Resilient Modulus of Laboratory and Field Compacted Cohesive Soils

실내와 현장다짐 점성토의 회복탄성계수

Lee, Woo-Jin *
이 우 진

요 지

실내다짐토와 현장다짐토의 회복탄성특성을 비교하기 위하여 5종류의 현장다짐점토와 3종류의 실내다짐점토에 대한 회복탄성계수실험(Resilient Modulus Test)을 실시하였다. 실험결과에 의하면 현장시료의 회복탄성계수는 동일한 함수비와 건조단위중량으로 다짐형성된 시료의 그것과 상당히 다른 것으로 나타났다. 회복탄성계수는 1% 변형률에서의 일축압축응력과 좋은 상관관계를 가지는 것으로 확인되었으며 이들간의 상관관계는 회귀분석에 의하여 간단한 식과 도표로 제안되었다. 구해진 상관관계는 실내다짐토와 현장다짐토에 대해 약간 다르나, 실제 노반에 존재할 것으로 예상되는 응력조건하에서는 크게 다르지 않다. 제안된 상관관계식 자체는 시공후 지반의 상태변화에 영향을 받지 않으므로 as-compacted나 in-service상태에 모두 적용 가능하다.

Abstract

Resilient modulus tests were performed on five cohesive soils sampled from in-service subgrades and three cohesive soils compacted in the laboratory. It was concluded that in-service resilient modulus can not be estimated from the resilient modulus of laboratory specimen compacted at same water content and dry density as in-service condition. The stress at 1 percent axial strain in unconfined compression test ($S_{u1.0\%}$) was found as a good indicator of the resilient modulus (M_R), and the unique relationship between M_R and $S_{u1.0\%}$ was obtained. This relationship for the laboratory compacted soil is slightly different from that for the field compacted soil and the difference is less pronounced at the confining stress level expected to exist in subgrade. A proposed relationship itself is not affected by the changes in subgrade after construction and, therefore, it is applicable to as-compacted and in-service subgrade conditions.

* 정회원, 삼성건설(주) 기술연구소, 선임연구원

1. Introduction

Extensive cracking of asphaltic pavement results from excessive plastic and elastic deflections. The primary cause of fatigue failure has been attributed to repeated stress applications over a long period of time. Since a significant part of transient deflection of pavement system results from the compression of subgrade, the resilient modulus of subgrade is an important characteristic of the multi−layer pavement system. The 1986 AASHTO guide for the design of pavement structures introduced the resilient modulus as a definitive material property to characterize roadbed soil. Although the incorporation of resilient modulus represents a significant advance in pavement design practice, most highway agencies do not have experience with this test.

Empirical correlations for resilient modulus have been proposed by many researchers (Heuklelom and Klomp, 1962 ; Fredlund et al., 1977 ; Jones and Witczak, 1977 ; Robnett and Thompson, 1976 ; Thompson and Robnett, 1979 ; The Asphalt Institute, 1982 ; Powell et al., 1984 ; Carmichael and Stuart, 1985 ; Brown, 1990 ; Drumm et al., 1990 ; Farrar and Turner, 1991). Most of these correlations were developed from the test results of laboratory compacted specimens and include the properties, such as water content and dry density or the degree of saturation. Since the compaction curves and the lines of optimum of field compaction are often different from those of laboratory compaction(Waterways Experimental Station, 1949 ; Johnson and Sallberg, 1960, 1962 ; Wahls et al., 1966) and since the moisture condition of subgrade changes with time after construction, a use of these correlations may cause a significant error in estimated resilient modulus. Correlations based on CBR (Heuklelom and Klomp, 1962 ; Powell et al., 1984 ; Brown, 1990) or R−value (Farrar and Turner, 1991 ; The Asphalt Institute, 1982) have been used extensively in practice. These correlations show a significant scatter in data and are considered reasonable only within a certain range of CBR or R−value. Most correlations proposed previously do not consider the level of stress and the resilient modulus is estimated as a single value for a given soil.

The purpose of this paper is to develop data base of resilient modulus for the cohesive soils frequently encountered in pavement construction ; to develop reliable correlations between resilient properties and parameters from simple routine tests ; and to compare resilient characteristics of field compacted and laboratory compacted cohesive soils.

2. Definition of Resilient Modulus

As the flow of traffic moves on the pavement surface, a large number of rapid stress pulses of varying magnitude are applied to each element of the pavement structure. Many studies (Grainger and Lister, 1962 ; Seed et al., 1962 ; HRB, 1962) showed that the measured loading pulse is approximately sinusoidal, and that its magnitude decreases with depth while the duration increases with depth. It was also shown by Barksdale(1971) that the magnitude and duration of the loading pulse are the function of the vehicle speed and the depth beneath pavement surface.

Since pavement structures are subjected to a series of loading pulses, a test simulating field conditions should be employed for the determination of resilient characteristics in the laboratory. The repeated-loading triaxial test applying a series of repeated deviator stress pulses separated by a rest period has been used for the determination of resilient modulus of granular and cohesive soils. A conceptual response of a soil sample in the resilient modulus test is shown in figure 1. The resilient modulus is analogous to the modulus of elasticity in the definition. The repeated deviator stress and the recoverable deformation occurring in the test are used to calculate the resilient modulus which is defined as :

$$M_R = \frac{\sigma_d}{\epsilon_R} \quad \cdots\cdots (1)$$

where M_R is the resilient modulus ; σ_d is the repeated deviator stress ; ϵ_R is the recoverable exial strain.

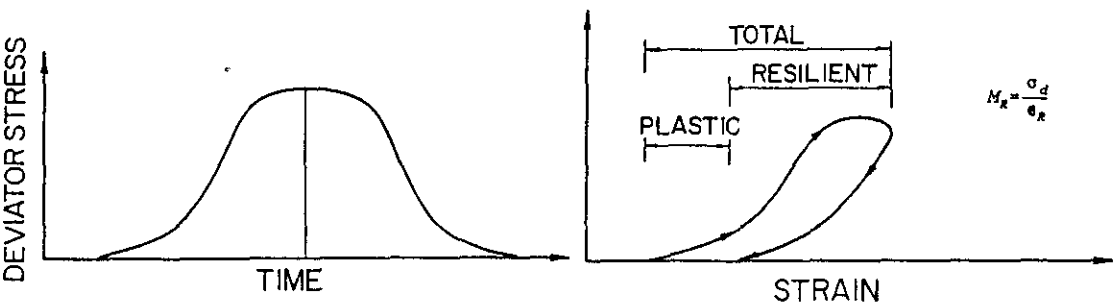

Figure 1. Definition of Resilient Modulus in Repeated-Loading Triaxial Test (After Robnett and Thompson, 1976)

3. Sample Preparation and Laboratory Experimental Program

3.1 Sampling and Preparation of Field Compacted Soil

Typical cohesive soils in Indiana were obtained from five in-service subgrades. Detailed information on sampling sites is given in Table 1. Pavement surface was cored by a coring machine, and base and subbase layers were removed by a hollow-stem auger to expose the top of subgrade. A 70 mm diameter shelby tube lubricated with silicone oil was pushed by a drill rig. The tube was rotated slowly and lifted from the hole. Sealed tubes were transported and stored in an humidity room. Soils were removed from the tube by cutting it longitudinally and were trimmed to heights of 147mm to 155mm. Figure 2 shows gubgrade conditions during construction and in-service for sampled sites, with compaction curves for standard proctor energy. Data for as-compacted condition were taken from the field inspection data measured by a Troxler 3440 moisture–density gauge or by sand cone method during construction.

