stress are shown in Figure 5. The confining stresses were 0, 9, and 21 psi. The effect of confining pressure seems to be to shift the curve to the right, but the shape remains the same. The shape of the curves strongly suggests the existence of different phenomena at low and high stress levels. Figure 6 shows the variations of  $M_r$  with dynamic stress for unconfined experiments and for  $N = 30$  and 3,000 cycles.  $M_r$  increases with an increase in dynamic stress up to a point and then decreases. Because there is little distinction between  $N = 30$  and  $N = 3,000$  at low stress levels, densification or work-hardening phenomena are apparently occurring. At higher dynamic stresses, however, the substantial difference between  $N = 30$  and  $N = 3,000$  cycles suggests a damage or work-softening phenomenon. Obviously the greater the number of load cycles is, the lower the modulus of resilience will be.

Of the above findings, the variation of  $M_r$  with the magnitude of the applied stress is of particular significance. Seed et al. (35) investigated deviator stresses ranging from 3.1 to 50 psi (representing about 5 and 80 percent of the ultimate strength of the samples) and reported values of  $M_r$  for  $N = 200$  and 100,000. Their results showed  $M_r$  to decrease rapidly and then increase at a very slow rate as stress was increased. No explanations were offered for that behavior.

In this study, deviator stresses varying from 22 to 126 psi (representing 12 to 68 percent of the ultimate strength) were used, and values of  $M_r$  are reported at  $N = 1,000$  and 3,000. There was agreement between this study and that of Seed et al. (35) in that they both indicated a range of stresses over which  $M_r$  increased and then decreased gradually. For unconfined experiments, however, this study showed that the values of  $M_r$  increased with stress under the endurance limit.

Figure 7 shows the relation of resilient modulus with the ratio of applied stress to ultimate strength for confining pressures of 0, 9, and 21 psi in triaxial tests. The general shape of the curve is very similar to that shown in Figure 6. However, the data are superimposed in a single master curve irrespective of confining pressure. Furthermore, the first branch of the  $M_r$ - $\sigma$  relation corresponding to the densification or work-hardening phenomena is apparently only occurring under unconfined experiments. Because subgrade soil elements under pavement structure are under confining pressure, it appears that the second branch of  $M_r$ - $\sigma$  (i.e., the work-softening branch) is of more practical value. That observation is in accordance with the findings of Seed et al. (35). The  $M_r$ - $\sigma$  relation can in general be expressed approximately by

$$M_r = B \theta^{-n}$$

where  $B$  and  $n$  are constants, and  $\theta$  is a stress invariant ( $\theta = \sigma_1 + \sigma_2 + \sigma_3$ ). This relation can also be expressed approximately by

$$M_r = B_1 \left( \frac{\sigma_{\text{dynamic}}}{\sigma_{\text{ultimate}}} \right)^{-n_1}$$

where  $B_1$  and  $n_1$  are material constants, and  $\sigma_{\text{ultimate}}$  is compressive strength determined in triaxial tests.

## ULTIMATE RESPONSE

### Compressive Strength

The factors influencing the shear strength of a subgrade soil are moisture content at compaction, moisture changes after compaction, dry density, and soil structure. Moisture has a very significant effect on the performance of clay subgrade soil. Taken in combination with compactive effort and type of compaction, it determines the resulting dry density and more important the soil structure (35, 36, 37). Soil structure is an important presage of soil behavior. For the same values of moisture content and dry density, the stress-strain curve for a soil with a flocculated structure is much steeper and has a different shape than that for the same soil with a dispersed structure.

Figure 5. Variation of  $M_R$  with stress invariant.

