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PREDICTION OF PAVEMENT ROUGHNESS.(U)
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PREDICTION OF PAVEMENT ROUGHNESS

by

Walter R. Barker

Geotechnical Laboratory
U. S. Army Engineer Waterways Experiment Station
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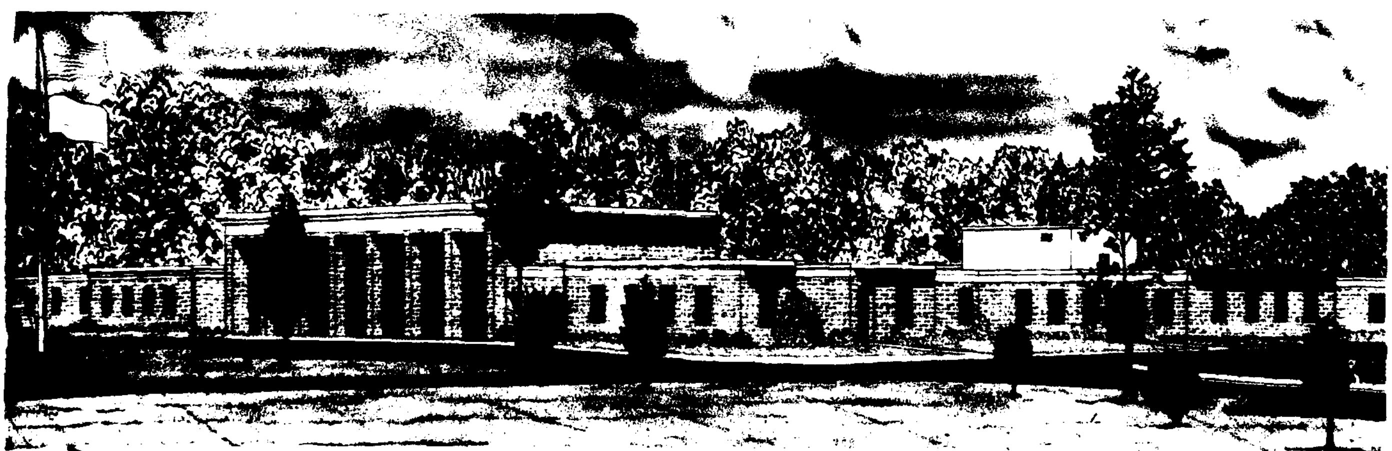
September 1982
Final Report



Prepared for Assistant Secretary of the Army (R&D)
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Under Project No. 4A161101A910, Task Area 02
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A simplified procedure has been developed for considering roughness in pavement design. The procedure utilizes statistical parameters to generate stochastic pavement material properties and profiles. A methodology was developed for predicting the rutting of a pavement section due to applied traffic. Thus the predicted rut depth can be applied to the profiles yielding a predicted surface profile. This profile can then be used to determine a measure of pavement roughness.

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Preface

The study reported herein was performed during the period January 1980 to September 1982 under Department of the Army Project No. 4A161101A91D, In-House Laboratory Independent Research (ILIR) Program, Task Area A2, Work Unit 13906, sponsored by the Assistant Secretary of the Army (R&D).

The report was prepared by Dr. W. R. Barker of the Pavement Systems Division (PSD), Geotechnical Laboratory (GL), U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss. During the conduct of the study Mr. A. H. Joseph was Chief of PSD and Messrs. H. H. Ulery and J. W. Hall were Acting Chiefs of PSD. The Chief of GL was Dr. W. F. Marcuson III.

Commanders and Directors of the WES during the performance of this work and the preparation of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. Fred R. Brown

Contents

Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (force) per square inch	6.894757	megapascals
knots (international)	0.5144444	metres per second
pounds (force) per square inch	6894.757	pascals

PREDICTION OF PAVEMENT ROUGHNESS

Introduction

For many years the emphasis in pavement research has been directed toward the development of a mechanistic approach to pavement design. Nearly all such design procedures have been based on an empirical correlation between pavement distress and computed pavement response parameters. The computed response parameters are normally based on a deterministic evaluation of material properties and pavement section geometries. These design procedures do represent an advancement in the state of the art of pavement design in that they provide a mechanism for considering basic material properties and pavement response. A shortcoming of these procedures is the fact the procedures do not lead directly to a prediction of the functional performance of a pavement. For military airfields the concept of functional requirements is particularly important. The importances of functional requirements are illustrated in the studies of methods for repairing bomb damage and the minimum pavement requirement for alternate pavements to be used only in emergency. In each of these examples the conventional definition of pavement distress becomes meaningless. The principal consideration is the effect the pavement surface has on the operating aircraft. An example of the acceleration forces generated by a B-52 aircraft operating on a hypothetical sinusoidal profile is shown in Fig. 1 (Horn 1977). For the hypothetical sinusoidal profile the critical wave length for various aircraft speeds is shown in Fig. 2 (Horn 1977). Thus it is seen that the forces generated are dependent on both the amplitude, frequency of rutting, and speed of the aircraft With this in mind then the prediction of pavement roughness is seen to involve not only the prediction of a rut depth but also the prediction of the distribution of the rutting.

Scope

This report presents a general methodology for predicting pavement roughness and a specific methodology for predicting rutting in flexible pavements. No specific method was developed for predicting of roughness for rigid pavements nor for the roughness caused by swell and frost heave of pavements.

Approach

The prediction of pavement roughness involves the generation of hypothetical pavements that meet known statistical parameters for surface profile, thickness and material properties. For each of these pavements a mechanistic model is used to predict the development and distribution of rutting with traffic. Thus at any time in traffic a profile for each pavement can be generated and the resulting profile analyzed as to the effect on using aircraft by using an aircraft simulation program, such as the one described by Horn (1977). If a sufficient number of hypothetical pavements are analyzed, then a distribution of roughness parameters can be generated such that the probability that a certain roughness develops is predicted.

Generation of Hypothetical Pavements

The generation of a series of parametric values that meet statistical data may be accomplished in a number of ways. One of the simplest is the use of a distribution function by which a series of values will be distributed. The values can be selected randomly from the distribution such that resulting set will have no order but will have a specified mean and standard deviation. The most commonly used function, the normal distribution, has some undesirable properties, mainly that the possible values are unbounded and the data are symmetrically distributed. Possibly a better distribution function would be the Beta function as described by Harr (1977). The Beta function overcomes some of the disadvantages of the normal distribution in that it may be skewed and has a specified minimum and maximum value. A computer program was written (Appendix A) that provides a set of values that are distributed according to the Beta function and has a given mean, standard deviation, minimum value and a maximum value.

Consider for example the generation of 40 values having a mean of 6.7, a standard deviation of 2.2, a minimum value of 2 and a maximum value of 15. The values generated by the computer program having the required statistical properties are given in Appendix A. The distribution of the values is shown in Fig. 3. Such a distribution could very well represent the values of soil strength in terms of California Bearing Ratio (CBR) where CBR values are bounded and have a skewed distribution. The parameters describing a pavement section can also be generated using the Beta distribution function.

From a statistical point the randomly generated values satisfy the requirement of the problem. If in the generation of the properties for a hypothetical pavement, the points for which properties are being generated are sufficiently far apart that the properties are independent of one another, then the procedure is correct. For airfield pavements where the critical wavelength is rather long and properties are only generated for these points, then the routine may be sufficiently correct to use. If the shorter wavelengths are to be considered and the properties at one point are influenced by the properties of adjacent points, then additional restraints must be placed on the generation scheme.

Dr. Per Ullidtz developed such a routine for prediction of roughness in highway pavements where the critical wavelengths are much shorter than in airfield pavements. Dr. Ullidtz first tried what he called a random walk in which the mean of a parameter distribution was set equal to the value of the previous point. This procedure resulted in unrealistically smooth pavements and thus the scheme was modified to what Dr. Ullidtz named the modified random walk. This scheme is different in that the mean of the parameter distribution is a projection of the values of the previously determined two values. The two schemes are illustrated in Fig. 4 (Fig. 2 of Ullidtz).

In a study of the significant characteristics of runway roughness, Berens and Newman (Berens, 1973, AFFDL-TR-73-109) developed a computer program for generating hypothetical runway profiles that had a specified power spectral density (PSD).

also the PSD of a number of different pavements were determined and a typical new pavement PSD was estimated. Thus using the computer program, a profile could be determined that would simulate a new pavement. In the simulation process a methodology was developed for random spacing of the different waves along the pavement profile. An example of the results of the simulation process is shown in Fig. 5.

The problem of developing probabilistic information about the engineering parameters of a space also exists in other areas of geotechnical engineering. Consider the paper by Wu and Wong (Wu and Wong 1981) that describes the case history of a problem in probabilistics soil exploration. In this case, the soil properties are measured at several locations. The probability contours are developed based on a proximity rule. For pavement design, it is usually not possible to have measured parameters since the pavement system is to be constructed in the future. Still such a proximity rule could be used particularly if short wavelengths are important. The basic concept would be to randomly select parameters at distances such that they can be considered mutually independent. These values can then be considered the same as measured values and a proximity rule used to develop parameters between the randomly determined values.

Prediction of Pavement Rutting

General

Two basic approaches were available for prediction of the pavement rutting - the statistical and mechanistic. Barber et al. (1978) conducted a study of numerous pavements and developed a statistical model for prediction of the rut depth as a function of CBR, thickness, and traffic. This particular model provides the rut depth as a deterministic value or a statistical value. If the rut depth is determined statistically, i.e. in terms of a mean and variance, then the rut depth can be distributed directly. Although Barber's model currently exists, for reasons of versitility and laboratory testing considerations a mechanistic model for prediction of rutting was developed.

Permanent Strain Model

The mechanistic model is based on a laboratory-determined relationship between the permanent deformation (ε_p) and the resilient deformation (ε_R) and state-of-stress along a vertical axis of the pavement system. The first work accomplished toward developing the mechanistic model was to review available laboratory data for the development of the relationship between ε_p and ε_R and to show the feasibility of using the approach for predicting rut depth. This study was rather lengthy and involved; therefore the results are presented separately in Appendix B. Although the methodology presented in Appendix B was a viable methodology, it did not lend itself to a computerized procedure.

From the data analyzed in Appendix B it was apparent that the relationship between ε_p and ε_R is influenced by the soil strength. To develop the relationship with soil strength the WES data were plotted as shown in Fig. 6. For each of the repetition levels a relationship is shown between the ratio of ε_p and ε_R and the resilient modulus (M_R). By considering the relationship shown in Fig. 6, the relationship that

$$\frac{\varepsilon_P}{\varepsilon_R} = .14 \left(\frac{70,000}{M_R}\right)^R$$

where

$R = 0.4 \text{ (Stress Reps)}^{0.12}$

$M_R = \frac{\sigma_d}{\varepsilon_R}$

σ_d = repeated deviator stress in laboratory triaxial test

ε_R = measured resilient strain in laboratory triaxial test

is developed.

As an independent check of the relationship, a comparison was made with experimental data reported by Ogawa (1972). This comparison is shown in Fig. 7.

Model for Resilient Modulus

For the purpose of this study a comparison with actual test data would be most desirable. One problem is that the strain model is based on M_R where the measured material property in the field test sections is the CBR. Making the conversion from CBR to M_R is rather difficult since the M_R is a function of stress as well as material properties. This conversion was made by developing an empirical correlation between field CBR and M_R. The correlation may be expressed by the equation

$$M_R = 10^x$$

where

$x = 4.5682 - \frac{1.9661}{8.55 \text{ CBR}} (6.5 + \sigma_d)$

σ_d = repeated deviator stress

The relationship is shown graphically in Fig. 8.

This correlation was developed from data obtained in connection with a study (Parker et al. 1979) to develop a correlation between plate bearing value and M_R. For the study a number of field sites were selected based on an attempt at having sites covering a wide range of soil types. At each site plate bearing and CBR tests were conducted and undisturbed soil samples were taken. Resilient modulus tests were conducted by the WES Geotechnical Laboratory. An example of the laboratory resilient modulus data and comparison with the model is given in Fig. 9. With the models now it is possible, provided the stress distribution is known, to make predictions of permanent deformation for various prototype pavement tests that have been conducted at WES.

Stress Model

When considering the stress distribution in flexible pavements, the structural layers above the subgrade are divided into bound and unbound materials. Bound materials are capable of substaining tensile stress and thus tend to distribute load as a slab; however, complete slab action is unlikely. Shrinkage and load associated cracking reduce the load-distributing characteristics of stabilized layers in pavement systems. In another study (the report to be published later) it was shown from data obtained in prototype testing of airfield pavements that bound layers give a load-distributing effect equivalent to a two-layered system having a modulus ratio (modulus of elasticity of the top layer divided by the modulus of elasticity of the bottom layer) between 3 and 4. This ratio will depend somewhat on the type material being stabilized and the amount of stabilization. For gravels that are well stabilized the ratio will approach 4; whereas for clays the ratio will be closer to 3. If there is very dense cracking in the stabilized material, the ratio could be lower but this is not likely to occur before complete failure of the pavement system.

Unbound structural layers, both crushed and uncrushed gravels, in a pavement system distributed the load equivalent to a two-layered system having a modular ratio of 1; that is to say the stresses will be distributed according to the Boussinesq stress model.

Since the procedure for predicting roughness requires the computation of stress many times, it is desirable to have a rapid means for performing these computations. For this a computer program was written using tabulated stress factors that are a function of the r/t ratio, r/z ratio, and the modulus ratio where r is the radius of the load area, t is the thickness of the structural layer, and z is depth to the point for computation of stress. An example of the stress distribution for the G-5A is given in Fig. 10.

Rutting Model

The rutting model consists of the permanent strain model and the stress distribution model. This model is illustrated in Fig. 11. There are some assumptions that must be included with the use of the model. The first assumption is that a stress repetition is synonymous with a coverage. The second assumption is that no permanent strain occurs in bound layers. For cement or lime-stabilized material, this has been found to essentially be true as long as the bound materials completely fail. For asphalt bound materials rutting can occur within these layers. This is particularly true if a poor design is used in the mix. This report presents no rutting model for use in predicting rutting in the asphalt bound materials. Fortunately most military airfields have well-designed asphalt bound layers that are fairly thin and thus the rutting within these layers will be negligible. The third assumption is that although the permanent strain model was developed based on testing of subgrade soils, it is assumed to be useable for granular materials. This assumption can be justified from the data presented in Appendix B.

The rutting model as presented in Fig. 11 provides a deterministic evaluation of the rut depth. If the model described for generation of a hypothetical pavement is used for generation of the material properties and structural layer thicknesses and a sufficient number of sections are generated, then the variations in rut depth can be simulated. This procedure, referred to as the Monte Carlo procedure, can be used to determine the probability of the rut exceeding a certain value or, if used in connection with an aircraft simulation program, the probability of the acceleration forces exceeding a certain value.

Prof. M. Harr introduced in a short course a procedure he referred to as Rosenblueth's procedure that could be used to compute the rut depth in statistical terms. The procedures uses a finite difference procedure where the dependent value of a function is computed at both the mean plus a standard deviation and the mean minus a standard deviation of the independent variable. These two values are used to compute the mean and deviation of the dependent variable. The procedure for a single independent variable is for

$$Y = f(X)$$

then $$\overline{Y} = 1/2\ (X_{+} + X_{-})$$

and $$S_{y} = 1/2\ (X_{+} + X_{-})$$

where

$\bar{Y}$ = mean of Y

S_y = standard deviation of Y

$Y_+ = F(\bar{X} + S_x)$

$Y_- = F(\bar{X} + S_x)$

$\bar{X}$ = mean of X

S_x = standard deviation of X

The variance, V_y, of Y can be computed by

$$V_y = \frac{S_y}{\bar{Y}} (100)$$

This procedure can be extended to multiple independent variables by considering the functional value for values of the mean plus or minus a standard deviation for each of the independent variables, i.e.,

$$\tilde{\mu}_1 = \frac{1}{2}\left(X_+^1 + X_-^1\right)$$

$$\tilde{S}_1 = \frac{1}{2}\left(X_+^1 + X_-^1\right)$$

$$\tilde{\mu}_1 = \frac{1}{2}\left(X_+^2 + X_-^2\right)$$

$$\tilde{S}_1 = \frac{1}{2}\left(X_+^2 - X_-^2\right)$$

where

X_+^1 = the mean plus a deviation for variable 1

X_-^1 = the mean minus a deviation for variable 1

X_+^2 = the mean plus a deviation for variable 2

X_-^2 = the mean minus a deviation for variable 2

$\tilde{\mu}$ = a mean of the function with respect to variable 1 with all other variables at the mean

$\tilde{S}_1$ = a standard deviation with respect to variable 2 with all other variables at the mean

$\tilde{\mu}_2$ = same as $\tilde{\mu}_1$ but with respect to variable 2

$\tilde{S}_2$ = same as $\tilde{S}_2$ but with respect to variable 2

Also the value of the function at the mean of all variables is used and is computed by

$$\mu^* = f\left(X^1, X^2\right)$$

where

X^1 = the mean of variable 1

X^2 = the mean of variable 2

Now the equation

$$\frac{\mu}{\mu^*} = \frac{\tilde{\mu}_1}{\mu^*} \cdot \frac{\tilde{\mu}_2}{\mu^*}$$

can be used to compute the mean, μ, of the function. For variance the equations

$$\tilde{V}_1 = \frac{\tilde{S}_1}{\tilde{\mu}_1}$$

$$\tilde{V}_2 = \frac{\tilde{S}_2}{\tilde{\mu}_2}$$

are used to compute the variance, $\tilde{V}_1$ and $\tilde{V}_2$, with respect to each of the variables. Now the equation

$$1 - V^2 = (1 + \tilde{V}_1^{\,2})\ (1 + \tilde{V}_2^{\,2})$$

can be used to compute the variance of the function with respect to all variables. The Rosenblueth procedure was combined with the rutting model and computerized to give a computer program for computing the rut depth in statistical terms. The listing of this program is given in Appendix C. With the rut depth computed in statistical terms, the rut could be distributed directly to produce the runway profile.