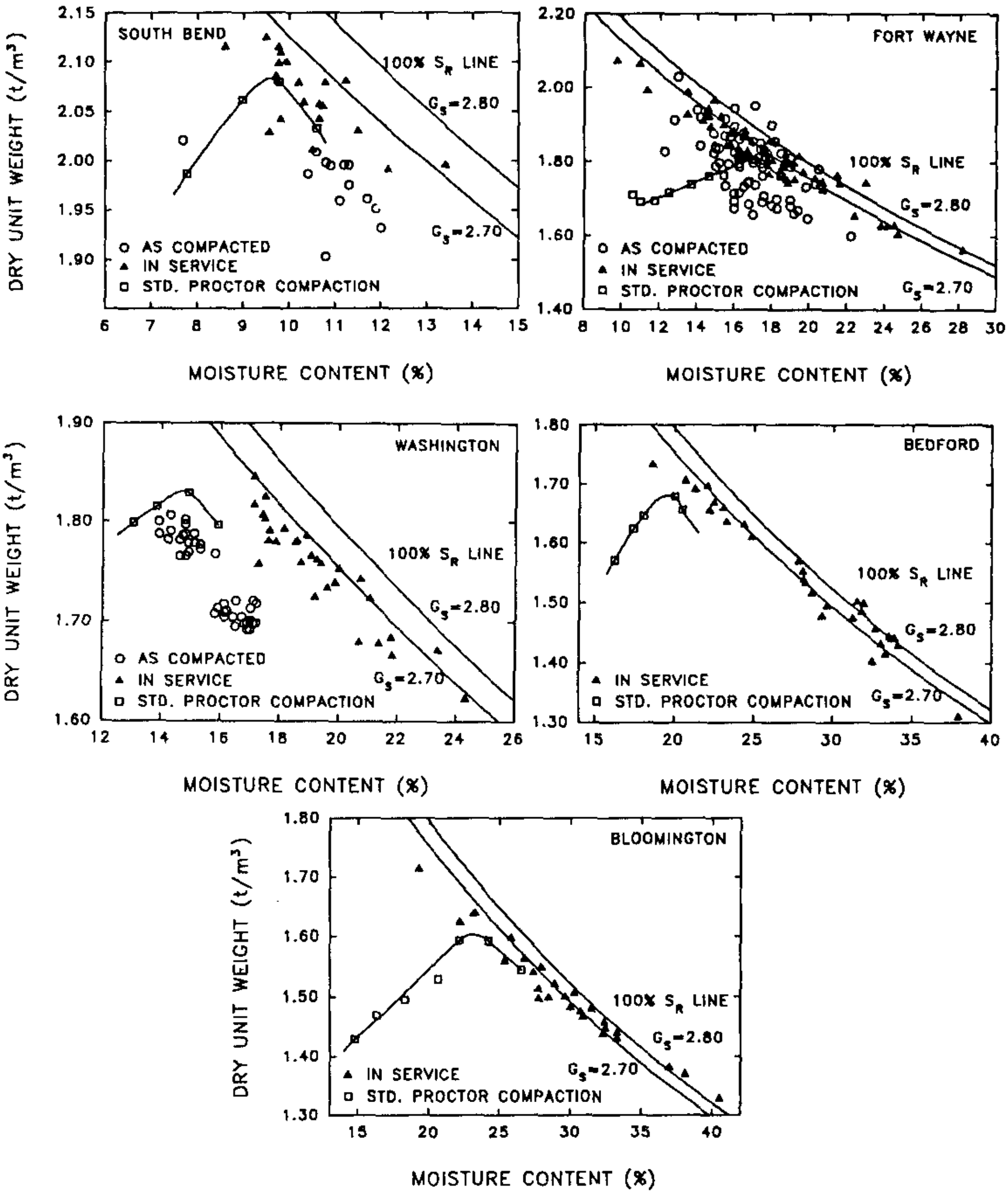

Figure 2. In–Service and As–Compacted Conditions of Tested Sites

Table 1. Information on Sampling Sites

Sites	Location	Station Number	Soil Classification	Date Constructed	Date Sampled
South Bend	US20 Bypass	140−141	A−4/A−6 (CL)	Aug. 89	Jan.; May; July 90
Fort Wayne	US24 Bypass	56, 73, 181	A−6/A−7−6 (CL)	Aug. 87	Sept. 90
Washington	US50 Bypass	290	A−6 (CL)	June 89	Jan. 91
Bedford	SR−37	530−532	A−7−6 (CH)	Aug. 70	April; Sept. 91
Bloomington	SR−37	849	A−7−6 (CH)	Oct. 72	Sept. 91

South Bend was the only site where the soil was sampled before the completion of pavement construction. Embankment fill was compacted with four passes of Catepillar CAT 815 sheepsfoot roller. It was observed by comparing Figures 2 and 3 that the field density is similar to the density obtained by laboratory compaction with energy between 2.197 and 3.271kg · cm/cm^3. At this site, there were the increase in dry density and the decrease in water content after construction of embankment. Since the subgrade at South Bend had not been covered until sampled, the reduction in water content may be due to the drying of the soil. The increase in density may be attributed to the construction traffic for an overpass bridge located at the end of the embankment.

The Fort Wayne soil was sampled at three stations ; all stations were compacted with Caterpillar CAT 825 sheepsfoot roller. The number of passes of the roller were four to six for station 56+00 and four for station 181+00. At station 73+00, the top 0.6 meter of subgrade was compacted with eight to nine passes while the lower part of embankment was compacted with four to six passes. All three stations show an insignificant change in dry density after construction, while the water content varied more widely and the degree of saturation increases slightly after construction.

A compaction of the Washington site was well controlled during construction. One group of data showing higher density and lower water content is for top 0.6 meter of compacted layer, the so−called special subgrade treatment. This site was compacted with three passes of Caterpillar CAT 553 sheepsfoot roller and two passes of a Raygo 404 vibrating sheepsfoot roller. Some of the special subgrade treatment was subjected to 3 passes of the Raygo 404 roller. Since the compactive efforts for the special subgrade treatment and embankment are almost same, the line connecting two groups of field compaction data is considered as a wet side portion of field compaction. This implies that the line of optimum for field compaction locates on the dry side of the line of optimum of laboratory compaction. It is also noted that there was a significant increase in degree of saturation after construction. The in−service condition is formed in a narrow range parallel to the 100 percent saturation line; the in−service density and water content vary over a wider range than the as−compacted condition.

Since the Bedford and Bloomington sites were constructed in early 1970's, information on in−place inspection test was not available. As shown in Figure 2, in−service water content for both sites ranges from 18 to 40 percent and the in−service dry density ranges from 1.31 to 1.73t/m^3. It is noted that the degree of saturation for these sites approaches almost 100 percent.