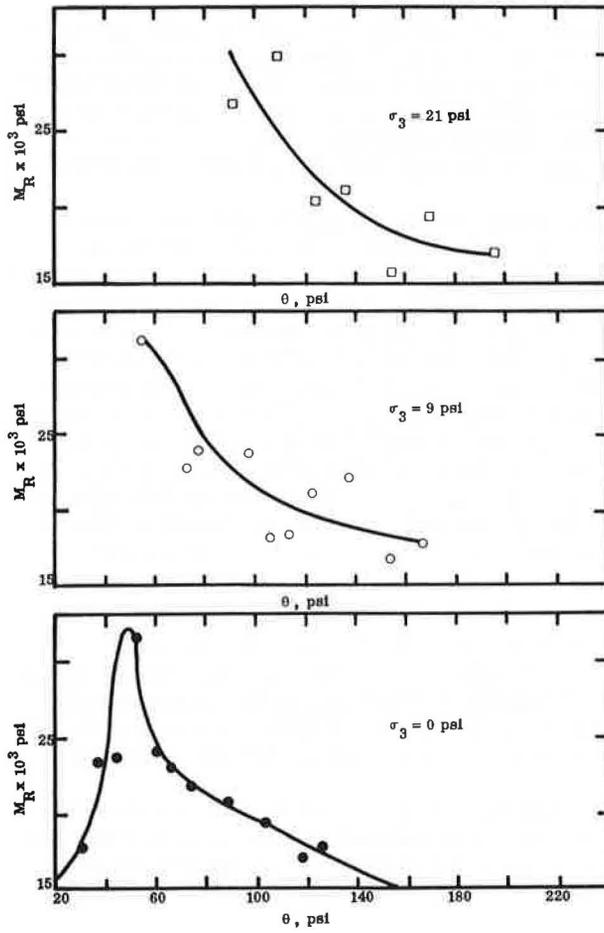
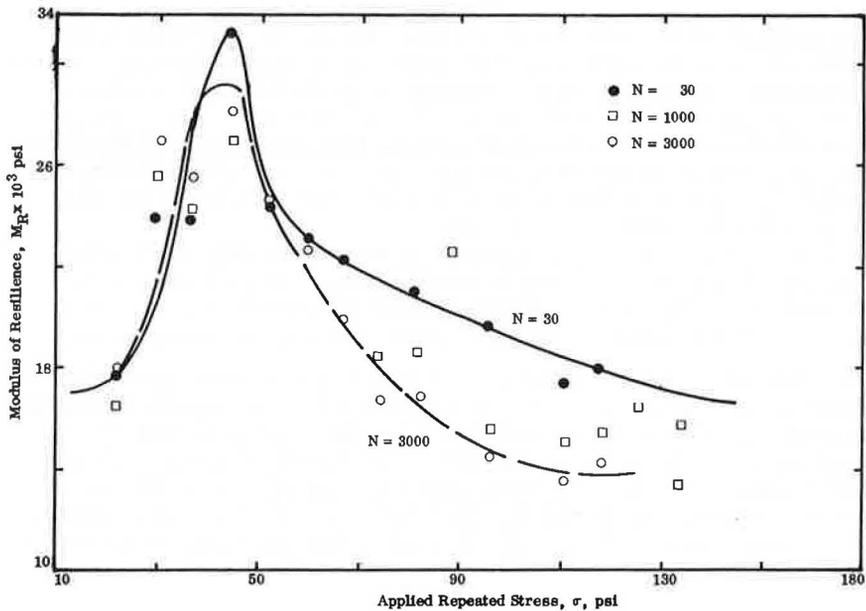


Figure 6. Variation of  $M_R$  with stress level after 30 and 3,000 cycles of load repetition.



Comparisons of the unconfined compressive strength of 700- and 1,000-lb gyratory-compacted specimens at various moisture contents (Fig. 2) show that the difference in the shear strength for specimens compacted on the wet side of optimum is rather small. That is obviously attributed to the dispersed structure formed at high moisture contents. The difference in the shear strength, however, becomes larger as specimens having less moisture content (i.e., compacted on the dry side of optimum) are compared.

Similarly, the difference in the unconfined compressive strength of drop-hammer-compacted soils on the wet side of optimum is rather small. The fact that all specimens attain approximately similar strength characteristics at moisture contents greater than 15 percent, despite their differences in dry density, again points to the overriding influence of structure on strength. However, for specimens compacted on the dry side of optimum by drop-hammer compaction, the influence of compactive energy on the strength is more pronounced. That is, the increase in the compactive energy results in a greater increase in the unconfined compressive strength. If the strength-moisture content relations are considered, the drop-hammer-compacted specimens are more sensitive to moisture variation than gyratory-compacted specimens. This again points to the differences in the structure of compacted clay as affected by the method of compaction. Finally, it is clearly noted that the unconfined compressive strength of field specimens agrees very closely with the results of gyratory-compacted specimens.