Other Considerations

A methodology has been presented for generating a new airfield pavement profile and material properties that meet given statistical data. For flexible airfield pavements these data can be used to predict the development of permanent deformation with traffic and thus construct a prediction of the profile with traffic. Using the profile as input in an aircraft simulation program, a measure of roughness is obtained. The methodology developed thus far is rather crude but does illustrate the feasibility of the approach and provides basic models necessary for future development. As mentioned earlier, a pressing need is for the development of the proximity rules to be used in generating the initial pavement sections. Verification of the proximity rules should be accomplished by comparison with actual measured airfield data.

Also a factor that could be considered is the development of structural cracking. Models are currently available for both rigid and flexible pavements for the prediction of structural cracking, but the effect of the cracking on the roughness of a pavement is a relatively unknown quantity.

Due to the interaction of the aircraft and runway surface the loading to the pavement is not necessarily equal to the stated loading of the aircraft. The consideration of dynamic load in the procedure would add an order of magnitude in the solution process. Such loadings are a function of both profile and speed,and thus consideration of the determination of the dynamic loading would require continuous updating as the profile changed.

The profile could also be affected by climatic conditions, particularly in frost areas. For pavements that are not designed for full frost protection the frost heave is likely to be very critical. The weakening of the subgrade during spring thaw is another factor that should be considered as the permanent deformation that occurs during this period of time may be greater than the deformation occurring during the remainder of the year.

For rigid pavements the joints pose a special problem. There is some statistical chance of dowel failures, spalling, pumping, or other joint problems that could create a rough pavement. For rigid pavements, more so than flexible pavements, research aimed at solving the problems of predicting pavement roughness is almost nonexistent.

Summary

A simplified procedure has been developed for considering roughness in pavement design. The procedure used the variation in pavement section to generate a stochastic airfield pavement. A model was developed for predicting the permanent surface deformation with aircraft traffic. Thus the predicted deformation can be added to the initial profile to yield a pavement surface at any point in time. The predicted pavement section can be tested for roughness by the use of an aircraft simulation computer program.

Extension of this simplified procedure to provide consideration of some of the other factors that affect pavement roughness would require a large research effort. Although the research effort for complete development of the methodology is large, it is a project that should be undertaken. The return on such an effort in terms of better pavement design and pavement management data would justify the monies spent.

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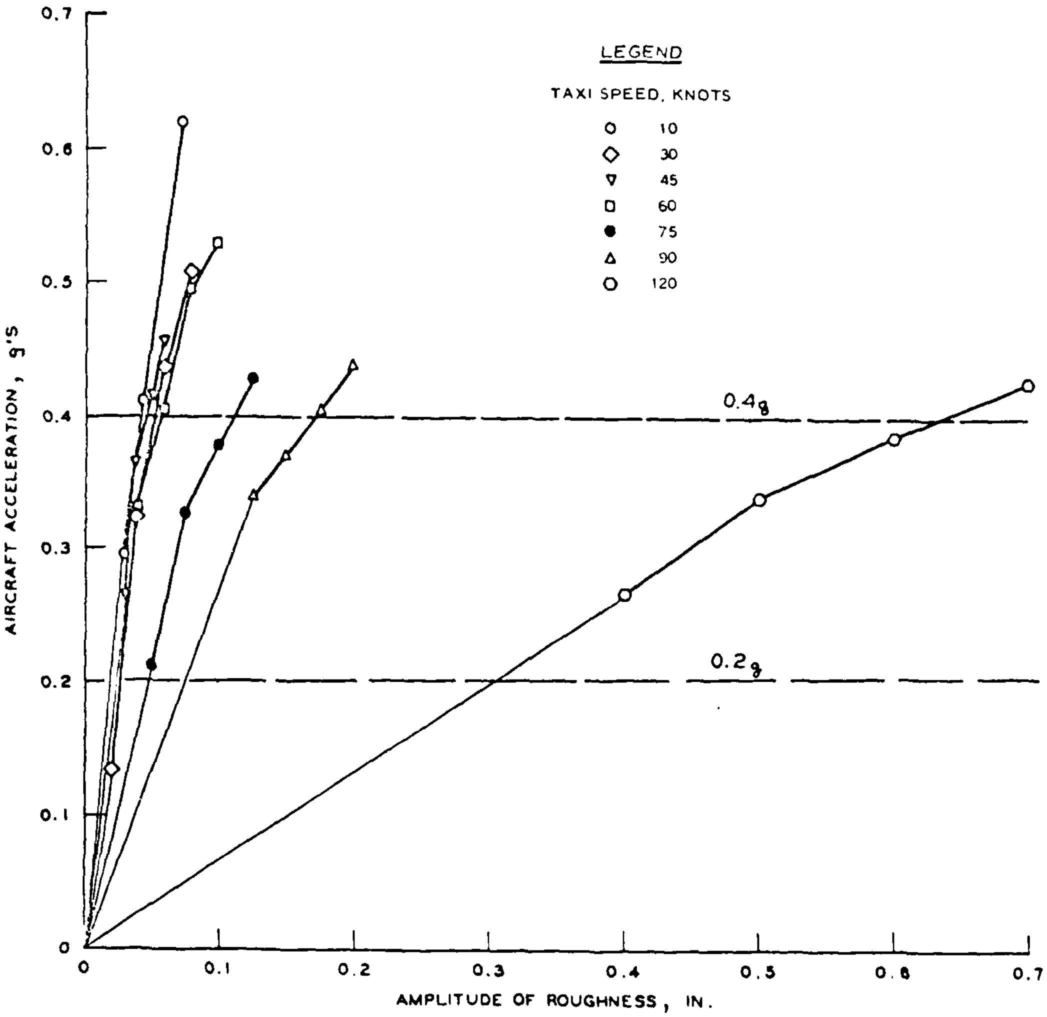


Figure 1. Acceleration forces for hypothetical sinusoidal profile; B-52H (after Horn 1977)

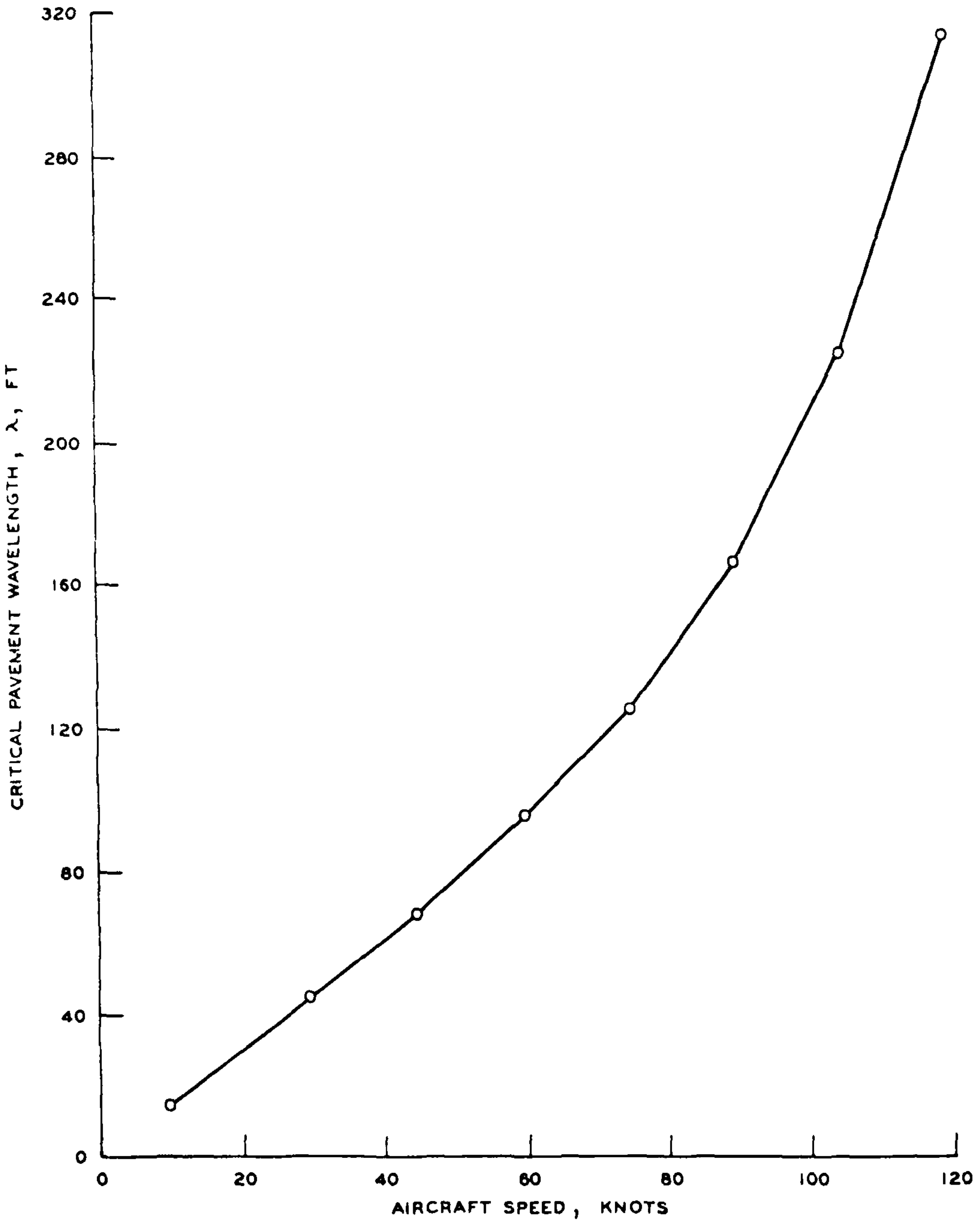


Figure 2. Critical pavement wavelength as a function of aircraft speed (after Horn 1977)

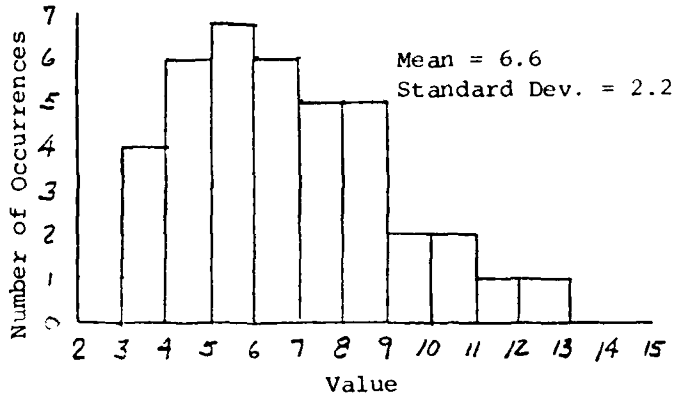
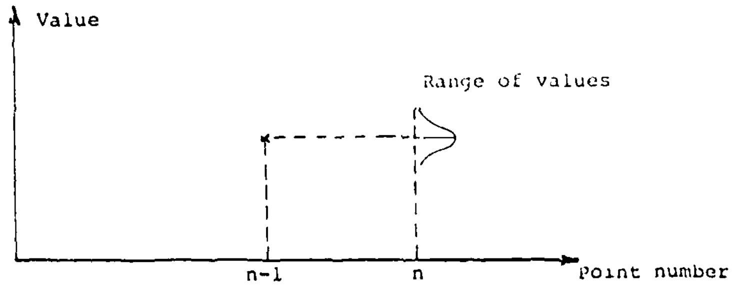
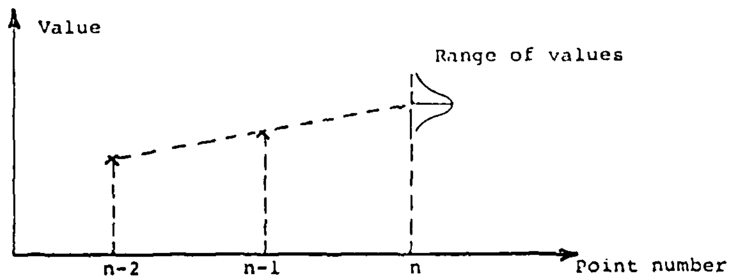


Figure 3. Histogram of values for Beta distribution



Random walk



Modified random walk

Figure 4. Illustration of random walk and modified random walk (after Ullidtz 1978)

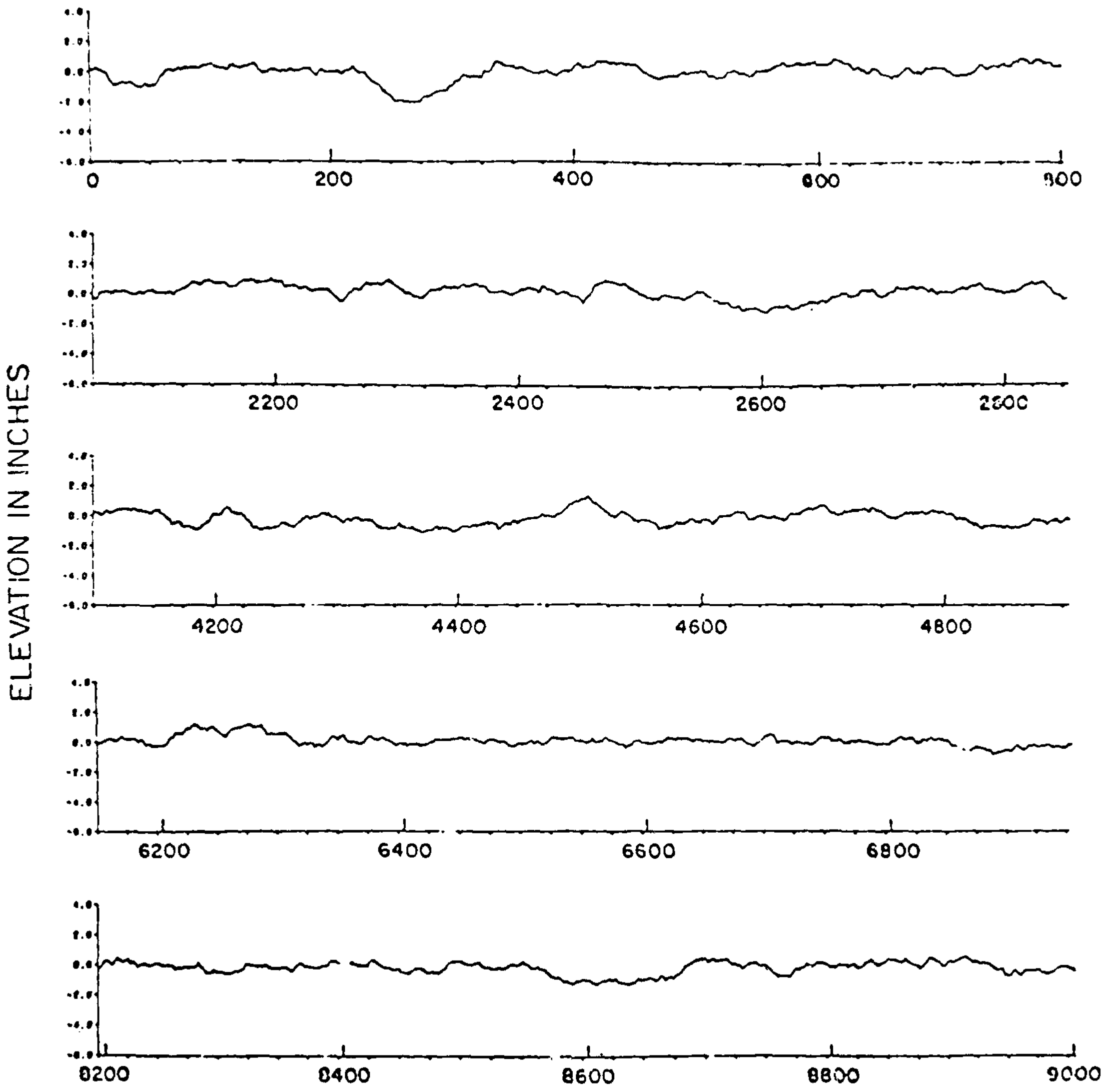
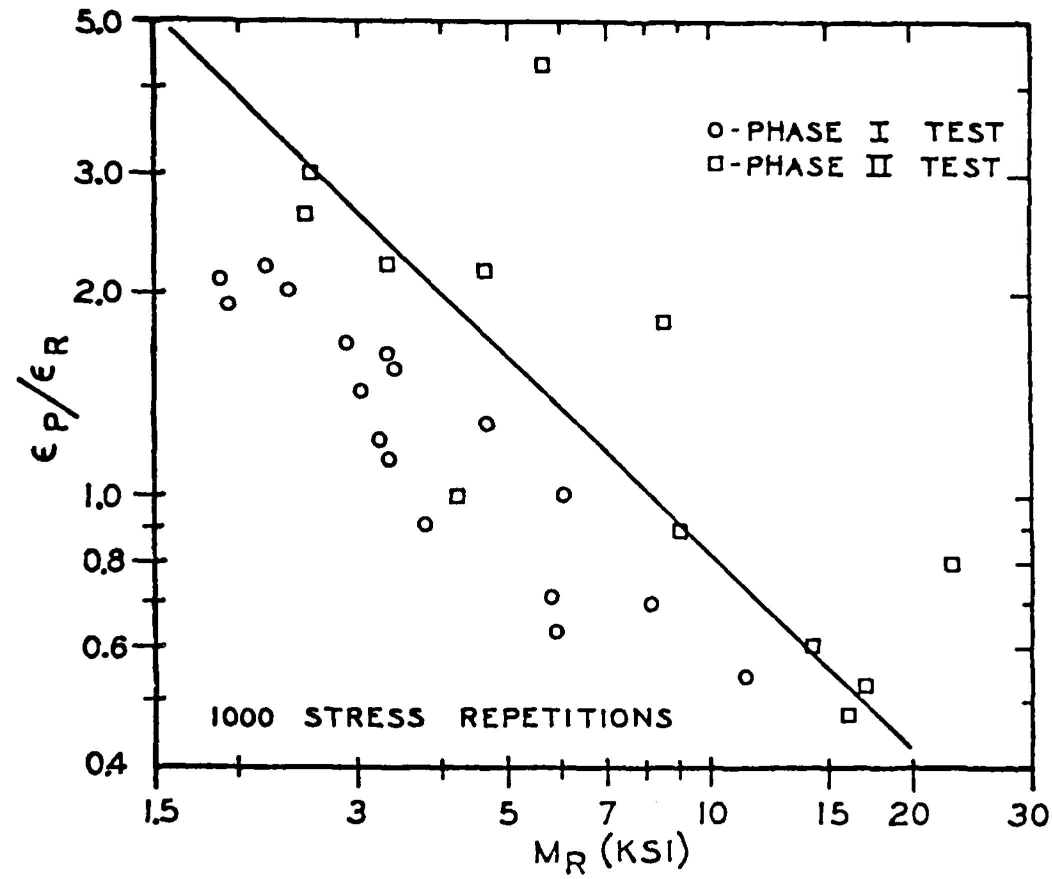
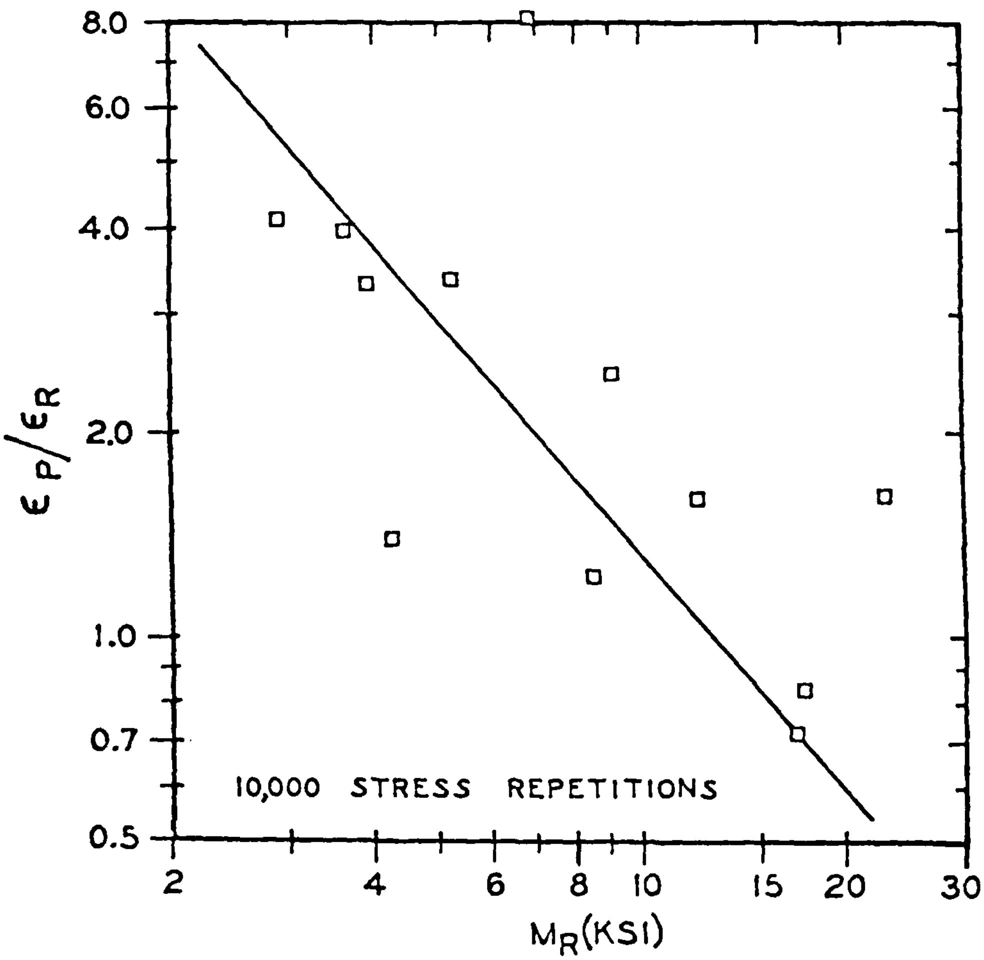