3.2 Laboratory Compaction

Disturbed soils were obtained from three in−service subgrades in Indiana. They were air dried until the soils became friable, and then they were broken up with a mallet. Properties of the processed cohesive soils are shown in Table 2. After measuring the water content of the processed soil, a calculated amount of water to obtain a desirable compaction water content was mixed into the air−dried soil. Mixed soil was wrapped in a plastic bag and stored in an humidity room for two days before compaction.

Table 2. Properties of Processed Soils

Site	Soil Classification	G_S	γ_{dmax} (t/m^3)	OMC(%)	W_L(%)	PI(%)
South Bend	A−4/A−6 (CL)	2.76	2.08	9.6	19−26	7−14
Washington	A−6 (CL)	2.74	1.83	14.8	28−38	11−19
Bloomington	A−7−6 (CH)	2.79	1.60	23.5	44−62	25−40

Each specimen was compacted in five layers of approximately equal height with a rammer of 1.6kg and drop height of 30 cm. A compaction mold of 73mm in diameter and 160mm in height was used. To validate the compaction method used in this study, three different compactions using different sizes of mold and hammer were carried out with similar compactive efforts. Result of these compactions are shown in Figure 3. It can be seen that similar compaction curves were created with almost identical maximum dry density; the optimum water content for Standard Proctor procedure is only about 0.6 percent larger than the others. Therefore, it is concluded that the compaction used in this study is compatible with both existing standard and modified proctor compaction procedures.

Before adding a new layer of soil, the surface of the previously compacted layer was scarified in order to promote bonding between layers. Four different levels of compaction energy were used and a series of at least four specimens was compacted for each level of compactive effort. Compactive efforts and blows per layer used for laboratory compaction are given in Table 3. Following the compaction, the extension collar was removed and the compacted specimen was carefully trimmed even with the top of the mold by means of a

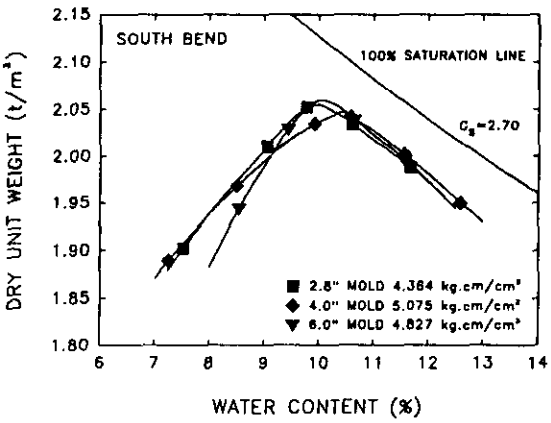

Figure 3. Effects of Mold and Rammer on Compaction

straightedge. After measuring the weight of mold and soil, the specimen was carefully extruded using a hydraulic jack. The extruded specimen was wrapped in a plastic bag and stored in an humidity room for two days before being tested. Results of laboratory compaction for South Bend and Washington soils are summarized in figure 4.

Table 3. Details of Laboratory Compaction

Compaction Energy kg'cm / cm^3(lb'ft / ft^3)	Total Blows	Blows per layer
6.037(12, 364)	82	16, 16, 16, 17, 17
4.364(8, 938)	60	12, 12, 12, 12, 12
3.273(6,703)	45	9, 9, 9, 9, 9
2.182(4,469)	30	6, 6, 6, 6, 6

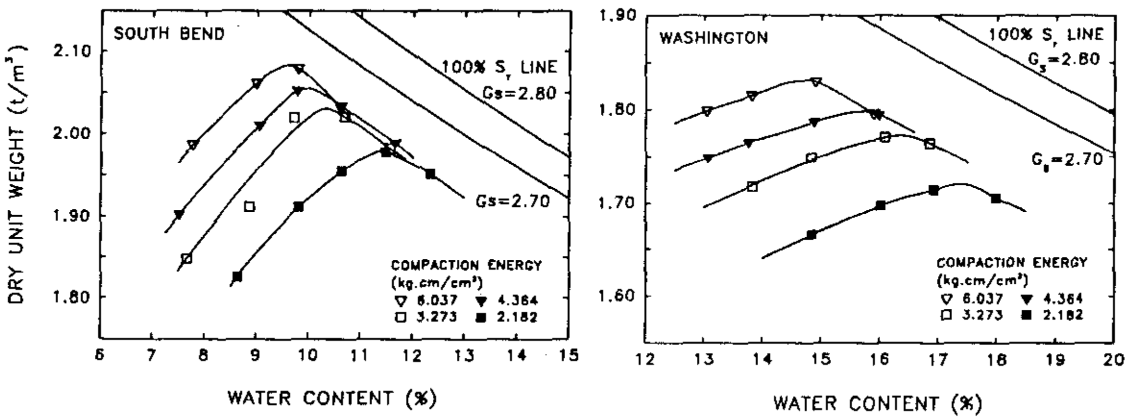

Figure 4. Laboratory Compaction Results of South Bend and Washington Soils

3.3 Unconfined Compression Test

After applying the rubber membrane on the trimmed specimen, an unconfined compression test was carried out up to 1.5mm deformation with 1 percent strain per minute(0.025mm/second), following the procedure described in ASTM D 2166-85. Since the level of stress applied to the specimen during the resilient modulus test is less than 0.7kg/cm^2 and the deformantion by dynamic loading is significantly smaller than that by static loading, a possibility to characterize resilient behavior by the parameters at small unconfined compressive strains was investigated. Stresses at different strain levels(0.25, 0.50, 0.75, and 1.0 percent), secant moduli at different deviator stress levels(0.07, 0.14, 0.35, and 0.7kg/cm^2) and initial tangent moduli were obtained from the unconfined compression test results for the comparison with parameters from resilient modulus test results.

3.4 Resilient Modulus Test

After the unconfined compression test, the resilient modulus test was conducted on the same specimen. Since the resilient modulus test has a conditioning stage and the deformation during the unconfined compression test is only 1.5mm, performing the unconfined compression test before the resilient modulus test is not believed to have had a significant effect on the resilient modulus test result. The resilient modulus tests in this study were performed by following the procedure described in AASHTO T 274-82. The test involves two stages of testing: conditioning and the main test. Conditioning is intended to minimize the effect of the interval between sampling and testing and of initially imperfect contact between the platens and the specimen. The main part of resilient modulus test for cohesive soil consists of 15 different combinations of confining stresses and repeated deviator stresses, applying 200 stress pulses for each stress combination. The haversine stress pulse with a load duration of 0.1 second and a cycle duration of 1 second was used to apply repeated loads to the specimen. Figure 5 is a plot of measured stress pulse. A typical