#### Cumulative Deformation: Rutting

Studies of elastomers, metals, and asphalts have shown that the cumulative-deformation and failure response of materials under repeated loading are very similar to that of the creep-rupture phenomena. It has been asserted that the principles of static and dynamic rupture are identical and that the observed difference in the life of specimens is due to the rate of cumulative damage and relaxation between load applications.

Figure 8 shows a typical deformation-time relation for a soil specimen under repeated loading in which the variation of cumulative permanent deformation  $\%$ , with number of load applications is presented. It is noted that, similar to creep-rupture phenomena, the cumulative permanent deformation-time relation might be divided into 3 distinct stages:

1. Initially the deformation increases rapidly but with a decreasing rate with the number of cycles of loading. Densification may occur here and involves the decrease in air-void content of the soil by the progressive rearrangement of the particles relative to each other.
2. The second stage involves the time-dependent rearrangement of the particles. It is an irrecoverable flow process and is effected by the successive yielding and deformation of particle contacts. The resultant deformation response during this state is in accordance with the postulates of the rate process theory; the rate of permanent deformation is expressed in terms of the number of load applications instead of the time of load applications. The rate of deformation and volume change are considerably reduced, consistent with the idea of a flow of particles into the vacated voids and rearrangement as opposed to a translation of particles.
3. The third stage is characterized by an increase in the rate of deformation leading ultimately to failure. Failure is due to an accumulation of damage resulting from the formation and growth of plastic zones at points of local overstressing and the enlargement of voids and weak boundaries in the soil structure under repeated loading.

In this study, the rate process theory concepts are used to explain the accumulation of permanent deformation of stages 1 and 2 for clay specimens under repeated loadings.

The theory of absolute reaction rates or rate process theory, proposed initially by Eyring et al. (15) and applicable to processes involving the motion of particles, has been used extensively to describe and predict the creep and consolidation behavior of clays as well as other materials (16, 17, 18, 19, 20, 21, 22, 23, 24, 25). The contributions of Mitchell, Campanella, and Singh (21, 27) in adapting the theory of absolute reac-

Figure 7. Variation of  $M_R$  with stress ratio for different confining pressures.

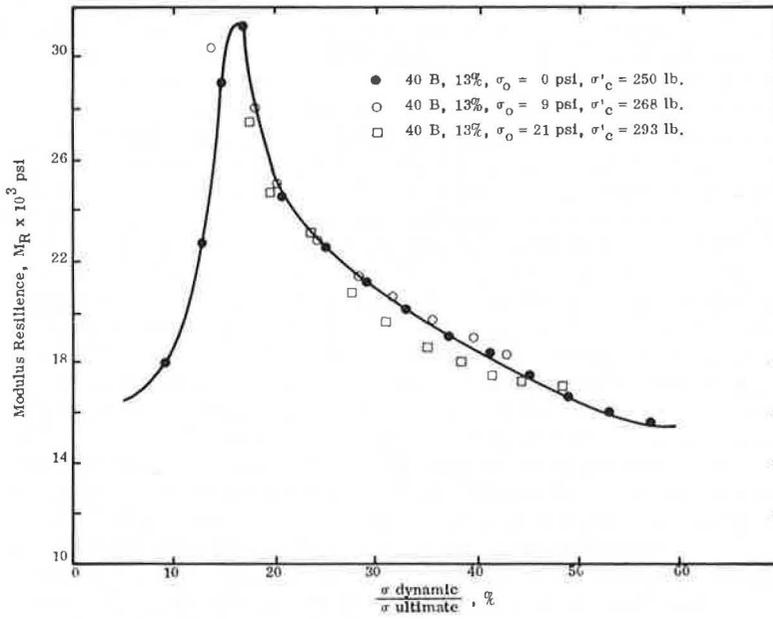
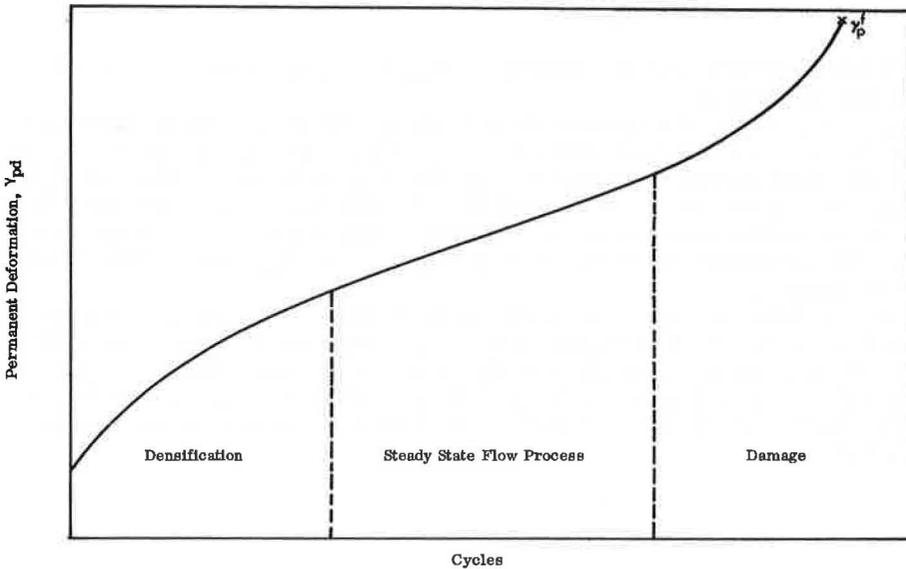