Figure 5. Illustration of generation of runway profile using power spectral density (after Berens and Newman 1973)



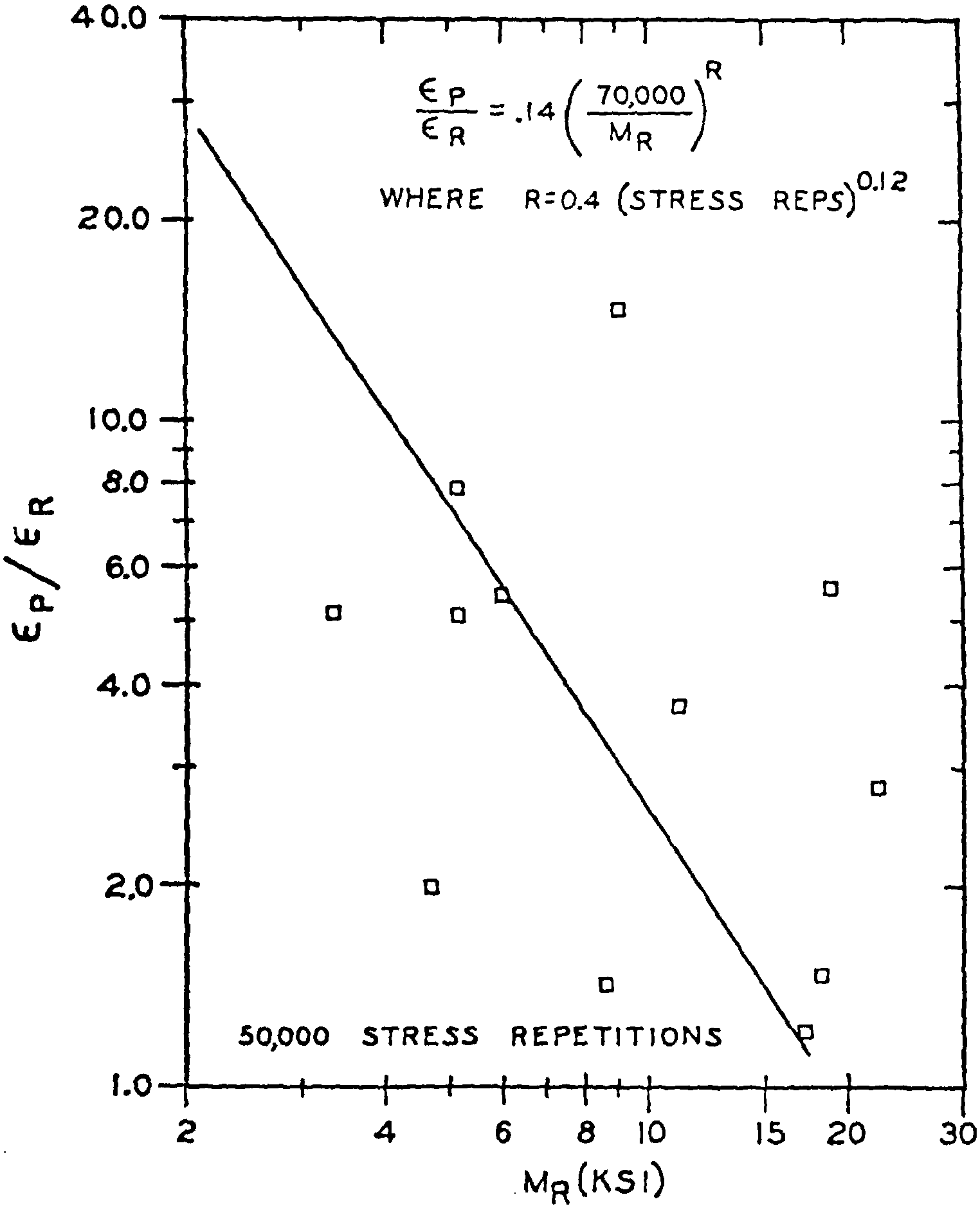
a. 1,000 stress repetitions

Figure 6. Relationship between $\varepsilon_P/\varepsilon_R$ and M_R (Sheet 1 of 3)



b. 10,000 stress repetitions

Figure 6. (Sheet 2 of 3)



c. 50,000 stress repetitions

Figure 6. (Sheet 3 of 3)