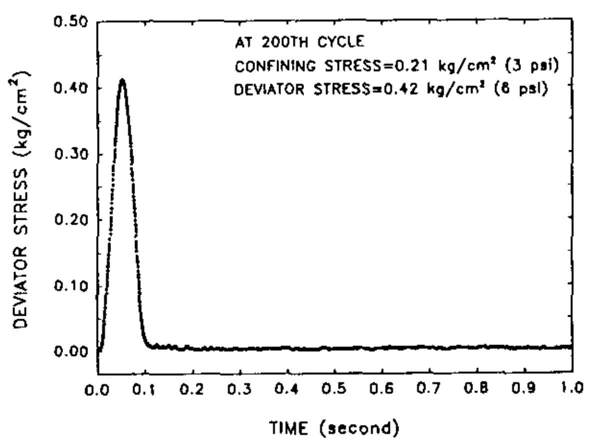

Figure 5. Stress Pulse Measured during the Resilient Modulus Test

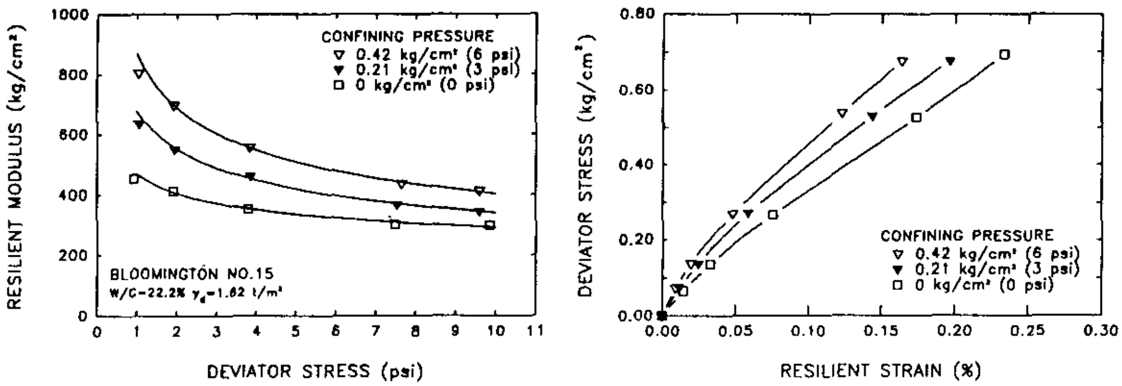

Figure 6. Typical Result of the Resilient Modulus Test on Cohesive Soil

result of the test is plotted as three curves corresponding to the different level of confining stresses, as shown in figure 6.

Equipment used for the resilient modulus test is composed of a loading frame with controlling and conditioning units, including a waveform generator, an oscilloscope monitoring the input and output stress pulse, and a PC controlling the M_R test and performing data acquisition. The load frame is a MTS−810 Soil Test System and is a closed−loop servo−hydraulic system. A load transducer of 100kg capacity is located between the loading piston and the triaxial cell. Deformation of the specimen was measured by a 4−mm displacement gage whose resistance is changed by the movement of sensing arm. A software package developed by MTS Systems Corporation was modified and used for the control and data acquisition, with a DT 2801 board, RS−232 interface and PCLAB software. Magnitudes of confining pressure and repeated deviator stress are automatically changed. Confining pressure is controlled by supplying air through a Bellofram pressure regulator actuated by the voltage signal sent from the controlling computer.

4. Test Results and Discussions

4.1 Resilient Properties of Field Compacted Cohesive Soils

Generally, the resilient modulus was observed to increase with the increase in dry density or the decrease in water content. However, these relationships are too scattered to give a good correlation. As shown by Lambe(1958), Seed and Chan(1959), Barden and Sides(1970), and Hodek and Lovell(1979), the soil structure and engineering properties are determined by as−compacted condition rather than in−service condition. Seed et al. (1962) and Monismith et al. (1967) also showed that the resilient behavior of soil initially compacted dry of optimum and subjected to an increase in water content to wet of optimum is different from that of soil compacted directly to wet of optimum. Therefore, it was concluded that the in－service condition is not a good basis for relating structure and engineering properties of compacted cohesive soils.

To select the best indicators of resilient modulus, a stepwise regression technique was applied to the independent variables ; such as the parameters from the unconfined compression test, the in－service water contents and dry densities. Among all the variables considered, the stresses at 0.25, 0.5, 0.75, and 1.0 percent axial strain($S_{u0.25\%}$, $S_{u0.5\%}$, $S_{u0.75\%}$, $S_{u1.0\%}$) are found to have the best correlations with the resilient modulus for all the sites investigated. Since the stresses at smaller axial strains appear to be subjected to the larger error due to the incorrect readings or the imperfect contact between a specimen and a top cap, and since the coefficient of determination tends to increase as the range of independent variable becomes smaller, the stress at 1 percent axial strain is adopted as a predictor variable. The results of repeated－load triaxial tests and unconfined compression tests are expected to be affected by the same factors, and relating both test results directly may exclude the influence of water content, dry density and other basic indices. Because the resilient strain caused by repeated－loading triaxial test ranges from 10^{-3} to 10^{-5}, $S_{u1.0\%}$ instead of the unconfined compressive strength would give a better indication of the resilient modulus. It is also noted that all sites, except South Bend which has no data of unconfined compression test, show the similar relationships between M_R and $S_{u1.0\%}$.

To investigate the possibility of an unified correlation, data from four sites are plotted together in a single plot, as shown in Figure 7. It can be seen that there exists an unique relationship between M_R at a specific level of stress and $S_{u1.0\%}$, regardless of the soil type. As expected, the resilient modulus increases as the magnitude of $S_{u1.0\%}$ increases. The resilient modulus at higher confining stress is larger than that at lower confining stress and the difference becomes larger as $S_{u1.0\%}$ increases. For soils with $S_{u1.0\%}$ of 2.0kg/cm², the resilient modulus at σ_3 of 0.42kg/cm²(6 psi) is about 50 percent larger than that at no confinement. Therefore, consideration of the effect of confining stress on M_R is important when the subgrade is a relatively stiff clay.

Robnett and Thompson(1976) suggested that the breakpoint resilient modulus obtained from the bilinear representation of M_R versus σ_d is a representative modulus which can be used for correlation and design purposes. They used this single modulus to characterize the entire M_R versus σ_d response and recommended it as the resilient modulus at a deviator stress of 0.42kg/cm²(6 psi) for practical purpose. In the present study, the resilient modulus at a deviator stress of 0.42kg/cm²(6 psi) and at a confining stress of 0.21kg/cm²(3psi) is used as a representative. To develop the empirical correlation, a multiple regression was performed and the addition of variables except $S_{u1.0\%}$ was found to improve the predictability insignificantly. Therefore, a following equation based on $S_{u1.0\%}$ was developed :