Figure 8. Typical deformation-load cycle curve.



tion rates to the study of the time-dependent deformation of soils have been especially significant. The most significant aspect of their work is the recognition of the fact that "structure" can affect the time response of soils, and they have accordingly adapted the general rate process equation by the inclusion of a parameter to account for this influence. The work by Mitchell et al. forms the basis for this investigation into the mechanism of the deformation of clay soil subjected to repeated loading.

Mitchell et al. (21), working with clay soil under static creep loading, observed that plots of axial strain versus time on a log-log scale gave parallel lines for different stress levels, which indicated an independence between creep stress and slope. The slope is negative, however, meaning that for any given stress the creep rate decreases in a regular manner with time. Similar observations were made in this study. Figure 9 shows a plot of  $\ln(d\gamma_{pd}/dN)$  versus  $\ln N$ ; the interrelation among  $d\gamma_{pd}/dN$ ,  $N$ , and repeated stress is similar to that among  $d\gamma/dt$ ,  $t$ , and creep stress under static loading conditions. That similarity in observed response leads to the obvious conclusion that the mechanisms of compacted clay deformation under static and dynamic conditions are very similar, if not the same. The possibility exists, therefore, of being able to predict dynamic response from static tests and vice versa. The term  $\gamma_{pd}$  used in this analysis denotes the deviatoric component of permanent deformation. In this investigation, in accordance with soil mechanics conventions, stress and strains are resolved into components responsible for change of volume and those responsible for change of shape. Because the flow process is essentially a constant volume process, the rate process is then applied to only the deviatoric component of permanent deformation  $\gamma_d$ .

The nature of the observed interrelations between  $\ln(d\gamma_{pd}/dN)$  and  $\ln N$  at constant repeated stress  $\sigma$  (Fig. 9) and between  $\ln(d\gamma_{pd}/dN)$  and  $\sigma$  at constant  $N$  (Fig. 10) suggests that the deformation rate of compacted clay under stresses that are not large enough to cause accelerated damage may well be expressed by the following equation:

$$\frac{d\gamma_{pd}}{dN} = A e^{B\sigma} N^D$$

where  $A$  and  $B$  are constants, and  $D$  is stress dependent.  $A$ , however, may be both structure and time dependent.

As mentioned before, the rate process theory concepts apply to primary and secondary stages of the deformation-time relation. For the third stage, McClintock (32, 33) has indicated that, once damage is initiated, a plastic zone extends a distance  $R$  in front of the overstressed region. As a first approximation, it might be assumed that failure depends only on the strain and that failure will occur whenever the strain reaches a critical value. This critical plastic strain at failure is called  $\gamma_p^f$ , which represents the critical level of damage.

Besides the cumulative damage rule and the critical magnitude of damage occurring at the event of failure  $\gamma_p^f$ , the rate of damage accumulation under repeated load application is of theoretical interest. In analogy to the processes of deformation, damage growth associated with cracking, and reaction rate principles describing bond breakage and deformation, Guirguis (34) has proposed an empirical power law for the damage growth equation given as

$$\frac{d\gamma_p}{dN} = c (\gamma_p)^m$$

where

- $\gamma_p$  = plastic deformation at any time of loading,
- $N$  = number of cycles,
- $c$  = soil constant, and
- $m$  = constant independent of the stress level.