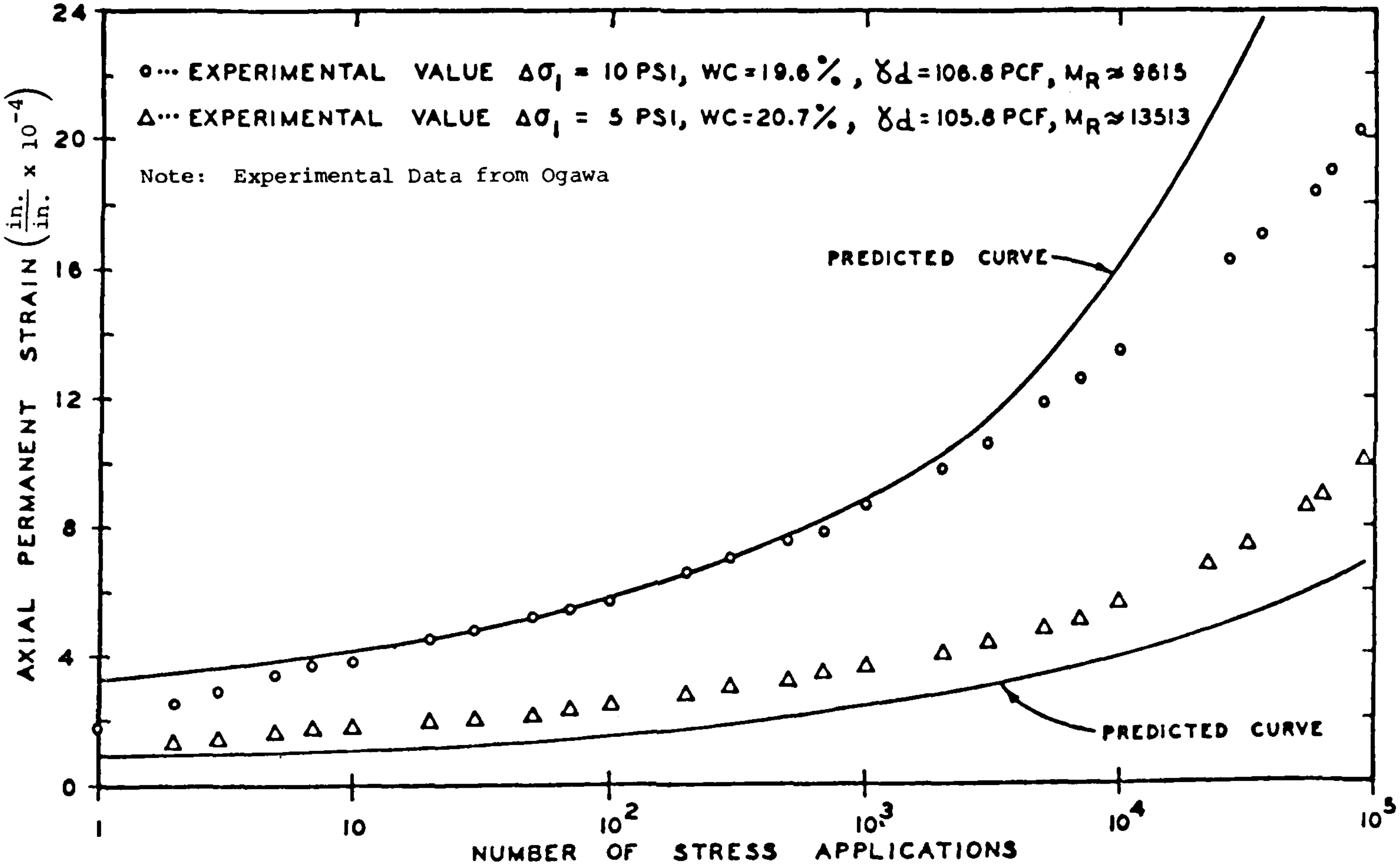


Figure 7. Comparison of permanent strain model with test data

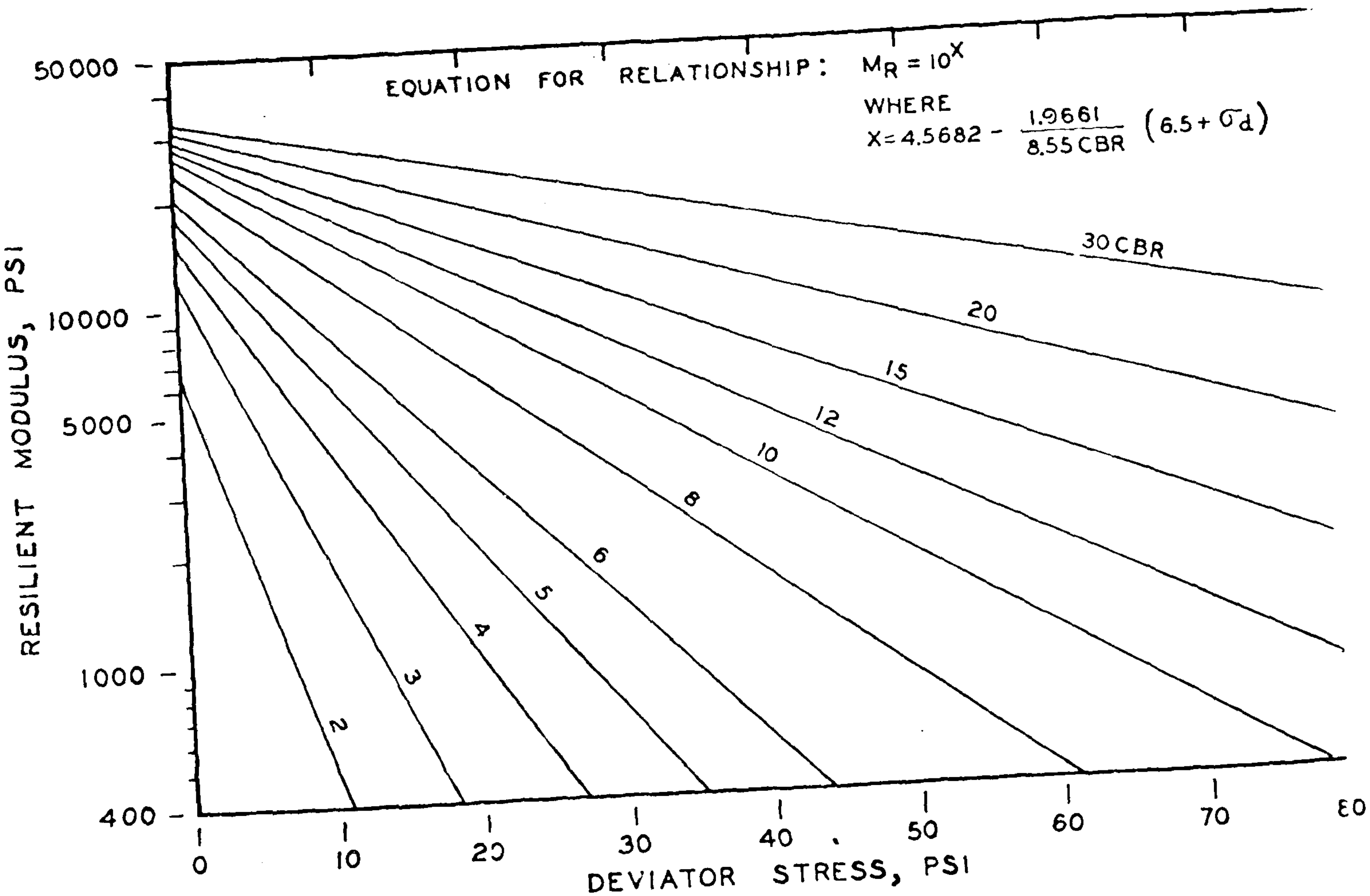


Figure 8. Relationship for predicting M_R from CBR and stress

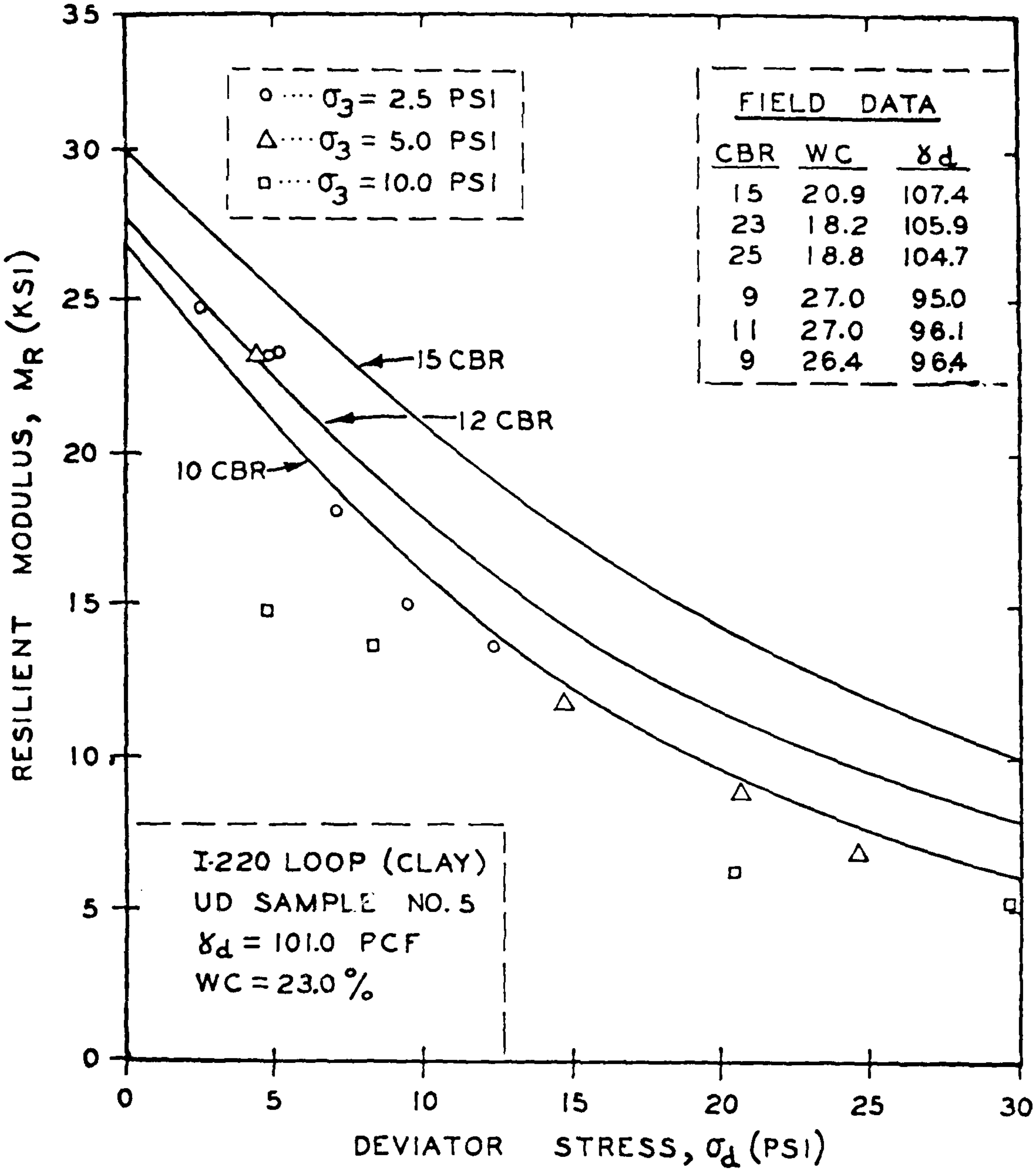


Figure 9. Example resilient modulus data and comparison with model

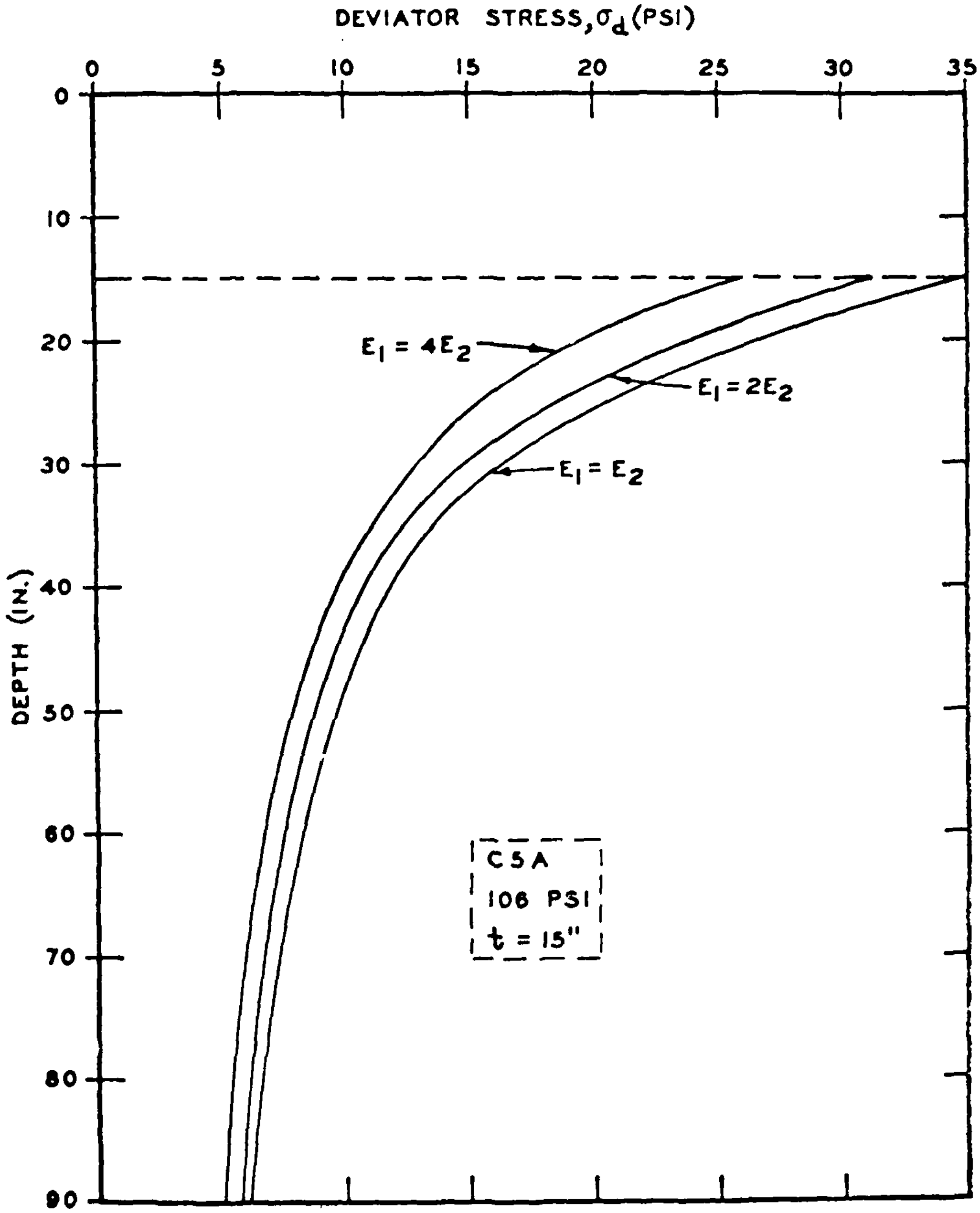
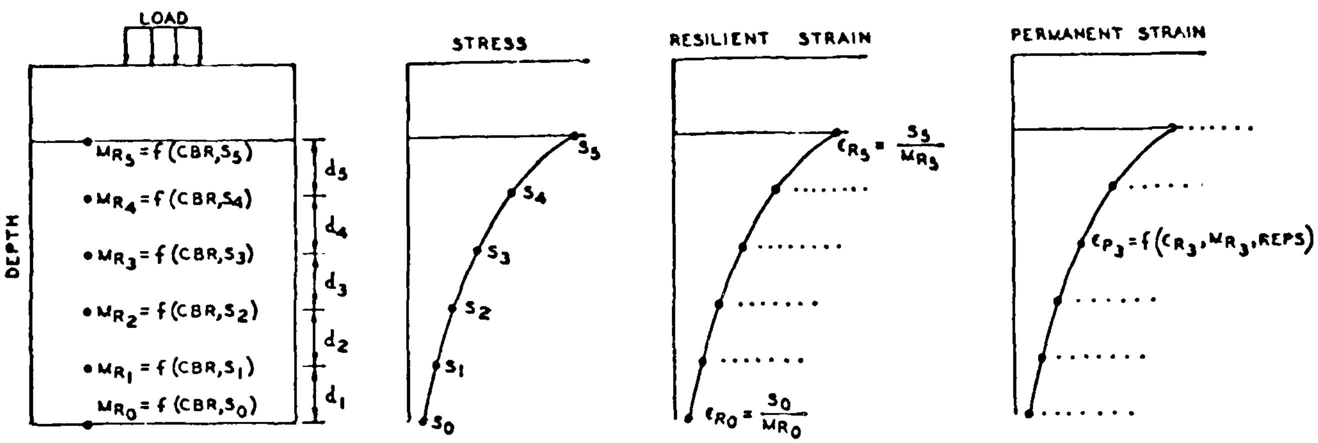
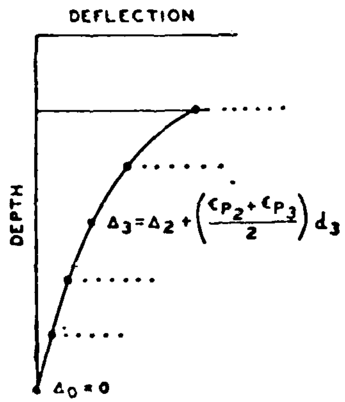


Figure 10. Stress distribution for C-5A for 15-in. structural layer

PROCEDURE FOR COMPUTING PERMANENT DEFORMATION OF SUBGRADE





THE FUNCTIONS

$M_{R_i} = 10^X$

WHERE $X = 4.5682 - \frac{1.9681}{8.55(CBR)}(8.5 + S_i)$

$\epsilon_{P_i} = \epsilon_{R_i}\left[.14\left(\frac{70000}{M_{R_i}}\right)^Y\right]$

WHERE $Y = 0.4(\text{STRESS REPS})^{0.12}$

Figure 11. Illustration of rutting model

Appendix A: Program BETADIS

Listing of Program BETADIS

```
      CHARACTER C*8
      DIMENSION KOUNT(20)
     DIMENSION XX(1000)
C     INPUT NUMBER OF NUMBERS TO GENERATED
      PRINT,' INPUT NUMBER OF NUMBERS TO BE GENERATED'
      READ,NO
     PRINT,' INPUT MEAN, STANDARD DEV., XMIN, XMAX'
      READ, XMEAN,STAN,XMIN,XMAX
C     FIRST COMPUTE ALPHA AND BETA
      R = XMAX-XMIN
      XM = (XMEAN-XMIN)/R
      VM = (STAN/R)**2
      A = (XM**2)*(1-XM)/VM-(1+XM)
      B = (A+1)/XM -(A+2)
      PRINT 901,A,B
  901 FORMAT(2X,'ALPHA =',F10.3,'   BETA=',F10.3)
      CALL DATIM(C,D)
      D = D *.01
      Y = RAND(-D)
      PRINT,'      RAND NUMBER          PARTIAL BETA        BETA NUMB
      SUM = 0.0
      SUM2 = 0.0
      TEMP = 0.0
      TEMP2 = 0.0
      DO 50 K = 1,10
      KOUNT(K) = 0
   50 CONTINUE
      KTOTAL = NO / 10
      I = KTOTAL * 10
      IF(I.LT.NO) KTOTAL = KTOTAL + 1
      DO 100 I = 1,NO
   60 CONTINUE
      Y = RAND(0.0)
      K = Y * 10.
      K = K + 1
      IF (KOUNT(K).GE.KTOTAL) GO TO 60
      KOUNT(K) = KOUNT(K) + 1
      TEMP = TEMP + Y
      TEMP2 = TEMP2 + Y**2
      X = BETA(1,Y,A,B)
      XX = XMIN + X*R
      PRINT 900,Y,X,XX
      SUM = SUM + XX
      SUM2 = SUM2 + XX ** 2
      XX(I) = XX
  900 FORMAT(2X,3F15.6)
  100 CONTINUE
 AVG1 = TEMP/NO
        DEV1=SQRT((TEMP2-(TEMP**2)/NO)/(NO))
      AVG = SUM/NO
```

```
      DEV = ((SUM2-(SUM**2)/ND)/ND)
      DEV = SQRT(DEV)
      PRINT 902,AVG,DEV
      SUM = 0.0
      SUM2 = 0.0
      DO 200 I = 1,ND
     XXX(I) = XMEAN + ((XXX(I)-AVG)*STAN)/DEV
      IF(XXX(I).GT.XMAX) XXX(I) = XMAX
      IF(XXX(I).LT.XMIN) XXX(I) = XMIN
      SUM = SUM + XXX(I)
      SUM2 = SUM2 + XXX(I)**2
  200 CONTINUE
      AVG = SUM/ND
      DEV = (SUM2-(SUM**2)/ND)/ND
      DEV = SQRT(DEV)
      PRINT 903,(XXX(I),I=1,ND)
      PRINT 902,AVG, DEV
  902 FORMAT(5X,'MEAN =',F15.5,'   STANDARD DEVIATION=',F15.5)
  903 FORMAT(6F12.3)
 PRINT 902,AVG1,DEV1
      STOP
      END


C     RANDX -- UNIFORM RANDOM NUMBER GENERATOR
C     OCT 1970
      FUNCTION RAND(X)
      IF(X) 10,20,20
   20 RN=RHO*RAND
      RN1=AMOD(RN,BN)
      RAND=RN1/BN
      RETURN
   10 RHO=7.0**13
      BN=10.0**10
      RAND=-X
      GO TO 20
      END
```

```
C    BETA
* *****INCOMPLETE BETA FUNCTION APPROXIMATION*****
          FUNCTION BETA(IX,BBB,XX,YY)
          ISIGN=IX
          BB=BBB
          X=XX
          Y=YY
          IF(ISIGN-1)100,200,200
  100     C1=GBETA(X,Y,0.,BB)
           C2=GBETA(X,Y,0.,1.)
           BETA=C1/C2
           RETURN
      200 IF(X-1.)232,225,232
      225 IF(Y-1.)229,228,229
      232 IF(Y-1.)237,233,237
  237      BET=GBETA(X,Y,0.,1.)
           BETA=X/(X+Y)
           DO 250 I=1,5
           C1=GBETA(X,Y,0.,BETA)
           C2=ETION(X,Y,BETA)
           BETA=BETA+(BET*BB-C1)/C2
           IF(BETA-1.)10,40,40
        10 IF(BETA)20,20,250
        20 BETA =0.
        30 GO TO 300
        40 BETA=1.
           GO TO 300
      250 CONTINUE
           GO TO 300
      228 BETA=BB
           GO TO 300
      229 BETA=1.-(1.-BB)**(1./Y)
           GO TO 300
      233 BETA=BB**(1./X)
           GO TO 300
      300 RETURN
           END
           FUNCTION GBETA(X,Y,A,B)
           DIMENSION U(8),H(8)
           DATA U/.04750625,.14080178,.22900839,.30893812,
           .37770220,.43281560,.47228751,.49470047/
           DATA H/.09472531,.09130171,.08457826,.07479799,
           .06231449,.04757926,.03112676,.01357623/
           SUM=0.0
           DO 20 I=1,8
           ARG1=((B-A)*U(I)+(B+A)/2.)
           ARG2=(A-B)*U(I)+(B+A)/2.
           U1=ETION(X,Y,ARG1)
           U2=ETION(X,Y,ARG2)
        20 SUM=SUM+H(I)*(U1+U2)
           GBETA=(B-A)*SUM
           RETURN
           END
```

```
      FUNCTION ETION(U,V,Z)
      UU= -1.
      IF(Z-1.E-37)100,100,20
   20 IF(UU*ALOG(Z)-80.)30,200,200
   30 IF(UU*ALOG(Z)+80.)300,300,40
   40 CA=UU*ALOG(Z)
      A=Z**UU
      GO TO 50
  100 Z=1.E-37
      GO TO 40
  200 A=1.E37
      GO TO 50
  300 A=0.
   50 VV=V-1.
      Q=1.-Z
      IF(Q-1.E-37)400,400,60
   60 IF(VV*ALOG(Q)-80.)70,500,500
   70 IF(VV*ALOG(Q)+80.)600,600,80
   80 CA=VV*ALOG(Q)
      B=Q**VV
   90 IF(A)82,95,82
   82 IF(B)85,95,85
   85 IF((ALOG(ABS(A))+ALOG(ABS(B))).GE.(-80.))GO TO 95
      ETION=1.E-36
      GO TO 96
   95 ETION=A*B
   96 RETURN
  400 Q=1.E-37
      GO TO 80
  500 B=1.E37
      GO TO 90
  600 B=0.
      GO TO 90
      END
```

List of Numbers Having Beta Distribution
Obtained from Program BETADIS

6.990	7.970	12.081	6.575	4.092	4.002
11.527	5.810	10.030	8.418	6.230	6.926
7.261	3.965	10.229	9.701	6.806	6.129
5.911	3.787	7.597	4.393	5.418	4.947
5.145	8.675	8.030	3.653	4.360	3.889
7.016	9.378	5.388	4.659	7.681	7.325
5.264	5.327	4.810	8.705		

Mean = 6.7; Standard Deviation = 2.2

Appendix B: Development of a Basic Rutting Model

INTRODUCTION

The Waterways Experiment Station (WES), in seeking a more rational approach to design of flexible pavements than the presently used CBR design procedure, has developed, under sponsorship of the Federal Aviation Administration (FAA) and the Office, Chief of Engineers (OCE), subgrade strain criteria for both roads (Brabston, et al.[1]) and airports (Barker and Brabston[2]). The criteria now being incorporated into both FAA and OCE design procedures are considered to be the basis for limiting the rutting of flexible pavement. The approach of limiting the resilient strain in the subgrade in order to limit rutting of the subgrade implies that a relationship exists between the permanent strain and the resilient strain of subgrade soils. In both of the design procedures the limiting subgrade strain criteria were presented as a function of the subgrade modulus. The author[2] discussed this point in detail and presented the results of laboratory tests which indicated that the relationship between the permanent strain and resilient strain was indeed a function of the stiffness of the material. The work in developing these strain criteria and in studying the results of laboratory tests for conformation of the criteria created an interest in the relationship between the permanent strain and resilient strain of subgrade soils.