$$M_R = -112.5+833.8S_{u1.0\%}-99.11S_{u1.0\%}^2 \quad \cdots\cdots (2)$$

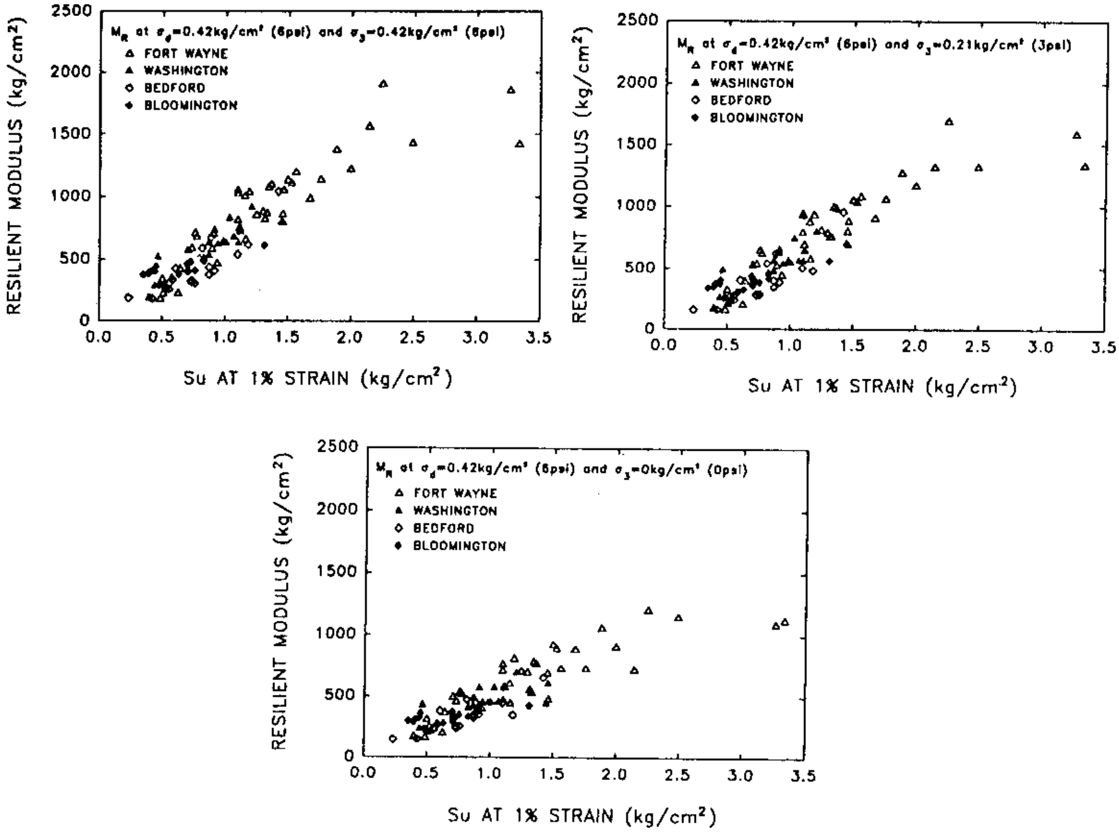

Figure 7. Relationship between M_R and $S_{u1.0\%}$ for Field Compacted Soils

Equation 2 gives the coefficient of determination (R^2) of 0.85 and square root of Mean Square Error of 128.8kg/cm^2. It is also observed that the resilient modulus is linearly proportional to $S_{u1.0\%}$ for soils having $S_{u1.0\%}$ less than 2.5kg/cm^2. For $S_{u1.0\%}$ less than 2.5kg/cm^2, the following simple relationship is valuable :

$$M_R = 606.6 S_{u1.0\%} \quad \cdots\cdots (3)$$

where M_R is the resilient modulus (kg/cm^2) at σ_3 of 0.21kg/cm^2(3 psi) and σ_d of 0.42kg/cm^2 (6 psi) ; and $S_{u1.0\%}$ is the stress causing 1 percent strain in the unconfined compression test. Equation 3 gives R^2 of 0.97 and square root of Mean Square Error of 121.6kg/cm^2. Figure 8 shows the prediction equations proposed in this study with 95 percent confidence and prediction intervals.

Since relationships between M_R and $S_{u1.0\%}$ for four soils which have been subjected to different traffic loadings and different changes in subgrade condition are similar, it appears that the relationship itself is not affected by the change in subgrade condition after construction and that it is applicable to the as−compacted condition as well as the in−service condition.

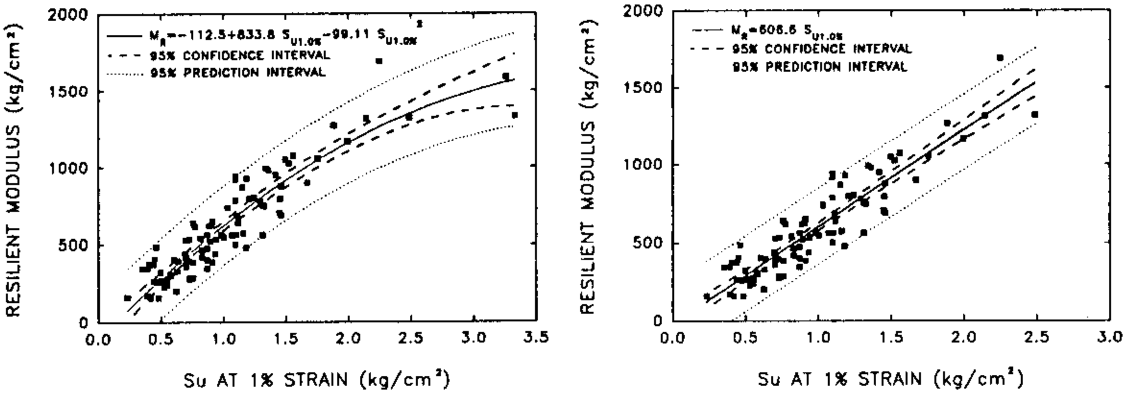

Figure 8. Proposed Relationship between M_R and $S_{u1.0\%}$ for Field Compacted Soils

4.2 Resilient Properties of Laboratory Compacted Cohesive Soils

Figure 9 is a plot of test results showing the changes of $S_{u1.0\%}$ with molding water content. It is shown than $S_{u1.0\%}$ starts to decrease at an increasing rate as water content approaches 1.0 to 1.5 percent dry of optimum. $S_{u1.0\%}$ decreases more slowly near the optimum water content, beyond which it is virtually constant. A larger compactive effort produces a greater $S_{u1.0\%}$ at water contents dry of optimum, while the reverse is true at wet of optimum. Figure 10 is a plot showing the relationship between the resilient modulus at σ_d of 0.42kg/cm²(6 psi) and σ_3 of 0.21 kg/cm² (3 psi) and the compaction water content.