He found  $m$  ranged from 8 to 11 for various soils compacted at optimum moisture content and used a range of stress levels and compaction energies (Figs. 11 and 12).

Figure 9.  $d\gamma_{pd}/dN$  versus  $N$  (log-log scale).

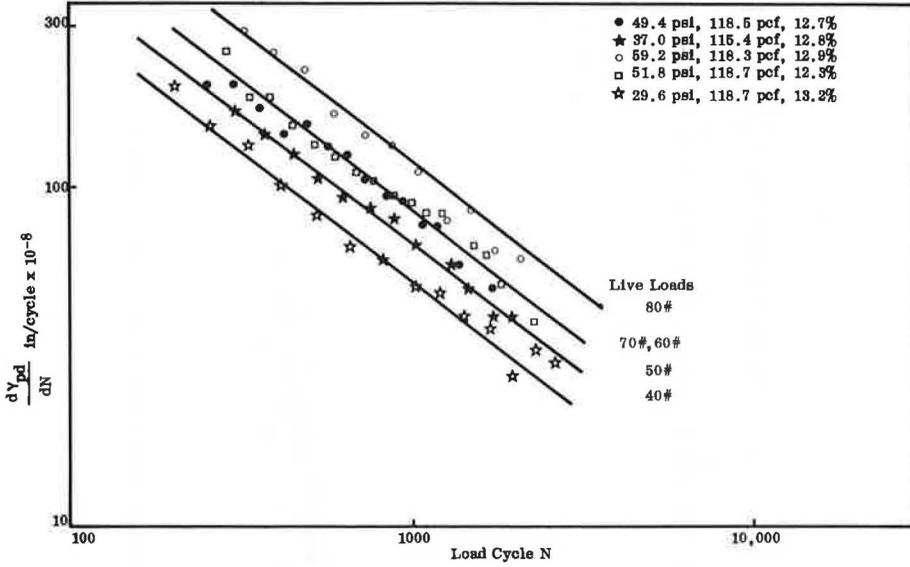


Figure 10.  $d\gamma_{pd}/dN$  versus  $p_d$  for 40-blow compaction.

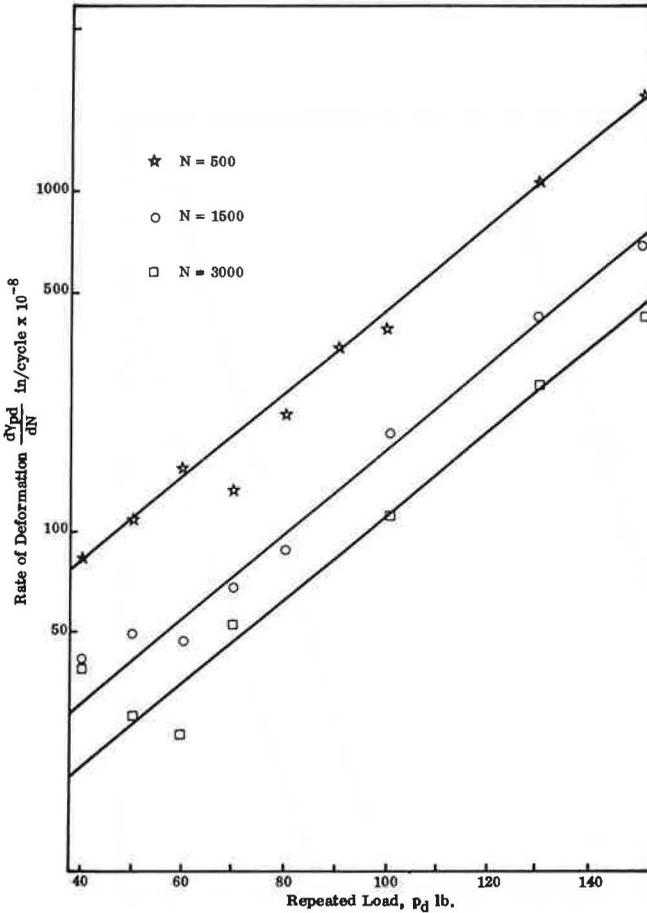


Figure 11.  $d\gamma_{pd}/dN$  versus  $\gamma_{pd}$  for 40-blow compaction (log-log scale).