SUBGRADE STRAIN CRITERIA

A comparison of the different subgrade criteria is presented in Figure 1. All of the criteria presented with the exception of that developed by Dr. Chou (Reference 3) has been developed from pavement sections conforming to some

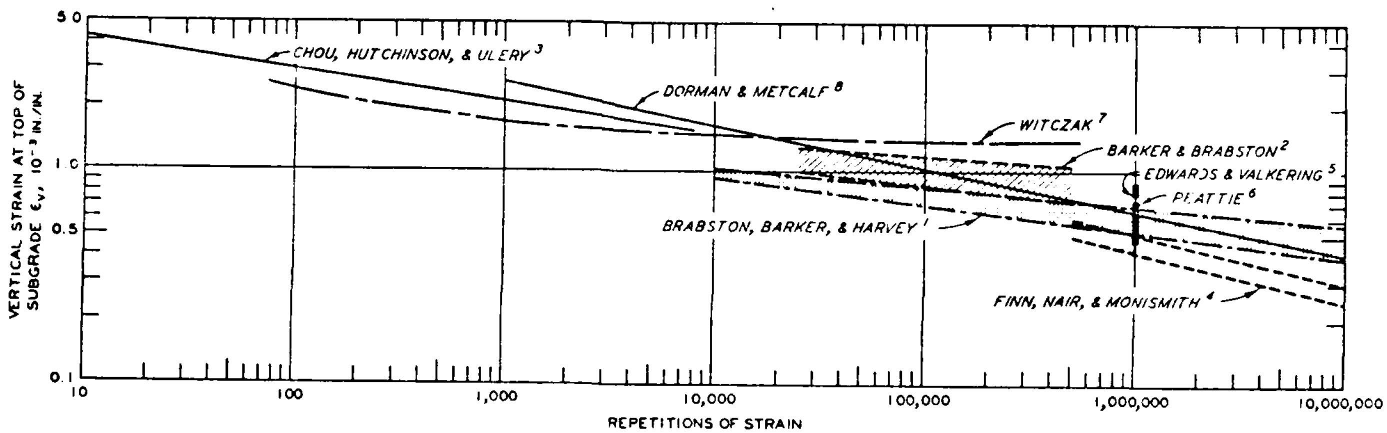


Figure B1. Comparisons of subgrade strain criteria previously established design standards. Although different design standards may have been used and different methods were employed in characterizing the granular materials, it is seen when grouped that all of the criteria form a relatively narrow band across a wide range of repetition levels. Consider that for repetition level of 1×10^5 repetitions the range of the criteria is from 0.45×10^{-3} to 1.4×10^{-3} in./in. Even if the criteria of Finn, et al.,[4] and Chou were extrapolated to this level of repetition, the range would not be increased. From the evidence it would appear that for a given level of repetitions the resilient strain at which the permanent strain becomes unacceptable would be within a fairly narrow band. The data presented in Figure 1 strongly indicate a unique relationship between permanent strain and resilient strain. Considering the emphasis being placed on the use of the repeated load triaxial test in which both the resilient and permanent strains are measured, it would seem that the concept of limiting subgrade strain criteria could be substantiated or disputed from the results of such laboratory tests.

LABORATORY TEST

In recent years much interest has been generated principally by the work of Barksdale[9] in the use of the repeated load triaxial test as a method to predict rutting of a pavement. The aim of nearly all of the laboratory rutting tests performed to date has been: first, to define the permanent deformation as a function of the applied stress; and second, to define the resilient modulus of the material as a function of stress. Only one experiment known to the author has been performed with the first objective of defining the relationship between resilient strain and permanent strain. This experiment reported by Chisolm[10] was conducted to substantiate the concept by the author that the allowable resilient subgrade strain is a function of modulus of the subgrade; i.e., to establish the relationship between resilient strain and permanent strain for soils having different moduli. To accomplish this objective, a Vicksburg heavy (buckshot) clay (CH and E-11) was molded at four different water contents and tested in a repeated load triaxial test. Since the object of the experiment was to determine only the relative relationships between resilient strain and permanent strain between the samples, only 1000 load repetitions were applied to each sample. The results of the experiment are shown in Figure 2. Another experiment in which the establishment of the relationships between resilient and permanent strain was only a minor part was conducted at the University of California at Berkeley by Ogawa (Reference 11). The results of this experiment in regard to the relationship between resilient strain and permanent strain are shown in Figure 3. The results of these tests are close to those of WES, even though the soils are different and the tests were conducted at different strain levels and a different number of strain repetitions were applied. Both Chisolm and Ogawa used almost identical equipment and procedures in conducting the two experiments. One seemingly important factor is that both used LVDT holding clamps placed on the specimen (a description of the measuring device is given in References 10 and 11) for measuring both resilient and permanent strains. When examining the results of other researchers,the evidence indicates a distinct difference in the test results depending on how the strains are measured.

The majority of research reports studied were written primarily to present the permanent or the resilient deformation characteristics of a material but not the relationship between the two. Thus, in order to develop these data, in many cases it was necessary to calculate missing parameters which, in most cases, was the resilient strain. The data studied were that reported by Barksdale,[9] Kalcheff and Hicks,[12] Fossberg,[13] Seed and McNeill,[14] Brown et al.,[15]

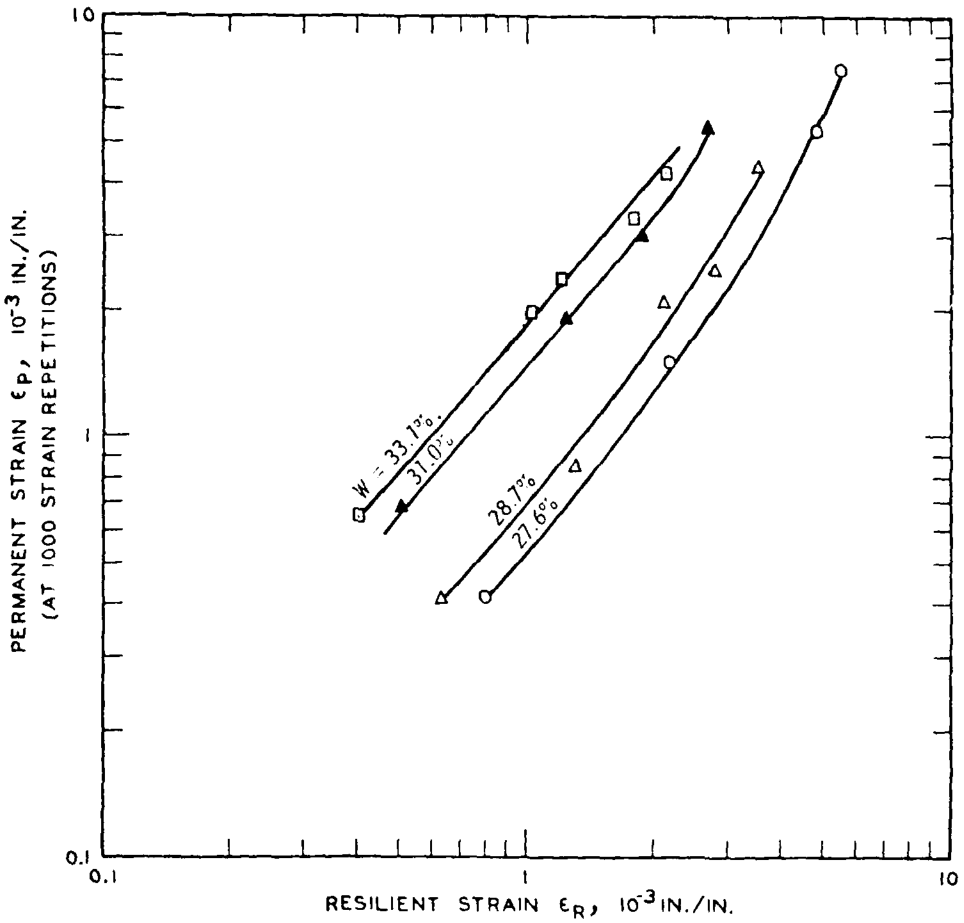


Figure B2. Relationships between permanent strain and resilient strain for buckshot clay

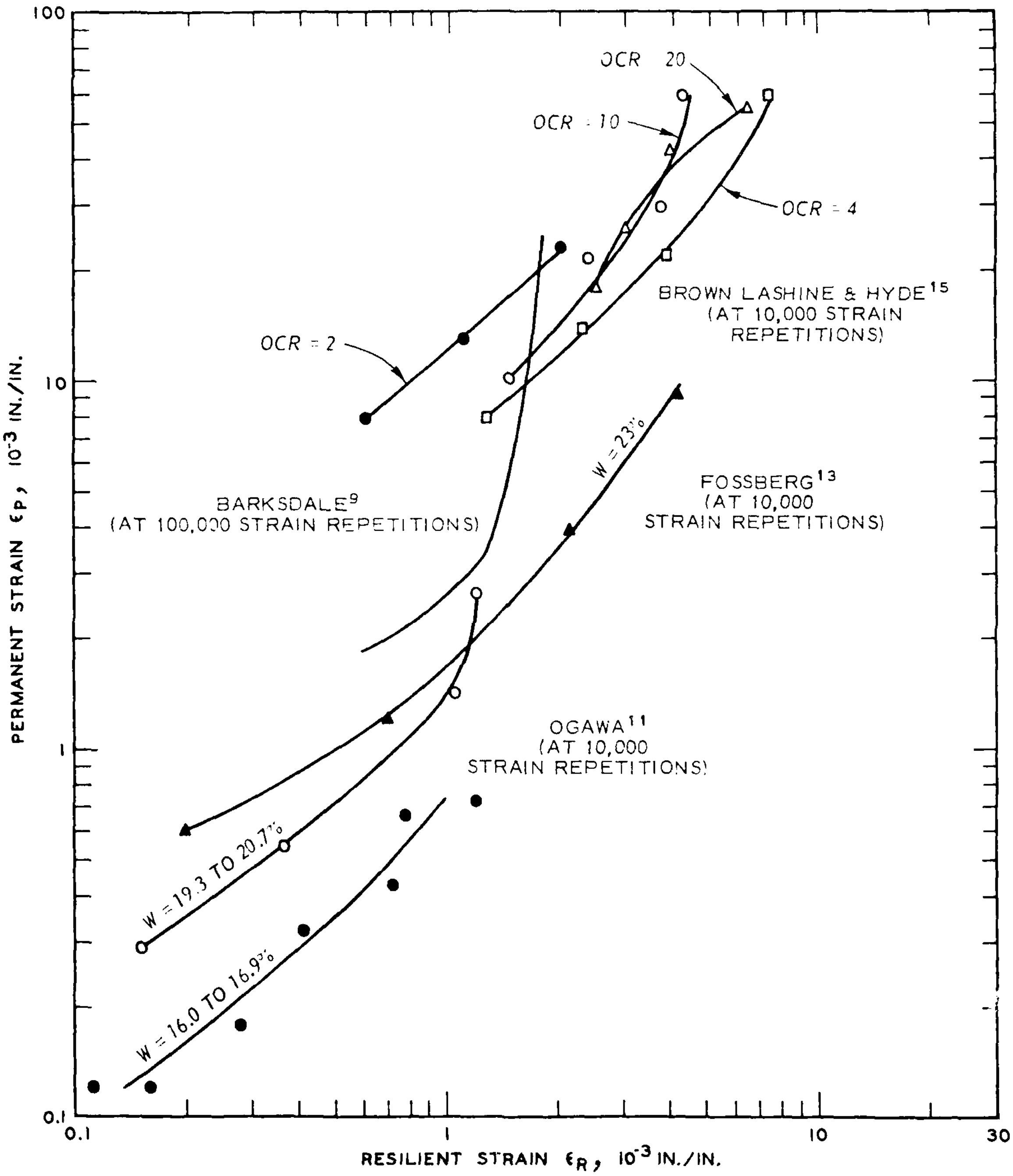


Figure B3. Relationship between permanent strain and resilient strain for different materials

Kirwan et al.,[illegible] and Chisolm.[17]

The experiment involving the most types of soil for which the relationships could be established was that conducted by Kirwan. The soils used in the experiment are listed in Table 1. This experiment was not conducted in a manner which would yield data compatible with the objective of the Chisolm experiment. Basically the tests were conducted at two stress levels for each soil strength. In each soil, different water contents were employed in order to obtain a wide range of permanent deformations. This procedure is in contrast to Chisolm's procedure in which the stress was varied to obtain a range of resilient strains. Thus, for a given soil strength, only two data points were available to obtain a relationship between the resilient and permanent strains for a given soil strength; therefore, the data were combined as shown in Figure 4. When presented in this manner, it appears that the data forms one relationship for all the soils except soil No. 5. It should be remembered that the test points for the higher strains were obtained by increasing the water content and would therefore represent data from samples of lower strength. If the soils in this experiment behave similarly to those tested by Chisolm and Ogawa, the permanent strain for a particular resilient strain would be higher for the soil at the higher water content but lower stress than for the soil at the lower water content but higher stress. The effect of the varying of the water content to obtain higher resilient strains would be to produce a steeper relationship than would have been obtained by maintaining a constant water content and increasing the applied stress to obtain higher resilient strains.

Another interesting but difficult set of data to interpret was the data presented in Reference 15. In this experiment, the effect of stress history of the samples on permanent deformation and resilient strain were

TABLE B1

(From Reference 16)

Soils Used in the Investigation

Clay No.	Soil Type	Location	Liquid Limit	Plastic Limit	Plasticity Index
1	CL	Lucan	29 percent	22 percent	7
2	CL	Wexford	30 percent	16 percent	14
3	CH	Coal Island	67 percent	25 percent	42
4	CH	Dublin	35 percent	19 percent	16
5	CH	Devon (TA)	63 percent	23 percent	40
6	CH	Darndale (A)	52 percent	27 percent	25
7	CL	Darndale (E)	46 percent	27 percent	19
8	CL	Darndale (F)	39 percent	22 percent	17
9	CL	Darndale (G)	33 percent	18 percent	15

Soil type refers to the Unified Soil Classification System.

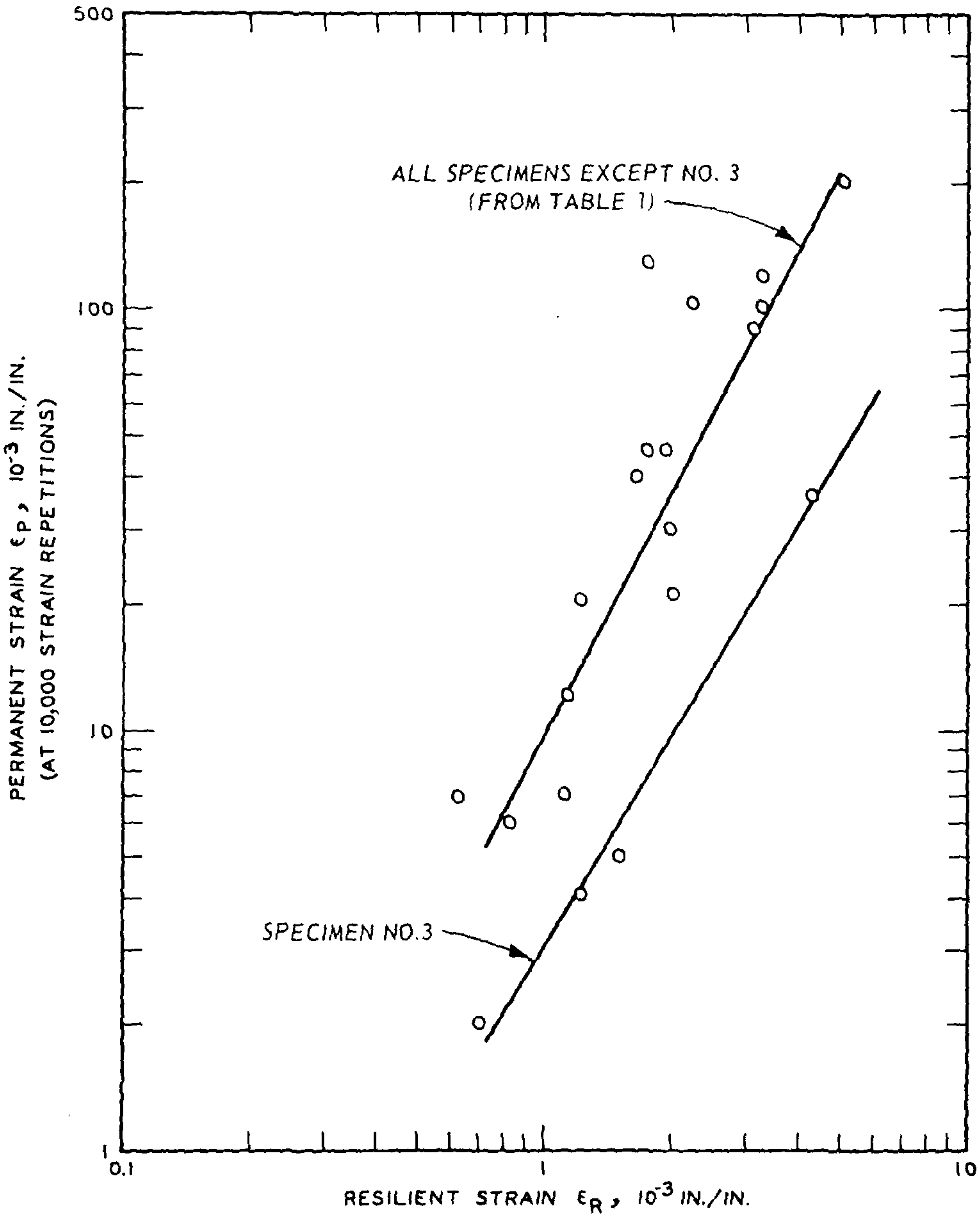


Figure B4. Relationships between permanent st··in and resilient from data devel·ped by ·irwan et al.[16]

studied. Although it was concluded in the paper that stress history had an effect on the permanent strain but no effect on resilient strain, the data as plotted in Figure 3 indicated the relationship between permanent strain and resilient strain is little affected by moderate differences in stress history. In the plot, a distinct relationship for each overconsolidation ratio is presented but the difference appears inconsistent and could be experimental error. A single relationship would probably suffice for this set of data. The data presented by Barksdale (Figure 3), Seed (Figure 5), and Fossberg (Figure 3) are all straightforward, each of which provided useful information. Of all the tests, those conducted by Fossberg covered the largest range (.0002 to .0042 in./in.) of resilient strains. Two tests, one by Barksdale and one by Ogawa, indicated very abrupt change in the slope of the relationship. For the test by Barksdale the abrupt change came at a resilient strain of .0013 in./in. and for Ogawa at .0011 in./in. A more gradual change at about the same magnitude of resilient strain was indicated in the results presented by Fossberg. Such behavior lends strong support to the use of the limiting subgrade strain concept in pavement design. It can also be pointed out that the strain criteria developed to date (presented in Figure 1) are consistent with the laboratory results.