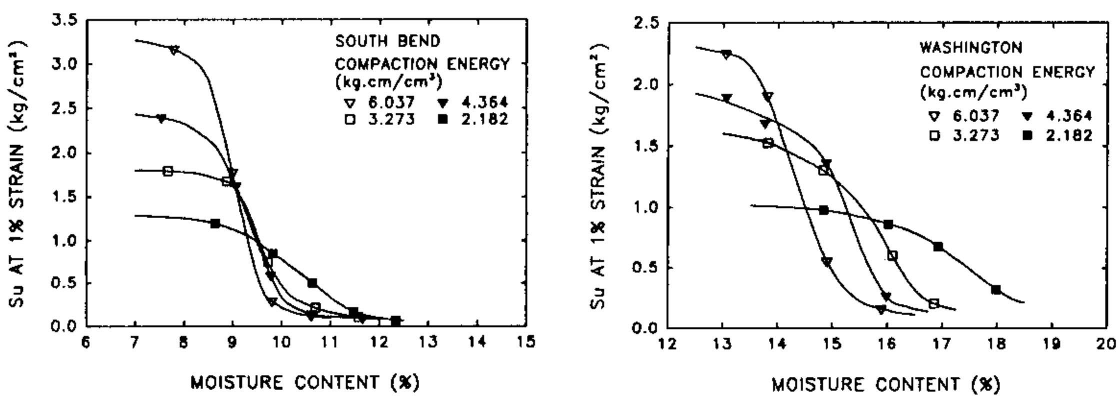

Figure 9. Variation of $S_{u1.0\%}$ with Water Content for Laboratory Compacted Soils

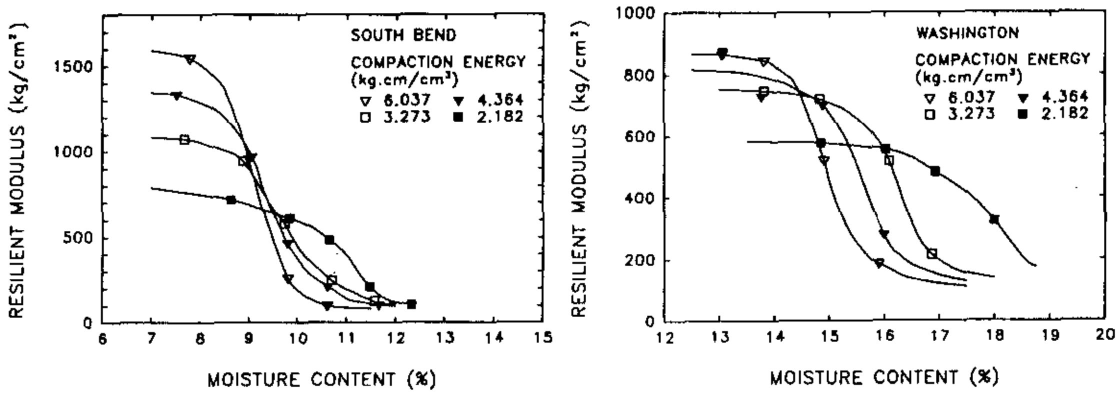

Figure 10. Variation of M_R with Water Content for Laboratory Compacted Soils

Since it is observed from Figures 9 and 10 that M_R .and $S_{u1.0\%}$ vary with water content in a similar manner, a direct comparison of both variables may offer a good correlation. Figure 11 is a plot of M_R versus $S_{u1.0\%}$ for tested sites. Plots for the Washington and South Bend sites are identical up to $S_{u1.0\%}$ of 1.5kg/cm^2 while M_R of the South Bend soil is slightly larger than that of the Washington soil as $S_{u1.0\%}$ increases. It is noted that test results on specimens compacted with four different compactive efforts fit well in a single relationship between M_R and $S_{u1.0\%}$, and that the relationship appears unique for a given soil. Therefore, it is concluded that the relationship between M_R and $S_{u1.0\%}$ for a given soil can be obtained by conducting resilient modulus tests on a series of only four or five specimens compacted at different molding water content with the same compactive effort. To look at the possibility of an unified relationship, four specimens of the Bloomington soil were compacted with Standard Proctor energy, and tested for the resilient modulus. It can be seen from figure 11 that the relationship between M_R and $S_{u1.0\%}$ of the Bloomington soil is not significantly different from those of South Bend and Washington sites.

Since the relationship between M_R and $S_{u1.0\%}$ of laboratory compacted soil shows a curvature, a square term of $S_{u1.0\%}$ is added to the prediction equation. The relationship applicable to the soils studied is developed by the regression analysis as follows ;

$$M_R = 695.4S_{u1.0\%} - 84.34S_{u1.0\%}^2 \quad \cdots\cdots (4)$$

where M_R is the resilient modulus(kg/cm^2) at σ_d of 0.42 kg/cm^2(6 psi) and σ_3 of 0.21kg/cm^2 (3 psi); and $S_{u1.0\%}$ is the stress(kg/cm^2) causing 1 percent unconfined compressive strain. The coefficient of determination(R^2) is 0.97 and square root of Mean Square Error is 115.4kg/cm^2.

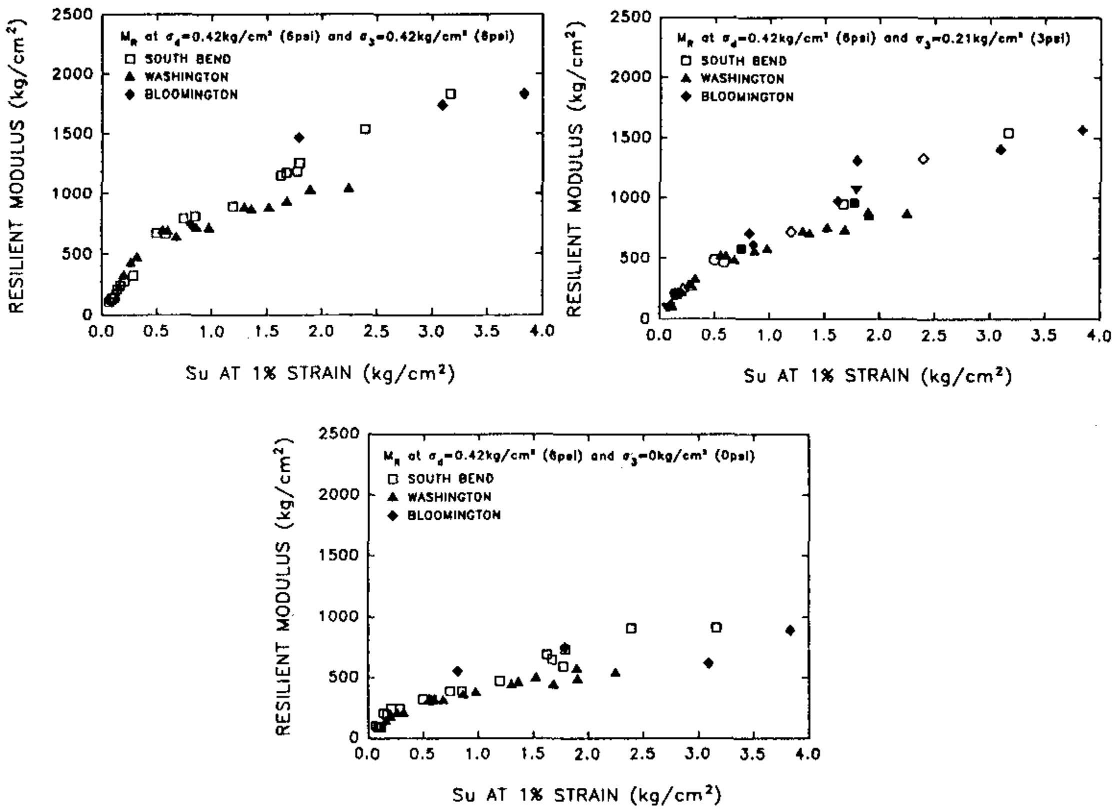

Figure 11. Relationship between M_R and $S_{u1.0\%}$ for Laboratory Compacted Soils