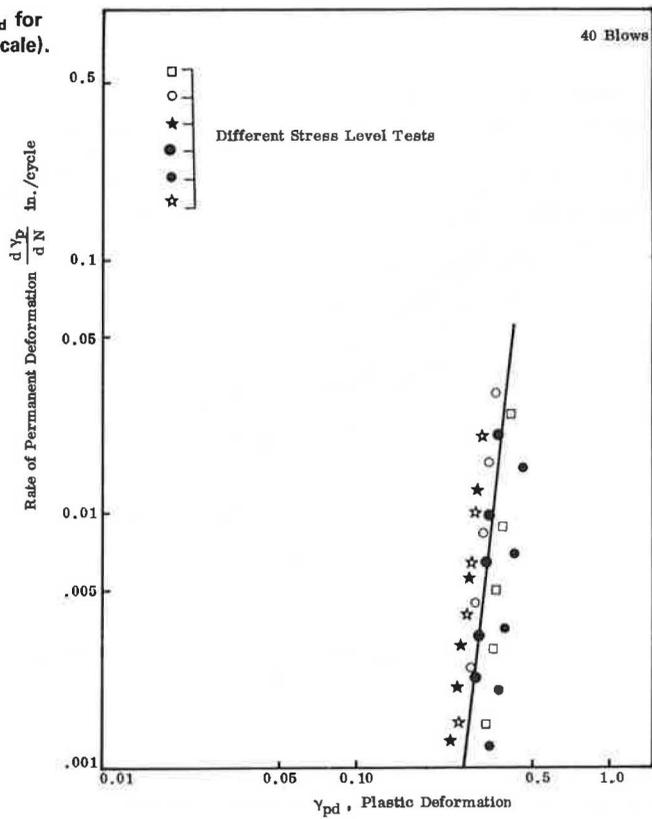
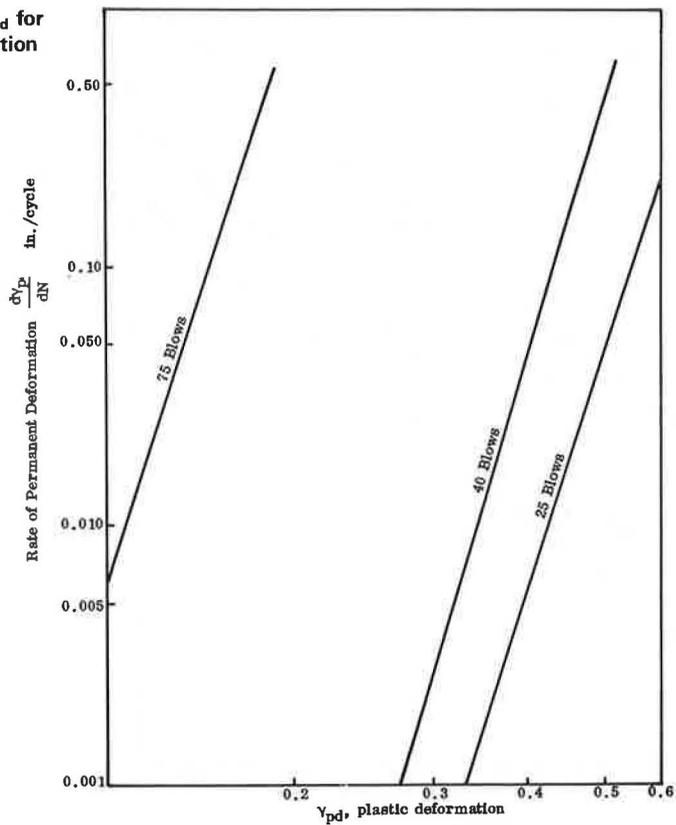


Figure 12.  $d\gamma_{pd}/dN$  versus  $\gamma_{pd}$  for the average of various compaction methods.



It can be easily shown that such a power law relation between the rate of permanent deformation and the magnitude of the permanent deformation itself exists during the entire repeated-loading deformation process up to failure. During densification and steady-state flow, however, the power law equation has a negative index; during the damage branch, the index changes signs. The point of transition of those lines gives the magnitude of permanent deformation at which the damage rate becomes critical. This suggests an additional criterion for pavement subgrade design: For the subgrade material and stress conditions anticipated in service, the permanent deformation must not exceed a certain critical value  $\gamma_p^c$  (can be assumed to be  $0.70 \gamma_p^f$ ). The design problem would then be to determine the value of  $\gamma_p^f$  for a given subgrade material and to predict the number of load applications  $N$  required to bring about  $\gamma_p^c$ . The signs are optimistic that this can be done.