In examining the data plots, it was noted that results of Chisolm, Fossberg, and Ogawa group together and the data of the other researchers fall into another group. It has already beem emphasized that Chisolm and Ogawa employed inside LVDT holding clamps attached to the specimen to measure both resilient and permanent strains. Fossberg also used the same devices for the tests he conducted. Barksdale found that such measuring devices gave him inconsistent readings of permanent deformation and therefore he employed outside measuring devices for measuring permanent strain but still used LVDT

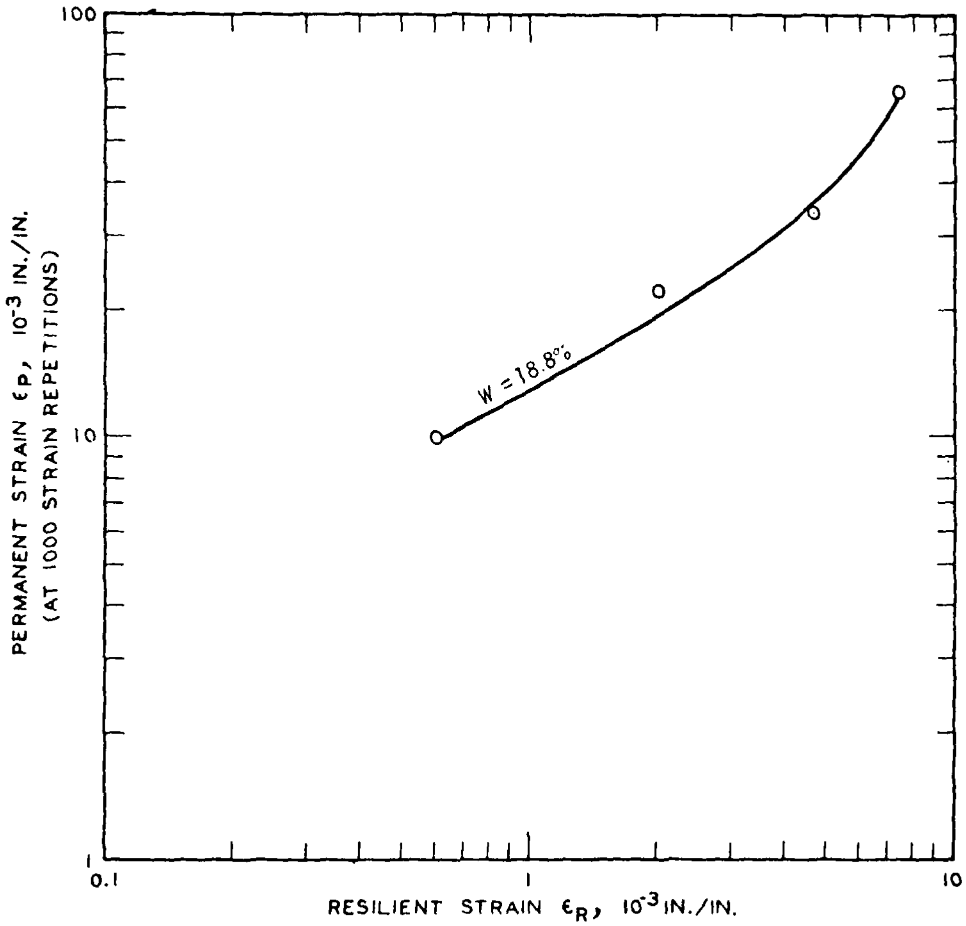


Figure B5. Relationship between permanent strain and resilient from data presented in Reference 14 clamps for measuring resilient strains. It is believed that the other researchers all used outside measuring devices which measured the deflection over the entire sample. Thus, the data fall into two groups; that in which the permanent strain was measured using LVDT clamps and thus only over the center portion of the sample, and that in which the permanent strain was measured using outside devices and thus measured the strain over the entire sample. The difference in the two groups of data is almost an order of magnitude.

Additional data are available by including tests on granular materials. Such data have been presented by Chisolm, Barksdale, and Kalcheff and Hicks. The plot of these data is shown in Figure 6. These data indicate a steeper relationship between resilient strain and permanent strain than was indicated for subgrade soils. Considering the great difference between the material properties, the relationships for the granular material are surprisingly close to those for subgrade soils. It is quite possible that the limiting strain concept could be extended to apply to granular subbase and base materials.

USE OF LABORATORY DATA

It has been shown that the repeated load triaxial test is a method for developing the relationship between resilient strain and permanent strain. There is still some question as to the best procedure for measuring the permanent strain and that the results obtained will depend on the particular procedure used.

In addition to establishing or verifying limiting strain criteria the data presented in this manner can also be used to estimate the permanent deformation of the subgrade. In a report on the structural analysis

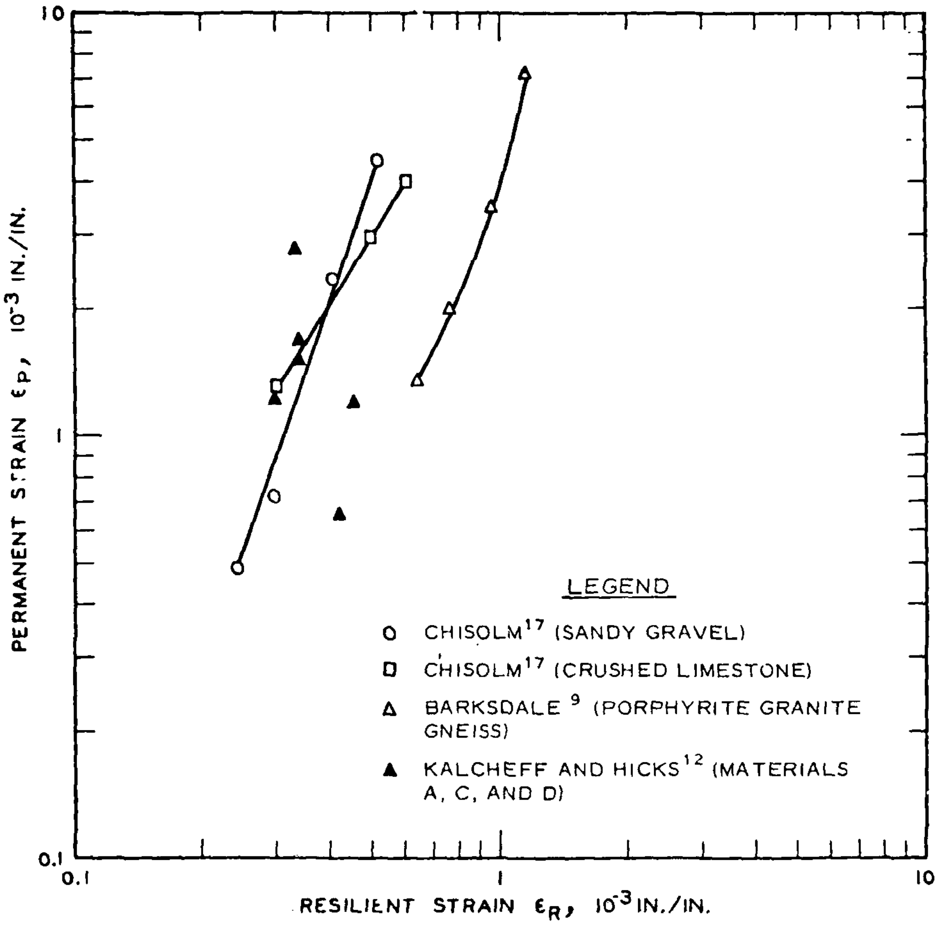


Figure B6. Relationships between permanent strain and resilient strain for granular mate·ials

(Reference 18) of insulated layers the author used the WES test data to show that the rutting of the subgrade was an insignificant part of the total rutting of the pavement. In the analysis the distribution of the computed vertical resilient strain within the subgrade is shown in Figure 7. Using this distribution of resilient strain and the relationship between resilient strain and permanent strain, as established by Chisolm, then the distribution of permanent strain as shown in Figure 7 was determined (that is, $\varepsilon_p = \varepsilon_R \left(\frac{\varepsilon_p'}{\varepsilon_R'}\right)$), where

ε_p = the computed permanent strain

ε_R = the computed resilient strain

ε_p' = measured permanent strain in repeated load triaxial test

ε_R' = measured resilient strain in repeated load triaxial test

The total permanent deformation (Δ_p) was then determined by assuming the permanent strain went to zero at a depth of 120 in. below the top of the subgrade and computing the area under the curve; i.e., $\Delta_p = \int_{120}^{0} \varepsilon_R \left(\frac{\varepsilon_p'}{\varepsilon_R'}\right)$. Using this procedure, the deformation at the top of the subgrade was estimated to be 0.08 in.

A conservative estimate of the permanent deformation at the top of the subgrade can be estimated quickly and easily by using the ratio of permanent strain to resilient strain as determined from the computed resilient strain at the top of the subgrade. The ratio is then used as a constant multiplier to the computed subgrade resilient deformation (Δ_R) to estimate the subgrade permanent deformation. This assumes that the ratio of permanent strain to resilient strain remains constant with depth which, of course, is not true. This is to say if $\frac{\varepsilon_p'}{\varepsilon_R'}$ is a constant, then the previous equation becomes

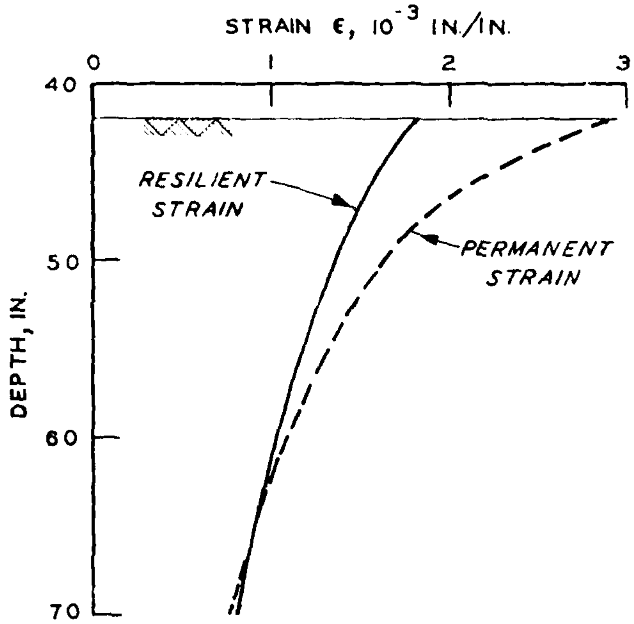


Figure B7. Computed distribution of resilient and permanent strain in the subgrade of a flexible pavement test section

$\Delta_p = \frac{\epsilon'_p}{\epsilon'_R}, \int_{\infty}^{0} \epsilon_R = \frac{\epsilon'_p}{\epsilon'_R} \Delta_R$. For the example given above the strain ratio is about 1.6 and the computed resilient deformation at the top of the subgrade was 0.115, giving an estimated permanent deformation of 0.184. This estimate is over twice the previous estimate. The difference being, as mentioned before, that the maximum strain was assumed constant with depth and also that previously it was assumed that the permanent deformation was zero at 120 in., whereas in the latter estimate it was assumed the permanent strain extends to an infinite depth.

The same procedure can be used to compute the permanent deformation in other layers of the pavement systems. Consider the three pavement types as shown in Figure 8 in which sections 1 and 2 were subjected to simulated traffic of a C-5A aircraft and section 3 to the simulated traffic of a 747 aircraft. To compute resilient strains the material properties as shown in Figure 8 were assumed (from procedure given in Reference 2) and a modified version of the Chevron computer program was employed as a mathematical model for computing resilient strains. From the previously discussed laboratory data the relationship of permanent to resilient strain for levels of 100, 1000, and 10,000 strain repetitions as shown in Figure 9 was assumed. These relationships must be considered as a pure guess at the true relationships between the computed resilient strains and the resulting permanent strains in the pavement system. The computations for determining the permanent strains are shown in Tables 2, 3, and 4, and resulting distributions of permanent strain with depth for repetition levels of 100, 1000, and 10,000 coverages are shown in Figures 10, 11, and 12 for pavement sections 1, 2, and 3, respectively. It is to be noted that it has been assumed that no permanent strain occurred in the asphalt concrete and that one coverage produces one strain repetition. It

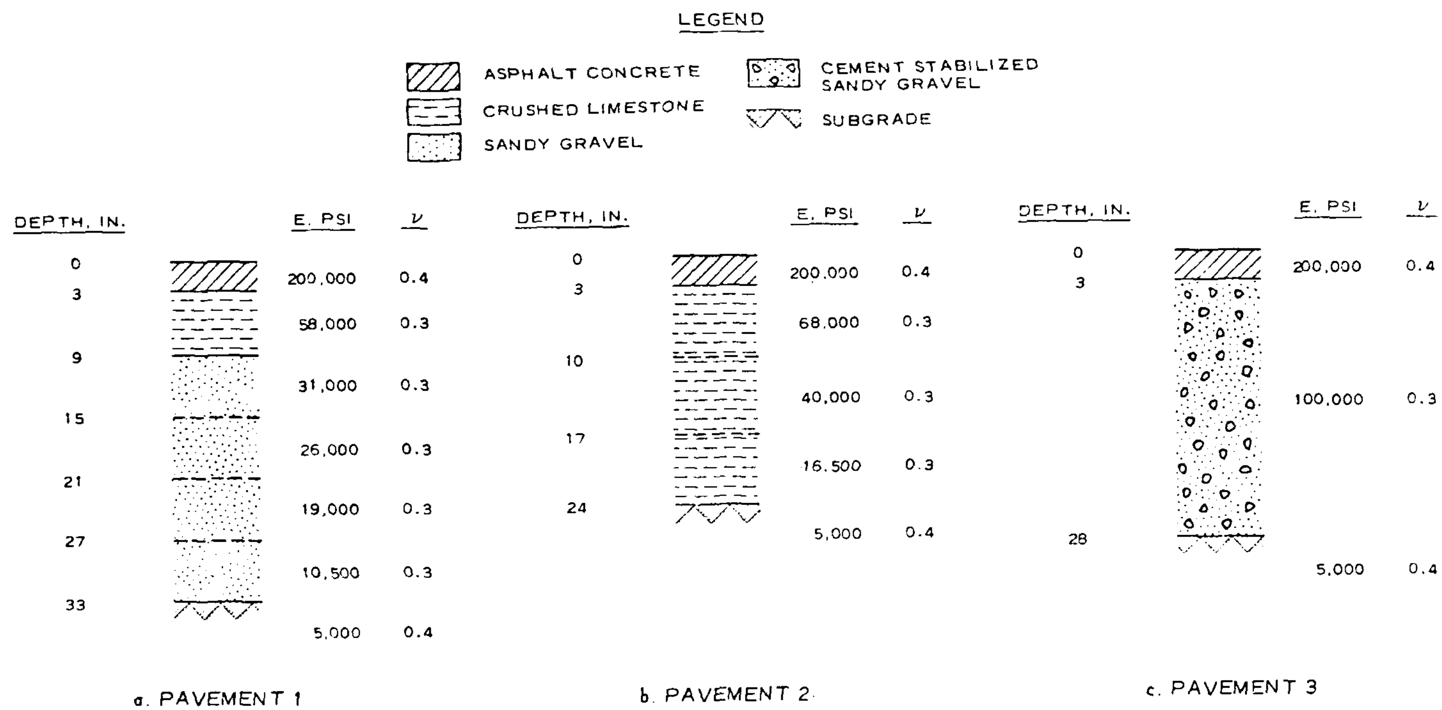


Figure B8. Pavement sections and material properties (References 19, 20, and 21, respectively)