5. Resilient Properties of Laboratory and Field Compacted Soil

5.1 Laboratory and Field Compaction

The primary purpose of laboratory compaction is to produce the compacted specimen having identical properties with the field compacted soil. Presently, it seems that no satisfactory laboratory compaction method exists that can exactly duplicate field compaction. Waterways Experimental Station(1949) conducted laboratory and field investigations on a compacted Vicksburg silty clay, and showed that the strength of laboratory compacted sample was quite different from that of the field compacted soil. Johnson and Sallberg(1960, 1962) and Wahls et al. (1966) reviewed previous studies, and concluded that the attempts to devise the laboratory compaction methods that simulate field compaction were unsuccessful. It was noted that the lines of optimum for field and laboratory compaction often did not coincide and the relative position of the lines of optimum with respect to the 100 percent saturation line varied for various laboratory and field compaction methods and for various soil types. Lambe(1958), Olson(1963), and Seed and Chan(1959) have recognized that similarities in molding water content and dry density do not necessarily imply similarities in engineering properties of the compacted soil.

5.2 Resilient Modulus of Laboratory and Field Compacted Soil

Seed et al. (1962, 1967) compared the range of measured resilient moduli of undisturbed samples with that of moduli estimated from the test results of laboratory compacted samples, and concluded that the properties of clays compacted by rubber-tire rollers are similar to those prepared by kneading compaction in the laboratory and, therefore, these two compaction methods induce similar soil structures. However, Lee et al. (1992) reinterpreted the data by Seed et al. (1962, 1967), and showed that the resilient modulus estimated from the test result of laboratory compacted soil is significantly different from that of the field compacted soil. The difference is attributed to the different moisture—density condition between laboratory and field compacted soils and the change in subgrade condition after construction. This clearly indicates that the replication of field density and water content by laboratory compaction will not allow a good estimation of field resilient modulus.

By comparing the M_R versus $S_{u1.0\%}$ relationship between field and laboratory compacted soils shown in figures 7 and 11, it is observed that the relationship at no confining stress is slightly different while relationships with confining stress are not significantly different between two compactions. At $S_{u1.0\%}$ less than $0.7kg/cm^2$, laboratory compacted soil has a highly nonlinear relationship and field compacted soil shows less curvature. At low confining stress, the resilient modulus of laboratory compacted soil is moderately smaller than that of field compacted soil at $S_{u1.0\%}$ larger than $1.4kg/cm^2$ and they are similar at smaller $S_{u1.0\%}$. At higher confining stress, laboratory compacted soil shows slightly smaller M_R than field compacted soil at $S_{u1.0\%}$ larger than $1.75kg/cm^2$ and slightly larger M_R than field compacted soil at $S_{u1.0\%}$ smaller than $1kg/cm^2$. For soils having $S_{u1.0\%}$ ranging from 0.35 to $1.4kg/cm^2$, field resilient modulus is approximately same as laboratory resilient modulus.

The difference in resilient characteristics of laboratory and field compacted soils is due to the different soil structure, the traffic load, type of soil, and the change of subgrade condition after construction. Since relationships between M_R and $S_{u1.0\%}$ for four soils which have been subjected different traffic loadings and different changes in subgrade condition were similar, the difference in relationships of M_R versus $S_{u1.0\%}$ between laboratory and field compactions is mainly due to the different soil structure. Since the relationships of M_R versus $S_{u1.0\%}$ for laboratory and field compacted soils are not significantly different at confining stress larger than $0.21kg/cm^2$, the field resilient modulus may be estimated from laboratory compacted soil using the relationship suggested by Equations 2 or 3.

6. Resilient Modulus at Different Level of Stresses

For mechanistic analysis of pavement system, it is necessary to express the resilient modulus in terms of deviator stress and confining stress since the level of stresses due to traffic loading is again a function of resilient characteristics of pavement materials. From regression analysis on resilient moduli at different levels of confining stress and deviator stress, the resilient modulus of cohesive soil is expressed as ;

$$M_R = AS_{u1.0\%} \quad \cdots (5)$$

where M_R is the resilient modulus at specific level of confining stress and deviator stress ; $S_{u1.0\%}$ is the stress causing 1 percent strain in unconfined compression test ; and "A" is a regression parameter. A chart to evaluate the regression parameter is developed as a function of confining stress and deviator stress, as shown in Figure 12. The resilient modulus can, then, be obtained by estimating the regression parameter from Figure 12 and substituting it into the Equation 5. By repeating this procedure and plotting calculated resilient moduli with corresponding deviator stress and confining stress, relationships of resilient modulus with deviator stress for each confining stress level are established.

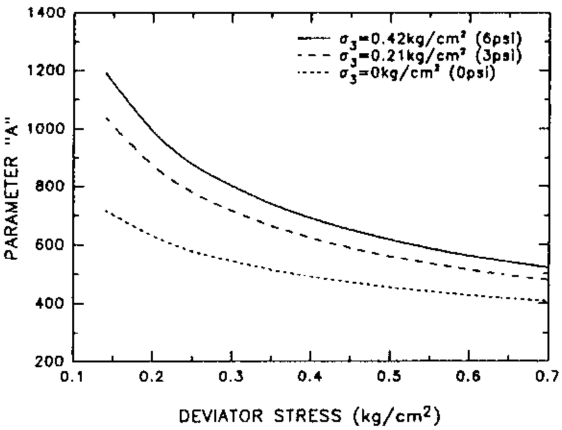

Figure 12. Chart for Estimation of Parameter in Equation 5

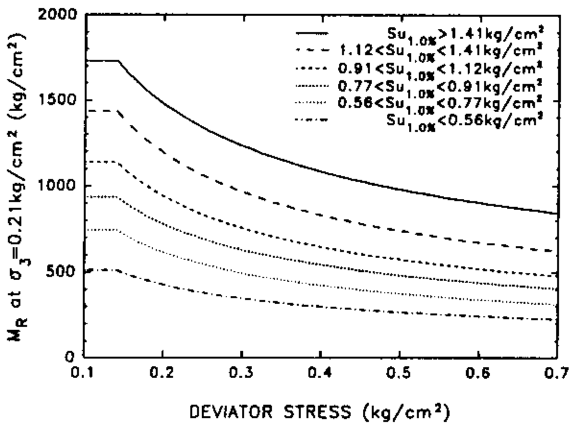

Figure 13. Relationships between M_R and σ_d for Each Group of $S_{u1.0\%}$

Another chart to determine the relationship between M_R and σ_d is developed for more convenient use. All the test results of field compacted soils are sorted and grouped by means of $S_{u1.0\%}$, and Figure 13 shows relationships between M_R and σ_d at σ_3 of 0.21kg/cm^2 (3 psi).

For both approaches proposed above, the parameters at σ_d less than 0.14kg/cm^2 (2 psi) are assumed identical to those at σ_d of 0.14kg/cm^2.