### SUMMARY AND CONCLUSIONS

This research has been carried out primarily to study the correlation between engineering and physical properties of field- and laboratory-compacted subgrade specimens. In addition, other fundamental material characteristics that best describe the subgrade performance have also been presented. The conclusions derived from this study are given below:

1. The 1,000-lb gyratory, the 40-blow drop-hammer, the undisturbed field samples, and the Ohio Department of Highways (curve K) compaction curves agree with each other.
2. Shear strength characteristics of undisturbed field samples and 700-lb gyratory laboratory samples agree very well.
3. The compaction of creep modulus  $E_c$  and moisture content relation of 700-lb gyratory, 40-blow laboratory samples, and undisturbed field samples agree closely. The maximum creep modulus occurs at the optimum conditions of density and shear strength.
4. The dynamic modulus  $/E^*/$  determined from the transformation of creep data has the same trend as  $E_c$ .
5. The resilient modulus  $M_R$  decreases as the number of load applications  $N$  increases for stresses above the endurance limit, that is, stresses large enough to cause damage. For stresses less than the endurance limit,  $M_R$  increases as  $N$  increases for the range of values of  $N$  investigated (3,600 maximum). The results also indicate that the resilient modulus  $M_R$  varies with the magnitude of the repeated load and the ratio of applied load to strength and is expressed as follows:

$$M_R = B_1 \left( \frac{\sigma_{dynamic}}{\sigma_{ultimate}} \right)^{-n_1}$$

It is also noted that  $M_R$  and  $/E^*/$  show close agreement.

6. It has been shown that, for deviatoric permanent deformation, the branches of densification and steady-state flow of the  $\gamma_{pd}$ - $N$  relation are in accordance with the postulates of the rate process theory and can be expressed as

$$\frac{d\gamma_{pd}}{dN} = A e^{B\sigma} N^D$$

where  $D$  is a stress-dependent constant. It has also been shown that the damage branch of the same relation can be expressed as a power law

$$\frac{d\gamma_p}{dN} = c (\gamma_p)^m$$

where  $m = 8-11$ . Furthermore, the data indicated that failure occurs when accumulated permanent deformation reaches a constant value  $\gamma_p^f$ , which has been found to be independent of the path taken to failure.