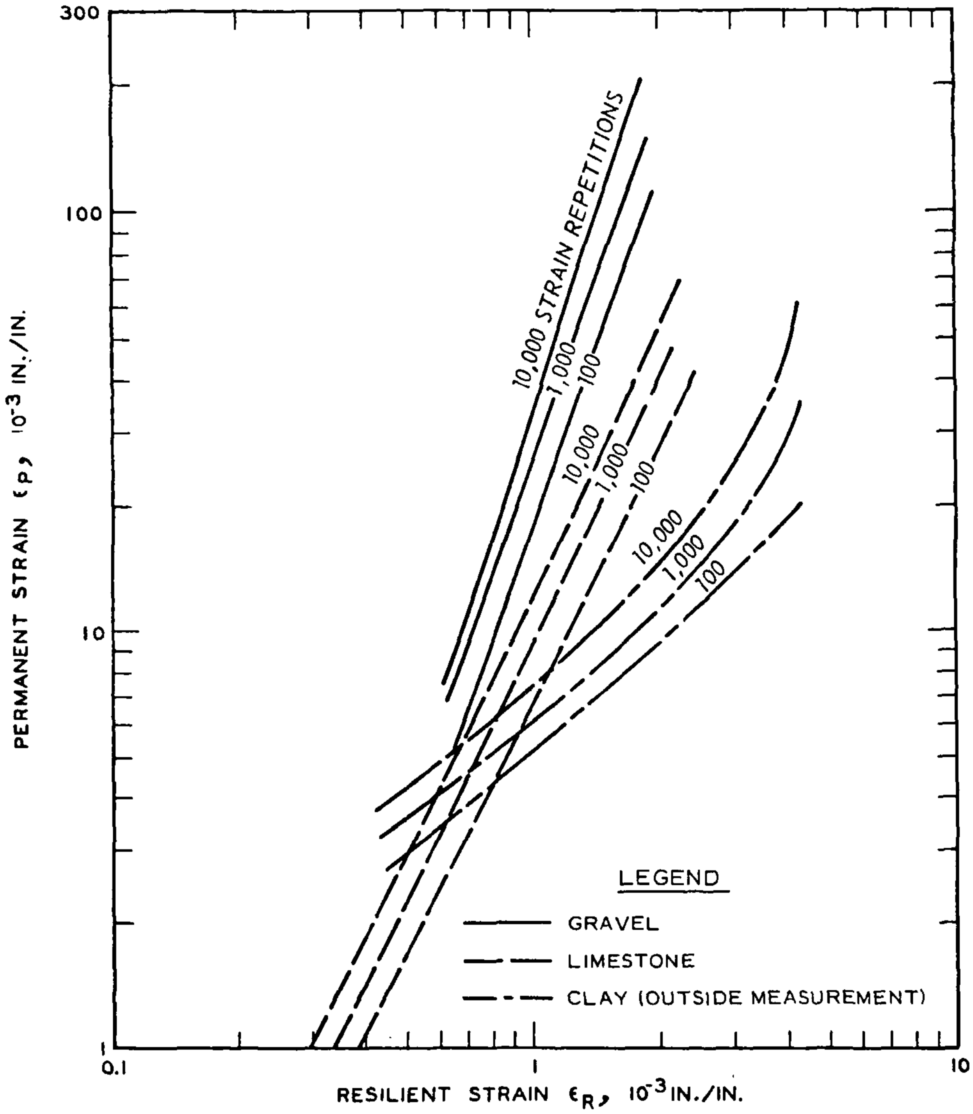


Figure B9. Assumed relationship between permanent strain and resilient

TABLE B2

Computations of Permanent Strain for Pavement 1

Material	Layer	Depth	Δ_R	ε_R	ε_p @ 100	ε_p @ 1000	ε_p 10,000
Cr. St.	2	3	.219	.0015	.0115	.0165	.022
Cr. St.	2	-9	.212	.0011	.0082	.012	.0155
Sand Gr.	3	9	.212	.0016	.068	.10	.145
Sand Gr.	3	12	.208	.0013	.038	.054	.076
Sand Gr.	3	-15	.204	.0010	.018	.026	.034
Sand Gr.	4	15	.204	.0012	.030	.044	.060
Sand Gr.	4	18	.201	.0010	.018	.026	.034
Sand Gr.	4	-21	.198	.0009	.014	.020	.025
Sand Gr.	5	21	.198	.0010	.018	.206	.034
Sand Gr.	5	24	.196	.0009	.014	.020	.025
Sand Gr.	5	-27	.192	.0009	.014	.020	.025
Sand Gr.	6	27	.192	.0012	.030	.044	.060
Sand Gr.	6	30	.189	.0011	.024	.035	.045
Sand Gr.	6	-33	.186	.0011	.024	.035	.045
Clay	7	33	.186	.0016	.0076	.0091	.0115
Clay	7	36	.181	.0015	.0074	.0086	.0109
Clay	7	48	.166	.0012	.0060	.0071	.0088
Clay	7	60	.153	.0010	.0052	.0061	.0074
Clay	7	120	.108	.0006	.0034	.0041	.0048

TABLE B3

Computation of Permanent Strain for Pavement 2

Material	Layer	Depth	Δ_R	ε_R	ε_p @ 100	ε_p @ 1000	ε_p @ 10,000
Cr. St.	2	3	.228	.0011	.0082	.012	.0155
Cr. St.	2	6.5	.225	.0010	.0068	.0097	.0126
Cr. St.	2	-10	.222	.0009	.0056	.0078	.0100
Cr. St.	3	10	.222	.0012	.0098	.014	.019
Cr. St.	3	13.5	.218	.0009	.0056	.0078	.0100
Cr. St.	3	-17	.215	.0009	.0056	.0078	.0100
Cr. St.	4	17	.215	.0013	.0115	.0165	.022
Cr. St.	4	20.5	.211	.0011	.0082	.012	.0155
Cr. St.	4	-24	.207	.0011	.0082	.012	.0155
Clay	5	24	.207	.0020	.0094	.0113	.0148
Clay	5	30	.196	.0017	.0082	.0096	.0123
Clay	5	36	.186	.0015	.0074	.0086	.0109
Clay	5	48	.170	.0012	.0060	.0071	.0088
Clay	5	60	.158	.0010	.0052	.0061	.0074
Clay	5	120	.108	.0006	.0034	.0041	.0048

TABLE B4

Computation of Permanent Strain for Pavement 3

Material	Layer	Depth	Δ_R	ϵ_R	ϵ_p @ 100	ϵ_p @ 1000	ϵ_p @ 10,000
Stab. Gr.	2	3	.226	.0013	.0015	.0165	.022
Stab. Gr.	2	8	.220	.0011	.0062	.012	.0155
Stab. Gr.	2	13	.216	.0007	.0034	.0046	.0060
Stab. Gr.	2	18	.213	.0005	.0017	.0023	.0030
Stab. Gr.	2	23	.211	.0004	.0011	.0014	.0019
Stab. Gr.	2	-28	.209	.0005	.0017	.0023	.0030
Clay	3	28	.209	.0016	.0076	.0091	.0115
Clay	3	30	.205	.0015	.0074	.0086	.0109
Clay	3	36	.197	.0014	.0069	.0081	.0100
Clay	3	48	.181	.0012	.0060	.0071	.0089
Clay	3	60	.168	.0010	.0052	.0061	.0074
Clay	3	120	.108	.0006	.0034	.0041	.0048

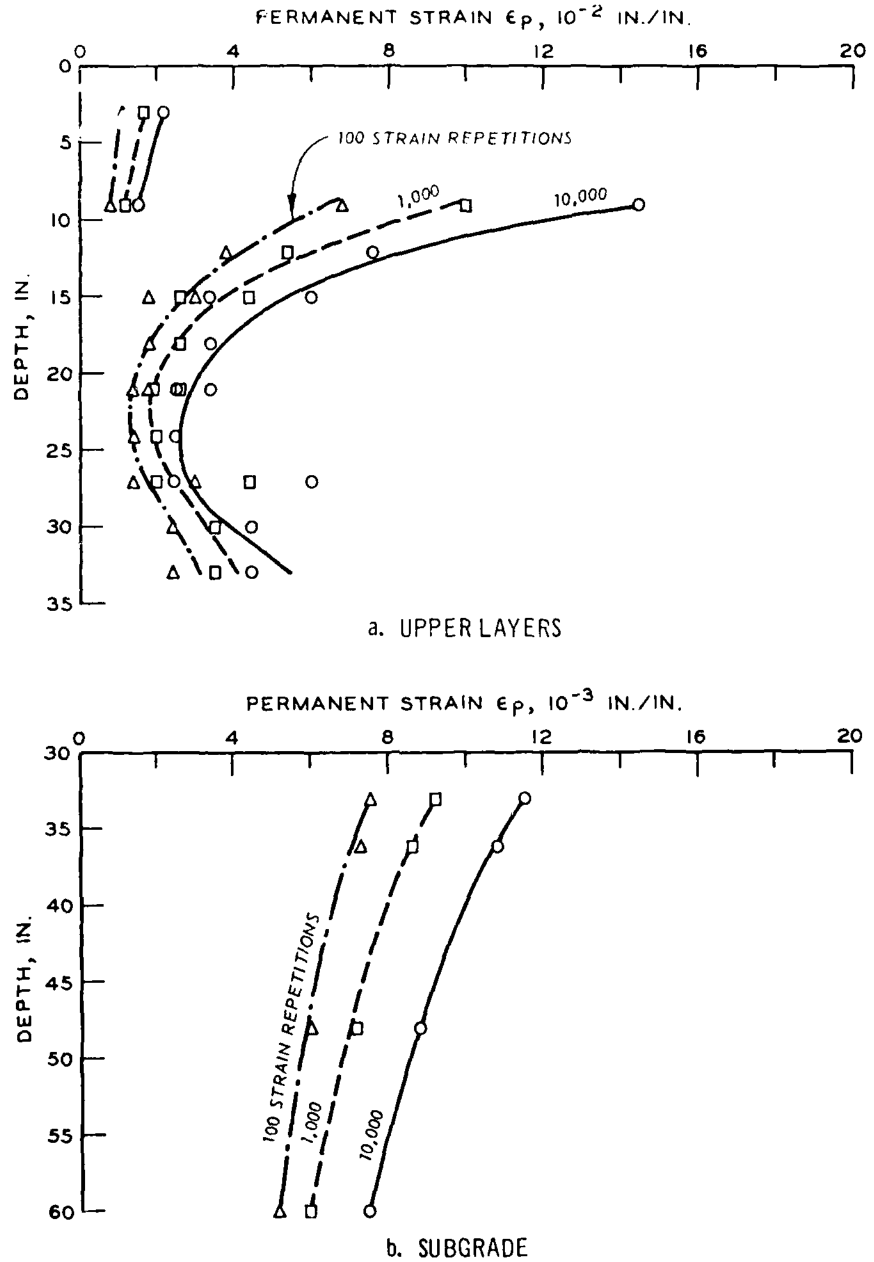


Figure B10. Computed permanent strain for pavement section 1

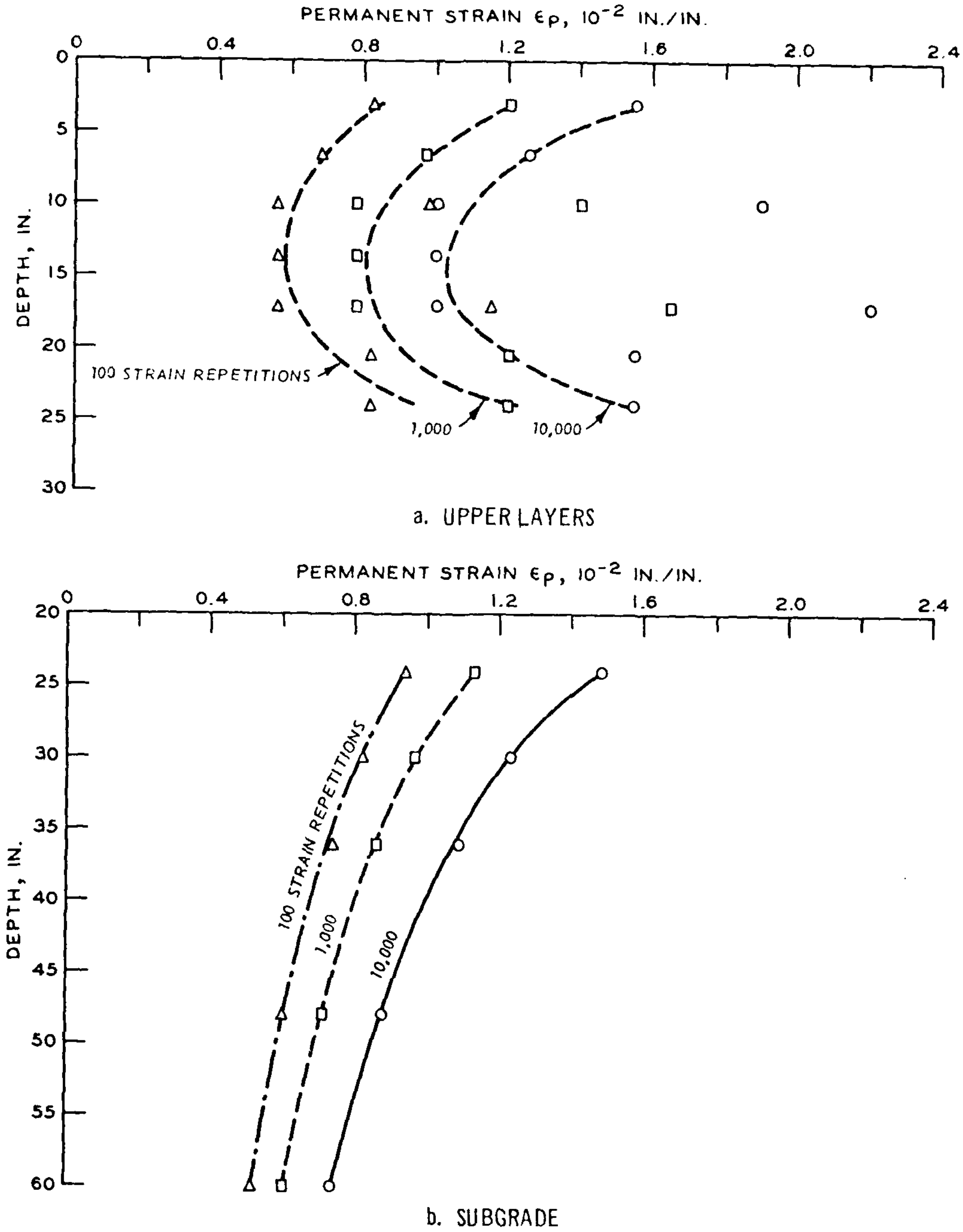
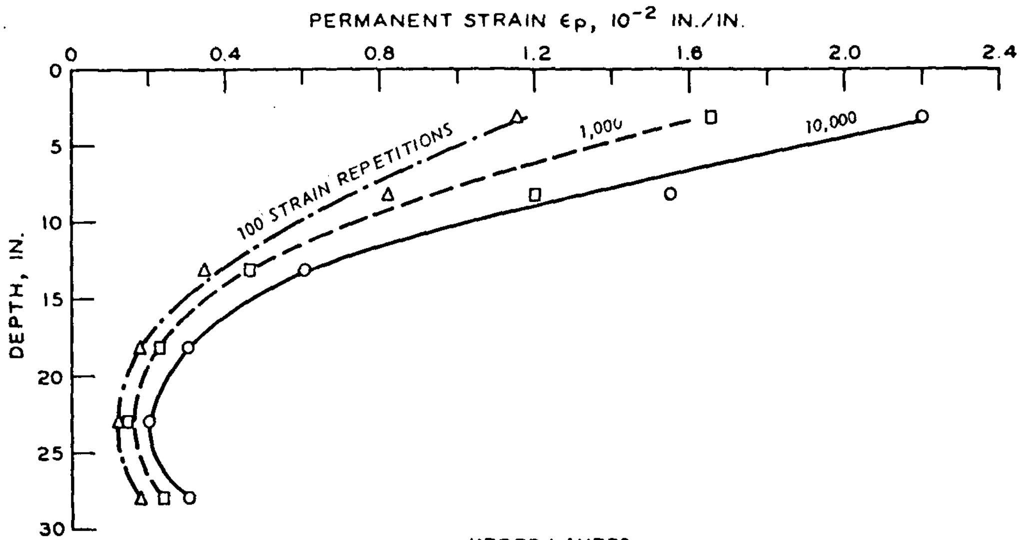
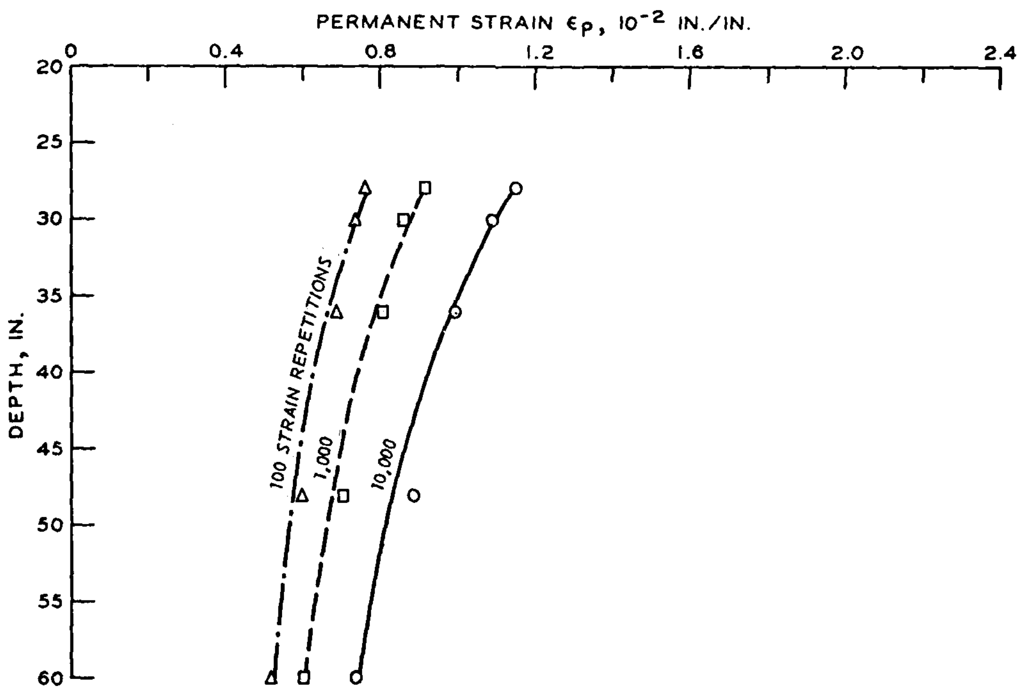


Figure B11. Computed permanent strains for pavement section 2



a. UPPER LAYERS



b. SUBGRADE

Figure B12. Computed permanent strains for pavement section 3 has also been assumed that the stabilized gravel had the same relationship between permanent and resilient strain as the crushed limestone.