7. Conclusions

From the observations and analyses on the test results of cohesive soils, following conclusions are drawn :

(1) Field compaction yields similar or lower dry density than that obtained by the laboratory compaction with Standard Proctor energy. The in-service condition of cohesive subgrade approaches the 90 to 100 percent degree of saturation after several years of service, regardless of as-compacted condition ;

(2) Generally, in-service resilient modulus increases with increase in dry density or decrease in water content. However, these relationships are highly scattered since the soil structure is affected by the condition during compaction rather than in-service condition. Therefore, replication of in-service density and water content by laboratory compaction does not give good estimation of resilient properties of in-service subgrade soil ;

(3) In-service resilient modulus of the field compacted cohesive soil is uniquely related to $S_{u1.0\%}$, regardless of soil type. The relationship itself between M_R and $S_{u1.0\%}$ (Equation 2 or 3) is applicable to the as-compacted condition as well as the in-service condition. Therefore, the resilient modulus can be estimated for any subgrade conditions if $S_{u1.0\%}$ is obtained for the specimen which has been subjected to corresponding conditions ;

(4) For laboratory compacted cohesive soils, the relationship between M_R and $S_{u1.0\%}$ for a given soil is unique regardless of compaction water content and compactive effort. It can be developed by conducting resilient modulus tests on a series of four or five specimens compacted at different molding water content with the same compactive effort ;

(5) The relationship between M_R and $S_{u1.0\%}$ at no confining stress is slightly different between the field and the laboratory compacted soils while that with confining stress is not significantly different between two compactions ;

(6) An addition of end-result specification of $S_{u1.0\%}$ to the current practice of compaction specification will allow field engineers to better control the resilient properties of the constructed subgrade.

ACKNOWLEDGEMENTS

This work was supported by the Indiana Department of Transportation and the Federal Highway Administration through the Joint Highway Research Project at Purdue University. The writer would like to thank MTS Systems Corporation for the help developing the resilient modulus testing system.

REFERENCES

AASHTO Guide for Design of Pavement Structures. (1986). Amer. Assoc. of State Hwy. and Transp. Officials, Washington, D.C.

AASHTO Designation : T 274-82 "Standard method of test for resilient modulus of subgrade soils." (1982). Amer. Assoc. of State Hwy. and Transp. Officials, Washington, D.C.

The Asphalt Institute. (1982). "Research and development of the Asphalt Institute's thickness design manual, ninth edition." Research Report No. 82-2.

Barden, L., and Sides, G. R. (1970). "Engineering behavior and structure of compacted clay." J. Soil Mech. Found. Engrg., ASCE, 96(4), 1171-1200.

Barksdale, R. D. (1971). "Compressive stress pulse times in flexible pavements for use in dynamic testing." Hwy. Res. Record 345, 32-44.

Brown, S. F. (1990). "The relationship between California Bearing Ratio and elastic stiffness for compacted clays." Ground Engrg., 23(8), 27-31.

Carmichael III, R. F., and Stuart, E. (1985). "Predicting resilient modulus: a study to determine the mechanical properties of subgrade soils." Transp. Res. Record 1043, 145-148.

Drumm, E. C., Boateng-Poku, Y., and Pierce, T. J. (1990). "Estimation of subgrade resilient modulus from standard tests." J. Geotech. Engrg. ASCE, 116(5). 774-789.

Farrar, M. J., and Turner, J. P. (1991). "Resilient modulus of Wyoming subgrade soils." Mountain-Plains Consortium Report No. 91-1.

Fredlund, D. G., Bergan, A. T., and Wong, P. K. (1977). "Relation between resilient modulus and stress conditions for cohesive subgrade soils." Transp. Res. Record 642, 73-81.

Grainger, G. D., and Lister, N. W. (1962). "A laboratory apparatus for studying the behavior of soils under repeated loading." Geotechnique, 12(1), 3-14.

Heuklelom, W., and Klomp, A. J. G. (1962). "Dynamic testing as a means of controlling pavements during and after construction." Proc. 1st Int. Conf. on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Michigan, 667-679.

Highway Research Board. (1962). The AASHO Road Test. Report 6. Special Studies. Special Report 61F.

Hodek, R. J., and Lovell, C. W. (1979). "A new look at compaction process in fills." Bull. Assoc. of Engrg. Geol., 16(4), 487-499.

Johnson, A. W., and Sallberg, J. R. (1960). "Factors that influence field compaction of soils: compaction characteristics of field equipment." Hwy. Res. Board Bull. 272, HRB.

Johnson, A. W., and Sallberg, J. R. (1962). "Factors influencing compaction test results." Hwy. Res. Board Bull. 319, HRB.

Jones, M. P., and Witczak, M. W. (1977). "Subgrade modulus on the San Diego test road." Transp. Res. Record 641, 1-6.

Lambe, T. W. (1958). "The engineering behavior of compacted clay." J. Soil Mech. Found. Engrg., ASCE, 84(2).

Lee, Woojin, Bohra, N. C., and Altschaeffl, A. G. (1992). Discussion to "Estimation of subgrade resilient modulus from standard tests." J. Geotech. Engrg., ASCE, 118(2), 352-354.

Monismith, C. L., Seed, H. B., Mitry, F. G., and Chan, C. K. (1967). "Prediction of pavement deflections from laboratory tests." 2nd Int. Conf. on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Michigan, 109−140.

Olson, R. E. (1963). "Effective stress theory of soil compaction" J. Soil Mech. Found. Engrg., ASCE, 89(2), 27−45.

Powell, W. D., Potter, J. F., Mayhew, H. C., and Nunn, M. E. (1984). "The structural design of bituminous roads." TRRL Report LR 1132.

Robnett, Q. L., and Thompson, M. R. (1976). "Final report−resilient properties of subgrade soils." Transp. Engrg. Series No. 14, Univ. of Illinois at Urbana−Champaign, Urbana, Illinois.

Seed, H. B., Chan, C. K., and Lee C. E. (1962). "Resilience characteristics of subgrade soils and their relation to fatigue failures in asphalt pavements." Proc. 1st Int. Conf. on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Michigan, 611−636.

Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K. (1967). "Prediction of flexible pavement deflections from laboratory repeated−load tests." NCHRP Report 35, HRB.

Seed, H. B., and Chan, C. K. (1959). "Structure and strength characteristics of compacted clays." J. Soil Mech. Found. Engrg., ASCE, 85(5), 87−128.

Thompson, M. R., and Robnett, Q. L. (1979). "Resilient properties of subgrade soils." J. Transp. Engrg., ASCE, 105(1), 71−89.

Wahls, H. E., Fisher, C. P., and Langfelder, L. J. (1966). "The compaction of soil and rock materials for highway purposes." U. S. Dept. of Transp., FHWA, Report No. FHWA−RD−73−8.

Waterways Experimental Station (1949). "Soil compaction investigation : compaction studies on silty clay." Waterways Experimental Station Tech. Memo. No. 3−271, U.S. Corps of Engineers, Vicksburg, Mississippi.

(접수일자 1993. 10. 7.)