## REFERENCES

1. An Outline of Suggestions for Study of Compaction of Earthwork. Highway Research Correlation Service, May 1964.
2. Symposium on Compaction of Earthwork and Granular Bases. Highway Research Record 177, 1967.
3. Soil Compaction and Proof-Rolling of Subgrades. HRB Bull. 254, 1960.
4. Compaction of Soils. ASTM, Spec. Tech. Publ. 377, June 1964.
5. Seed, H. B., and Chan, C. K. Undrained Strength of Compacted Clays After Soaking. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Oct. 1959, pp. 31-47.
6. Seed, H. B., Mitchell, J. K., and Chan, C. K. The Strength of Compacted Cohesive Soils. *Proc., ASCE Res. Conf. on Shear Strength of Cohesive Soils*, 1960, pp. 877-964.
7. Seed, H. B., and Chan, C. K. Structure and Strength Characteristics of Compacted Clays. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Oct. 1959.
8. Mitchell, J. K., Hooper, D. R., and Campanella, R. G. The Permeability of Compacted Clays. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, July 1965.
9. Pagen, C. A., and Jagannath, B. N. Fundamentals of Soil Compaction. Eng. Exp. Station, Ohio State Univ., Rept. EES 248-1, July 1966.
10. Pagen, C. A., and Jagannath, B. N. Effect of Gyrotory Compaction on the Rheological Characteristics of Clay. Eng. Exp. Station, Ohio State Univ., Rept. EES 248-3, 1966.
11. Pagen, C. A., Wang, C., and Jagannath, B. N. Effect of Compaction and Increase of Saturation After Compaction on the Engineering Properties of Compacted Clay. Eng. Exp. Station, Ohio State Univ., Rept. EES 248-6, July 1968.
12. Pagen, C. A., and Khosla, V. K. Frost Action Characteristics of Compacted Cohesive Soil. Eng. Exp. Station, Ohio State Univ., July 1968.
13. Pagen, C. A., and Jagannath, B. N. Resilient and Residual Mechanical Properties of Compacted Kaoline Clay. Eng. Exp. Station, Ohio State Univ., July 1968.
14. Pagen, C. A., and Jagannath, B. N. Rheological Properties of Hybla Valley Clay. Eng. Exp. Station, Ohio State Univ., July 1969.
15. Glasstone, S., Laidler, K., and Eyring, H. *The Theory of Rate Process*. McGraw-Hill, New York, 1941.
16. Abdel-Hady, M., and Herrin, M. Characteristics of Soil-Asphalt as a Rate Process. *Jour. Highw. Div., Proc., ASCE*, Vol. 92, No. HW1, March 1966.
17. Andersland, O. B., and Akili, W. Stress Effect on Creep Rates of a Frozen Clay Soil. *Geotechnique*, Vol. 17, March 1967.
18. Christensen, R. W., and Wu, T. H. Analysis of Clay Deformation as a Rate Process. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Vol. 90, No. SM6, Nov. 1964.
19. Herrin, M., and Jones, G. The Behavior of Bituminous Materials From the Viewpoint of the Absolute Rate Theory. *Proc., AAPT*, Vol. 32, Feb. 1963.
20. Herrin, M., Marek, C. R., and Strauss, R. The Applicability of the Absolute Rate Theory in Explaining the Behavior of Bituminous Materials. *Proc., AAPT*, Vol. 35, Feb. 1966.
21. Mitchell, J. K., Campanella, R., and Singh, A. Soil Creep as a Rate Process. *Jour. Soil Mech. Div., Proc., ASCE*, Vol. 94, No. SM1, Jan. 1968.
22. Mitchell, J. K. Shearing Resistance of Soils as a Rate Process. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Vol. 90, No. SM1, Jan. 1964.
23. Murayama, S., and Shibata, T. Rheological Properties of Clays. *Proc., 5th Int. Conf. Soil Mech. and Found. Eng., Paris*, 1961.
24. Singh, A., and Mitchell, J. K. General Stress-Strain-Time Function for Soils. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Vol. 94, SM1, Jan. 1968.
25. Wu, T. H., Resendiz, D., and Neukirchner, R. J. Analysis of Consolidation by Rate Process Theory. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Vol. 92, No. SM6, Nov. 1966.
26. Murayama, S., and Shibata, T. On the Rheological Characteristics of Clay—Part I. *Disaster Prevention Res. Inst., Kyoto Univ., Japan, Bull.* 26, 1958.
27. Mitchell, J. K., Singh, A., and Campanella, R. Bonding, Effective Stresses and Strength of Soils. *Jour. Soil Mech. and Found. Div., Proc., ASCE*, Vol. 95, No. SM5, Sept. 1969.

28. Christensen, R. W., and Kim, J. S. Rheological Model Studies in Clay. Dept. Eng. Mech., Univ. of Wisconsin, Madison, 1969.
29. Polakowski, N. H., and Ripling, E. J. Strength and Structure of Engineering Materials. Prentice-Hall, New Jersey, 1966.
30. Fletcher, T. A. Fatigue Analysis of a Sand-Asphalt Mixture. Ohio State Univ., MS thesis, 1968.
31. Bartenev, G. M., and Zuyev, Yu. S. Strength and Failure of Viscoelastic Materials. Pergamon Press, New York, 1968.
32. McClintock, F. A., and Argon, A. S. Mechanical Behavior of Materials. Addison-Wesley, 1966.
33. McClintock, F. A. On the Plasticity of the Growth of Fatigue Cracks. In Fracture of Solids, John Wiley and Sons, New York, 1962.
34. Guirguis, H. R. Subgrade Compaction and Its Performance Under Traffic Loads. Ohio State Univ., MS thesis, 1970.
35. Seed, H. B., Chan, C. K., and Lee, C. E. Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements. Proc., Int. Conf. Struct. Des. of Asphalt Pavements, Univ. of Michigan, Aug. 1962.
36. Lambe, T. W. The Structure of Compacted Clay. Jour. Soil Mech. and Found. Div., Proc., ASCE, Vol. 84, No. SM2, 1958.
37. Langfelder, L. J., and Nivargikar, V. R. Some Factors Influencing Shear Strength and Compressibility of Compacted Soils. Highway Research Record 177, 1967, pp. 4-21.