To compute the total permanent deformation the function representing the distribution of the permanent strain was numerically integrated by computing the area under the distribution curves. For this computation, the permanent strain was assumed to be zero at a depth of 240 in. The total permanent deformations for the pavement sections at the different coverage levels is given in Table 5. The comparison of the computed permanent deformation with the measured permanent deformation is shown in Figure 13. For each of the test sections the permanent deformation was overestimated at 100 coverages and underestimated at failure.

There are deficiencies in the procedure, notably the assumed relationships between resilient strain and permanent strain, the inability of layered elastic theory to predict strains and the use of a coverage as a strain repetition, which affected the predicted results. Of these deficiencies probably the most serious and the most difficult to correct is the inability of the analytical model to predict the resilient strains. The accuracy of the prediction is greatly affected by the accuracy of the computed resilient strain which emphasizes the need for more accurate models for predicting responses of pavement systems.

TABLE B5

Computed Surface Deformation

Pavement Section	@ 100 Coverages	@ 1000 Coverages	@ 10,000 Coverages
1	1.452	1.785	2.344
2	0.866	1.059	1.285
3	0.772	0.933	1.143

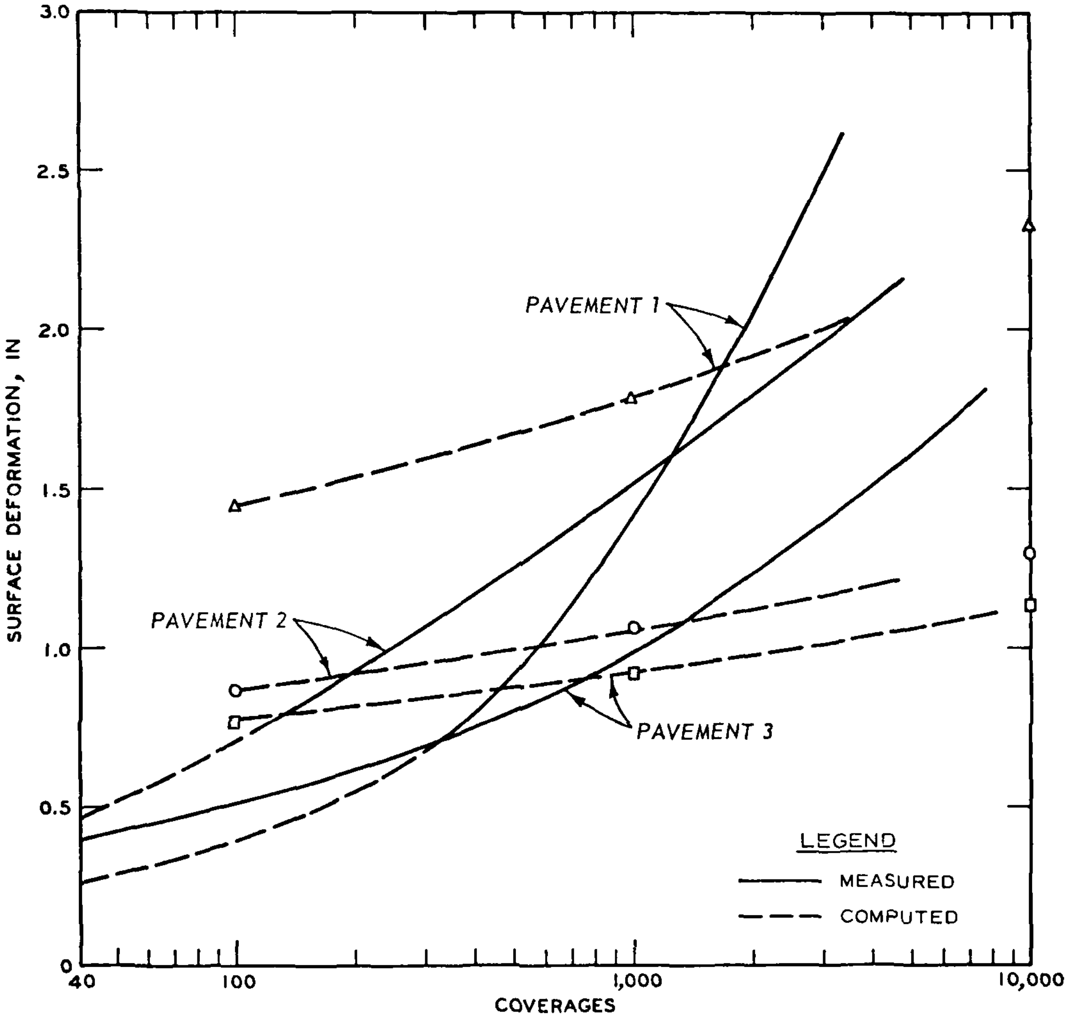


Figure B13. Comparison of computed permanent deformation with measured permanent deformation

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```
      DIMENSION  CB(6,15),S(15),M(15),SS(15),TEMP(15),DD(2,15,2)
&             ,DSTAR(2),SSS(15)
      COMMON  REPS,NPS,Z(15),TR,R,ZR(15)
      CALL ATTACH(10,"/STRESS:",3,0,,)
C
     READ(10,800) LINENO,NLAY,NPS,T,R,SMEAN,SDEV,PSI
C      ************
      PRINT 801,LINENO,NLAY,NPS,T,R,SMEAN,SDEV
      PRINT 810,PSI
      PRINT 802
      DO 10 I = 1,NLAY
C
      READ(10,800) LINENO,(CB(J,I),J=1,2)
C      ************
      PRINT803,LINENO,I, (CB(J,I),J=1,2)
   10 CONTINUE
      PRINT 804
      DO 20 I = 1,NPS
      READ(10,800) LINENO,Z(I),M(I)
      PRINT 805,LINENO,I,M(I),Z(I)
   20 CONTINUE
C
 1000 CONTINUE
C***************2190
C
      READ(10,800) LINENO,REPS
      WRITE(6,806) REPS
      IF(REPS.EQ.0.) GO TO 999
C      COMPUTE DSTAR USING THE MEANS OF ALL VARIABLES
         TR = T/R
      DO 30 I = 1,NPS
         ZR(I) = Z(I)/R
         K = M(I)
         TEMP(I) = CB(1,K)
   30 CONTINUE
C
      CALL STRESS(SMEAN,PSI,S)
C
      CALL DELTA(TEMP,S,DSTAR(1),DSTAR(2))
      WRITE(6,808) DSTAR(1),DSTAR(2)
C
         NVAR = NLAY + 1
C     COMPUTE DD+ AND DD- FOR STRESS
         DUM = SMEAN + SDEV
C
      CALL STRESS(DUM,PSI,SS)
C
      CALL DELTA(TEMP,SS,DD(1,1,1),DD(1,1,2))
C
         DUM = SMEAN - SDEV
C
```

```
      CALL STRESS(DUM,PST,SSS)
      CALL DELTA(TEMP,SSS,DD(2,1,1),DD(2,1,2))
C
C     COMPUTE DD+ AND DD- FOR EACH LAYER
      DO 295 I = 1,NLAY
         J = I + 1
         JK = -1
      DO 295 KK = 1,2
         JK = JK * (-1)
      DO 250 L = 1,NPS
         K = M(L)
      IF(K.NE.I) GO TO 260
         TEMP(L) = CB(1,K)+CB(2,K)*JK
      GO TO 250
  260    TEMP(L) = CB(1,K)
  250 CONTINUE
C        .
      CALL DELTA(TEMP,S,DD(KK,J,1),DD(KK,J,2))
C
C
  295 CONTINUE
      DO 350 I = 1,NVAR
      WRITE(6,809) ((DD(L,I,K),L=1,2),K=1,2)
  350 CONTINUE
C     COMPUTE MEAN AND STANDARD DEVIATION
      DO 400 I = 1,2
         AVG = 1.
         VAR = 1.
      DO 395 J = 1,NVAR
         AV = (DD(1,J,I)+DD(2,J,I))/(2*DSTAR(I))
         AVG = AVG*AV
         V = (DD(1,J,I)-DD(2,J,I))/2
         W = 1-(V/AV)**2
         VAR = VAR * W
  395 CONTINUE
         AVG = AVG * DSTAR(I)
         VAR = SQRT(1-VAR)
         DEV = AVG * VAR
      WRITE(6,807) AVG,DEV,VAR
C     ************
  400 CONTINUE
     GO TO 1000
  999 CONTINUE
  800 FORMAT(V)
  807 FORMAT(5X,'MEAN PUT DEPTH     =',E15.5,/,
     *           5X,'DEVIATION OF PUT  =',E15.5,/,
     *           5X,'VARIATION OF PUT  =',E15.5)
```

```
  801 FORMAT(//,25X,'******************',/,
     5X,'FROM DATA LINE NO.',I5,/,
     5X,'NUMBER OF LAYERS IN SUBGRADE =',I5,/,
     5X,'NUMBER OF STRESS POINTS      =',I5,/,
     5X,'THICKNESS OF STRUCTURAL LAYER =',F15.3,/,
     5X,'RADIUS OF LOADED AREA        =',F10.3,/,
     5X,'MEAN MODULUS RATIO           =',F10.3,/,
     5X,'STANDARD DEVIATION OF RATIO  =',F10.3,//)
  802 FORMAT(5X,'LINE NO.   LAYER NO.   MEAN CBR   STAND. DE.',/)
  803 FORMAT(2(5X,I5),2F15.3)
  804 FORMAT(//,5X,'LINE NO.   POINT NO.   LAYER    DEPTH',/)
  805 FORMAT(3(5X,I5),F15.3)
  806 FORMAT(//,25X,'* * * * * * * * * * *',//,
     5X,'SOLUTION FOR REPITITION LEVEL =',F15.0,//)
  808 FORMAT(//,5X,'DSTAR FOR RESILIENT DEFLECTION  =',E15.5,/,
     5X,'DSTAR FOR PERMANENT DEFLECTION   =',E15.5,//,
     5X,'            DD    VALUES',//,
     5X,'      RESILIENT                     PERMANENT',/,
     5X,'    DD+            DD-           DD+            DD-',/)
  809 FORMAT(4E12.5)
  810 FORMAT(5X,'SURFACE CONTACT PRESSURE  = ',F10.3,//)
  811 FORMAT(//,5X,' DEPTH      MEAN      -SDEV      +SDEV',/,
     5X,'            STRESS     STRESS     STRESS')
  812 FORMAT(//,5X,4F8.2)

      STOP
      END
      SUBROUTINE DELTA(CBR,S,DD1,DD2)
      DIMENSION CBR(15),S(15),D(2),D1(2)
     ,E1(2),E(2)
      COMMON REPS,NPS,Z(15),TR,R,SR(15)
        Y = .4*(REPS**.12)
      E1(1)=0.
      E1(2)=0.
      D(1)=0.
        N = NPS-1
      D(2) =0.
      DO 100 I = 1,N
        J = NPS-I
        S = 4.5682-(1.8681*(2.55*CBR(J)))*(6.5+S(J))
        RM=10**S
        E(1) = S(J)/RM
        E(2)=E(1)*(.14*(70000./RM)**Y)
      DO 50 K = 1,2
        D(K)=((E(K)+E1(K))/2)*(Z(J+1)-Z(J))+D(K)
        E1(K) = E(K)
   50 CONTINUE
      PRINT 800,Z(J),CBR(J),S(J),RM
```

```
      PRINT 800, ((E(I,II),E(II),D(II)),II=1,3)
  800 FORMAT(6F10.3)
  100 CONTINUE
         DD1 =D(1)
         DD2 =D(2)
      RETURN
      END
      SUBROUTINE STRESS(RATIO,RII,S)
      DIMENSION S(15),XX(5),YY(4),ZZ(12),STRAIN(11,5,4),
             XNUMEL(4),NUMELX(4)
      COMMON REPS,NPI,Z(15),TR,R,ZR(15)
      DIMENSION NUMELY(4),CC(4),BB(4),AA(4,4)
      DATA XX/0.,.5,1.,2.,4./
      DATA YY/.0,.20103,.6987,1./
      DATA ZZ/.3,.5,1.,1.5,2.,3.,4.,5.,6.,8.,10.,999./
      DATA STRAIN/.395,.537,.53,.384,.268,.142,.086,.057,.04,.023,.015,
            .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
            .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
            .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
            .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
            .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
      .5025,.4864,.4964,.3678,.2621,.1420,.0862,.0572,.0404,.0232,.0151,
      .2926,.5141,.4237,.3217,.2344,.1221,.0824,.0557,.0399,.0231,.0141,
      .3327,.4895,.5095,.3259,.2011,.1143,.0729,.0503,.0268,.0220,.0146,
      .3735,.5182,.5171,.3740,.2611,.1450,.0630,.0433,.0218,.0194,.0133,
      .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
       .7032,.3905,.4203,.3264,.2415,.1370,.0851,.0571,.0406,.0235,.0152,
     .1053,.4811,.2788,.2289,.1785,.1111,.0739,.0519,.0381,.0228,.0151,
     .2285,.4122,.4946,.4444,.1206,.0766,.0533,.0394,.0304,.0195,.0136,
      .3387,.4880,.4960,.3596,.2529,.1539,.0368,.0269,.0209,.0142,.0104,
      .395,.537,.530,.384,.268,.142,.086,.057,.040,.023,.015,
      .9065,.3042,.3457,.27707,.2154,.1286,.0821,.0559,.0302,.0234,.0153,
     -.0669,.4597,.1873,.1640,.1351,.0914,.0642,.0469,.0354,.0219,.0147,
      .1394,.3473,.4710,.4902,.0759,.0525,.0393,.0307,.0247,.0169,.0123,
     .3095,.5626,.4786,.3490,.2485,.1648,.0227,.0175,.0142,.0104,.0081/
C LOCATE Y-OFFSET.
        YLOC = ALOG10(RATIO)
        XLOC = TR
      NR = 5
      NC = 4
      IF(XLOC.LT.XX(1))XLOC=XX(1)
      IF(XLOC.GT.XX(NR))XLOC=XX(NR)
      DO 55 I=1,NR
      IF(XX(I).LT.XLOC)GO TO 55
      IF(XX(I).GT.XLOC)GO TO 42
      N9=I
      GO TO 60
   42 N9=I-1
```

```
      GO TO 60
   55 CONTINUE
   60 CONTINUE
C LOCATE ASPHALT MODULUS.
      IF(YLOC.LT.YY(1))YLOC=YY(1)
      IF(YLOC.GT.YY(NC))YLOC=YY(NC)
      DO 70 I=1,NC
      IF(YY(I).LT.YLOC)GO TO 70
      IF(YY(I).GT.YLOC)GO TO 88
  942 FORMAT(I4,F12.2,F15.6,F15.6,F15.6)
      M9=I
      GO TO 90
   88 M9=I-1
      GO TO 90
   70 CONTINUE
   90 CONTINUE
      NZ = 1
      N1 = NPZ-1
      DO 400 JJ = 1,N1
      DO 300 II =NZ,11
      IF(ZR(JJ).NE.ZZ(II)) GO TO 301
      NZ = II
      GO TO 399
  301 IF(ZR(JJ).GE.ZZ(II+1)) GO TO 300
      TEMP = (ZZ(II)+ZZ(II+1))/2
      IF(ZR(JJ).GT.TEMP) GO TO 302
      ZR(JJ) = ZZ(II)
      NZ = II
      Z(JJ) = ZR(JJ)*R
      GO TO 399
  302 ZR(JJ) = ZZ(II+1)
  300 CONTINUE
  399 CONTINUE
  333 CONTINUE
      XNUMEL(1)=STRAIN(NZ,N9,M9)
      XNUMEL(2)=STRAIN(NZ,N9,M9+1)
      XNUMEL(3)=STRAIN(NZ,N9+1,M9)
      XNUMEL(4)=STRAIN(NZ,N9+1,M9+1)
      NUMELX(1)=N9
      NUMELY(1)=M9
C
      NUMELX(2)=N9
      NUMELY(2)=M9+1
      NUMELX(3)=N9+1
      NUMELY(3)=M9
      NUMELX(4)=N9+1
      NUMELY(4)=M9+1
      DO 111 I=1,4
      CC(I)=0.
```

```
      BB(I)=XNUMEL(I)
      IIX=NUMELX(I)
      IIY=NUMELY(I)
      AA(I,1)=1.
      AA(I,2)=XX(IIX)
      AA(I,3)=YY(IIY)
  111 AA(I,4)=XX(IIX)*YY(IIY)
      KODE=0
      CALL SLAE(AA,BB,4,0.,4,4,CC,DET,KODE)
      IF(KODE.EQ.0)GO TO 150
      STOP
  150 SIG=CC(1)+CC(2)*XLOC+CC(3)*YLOC+CC(4)*XLOC*YLOC
      S(III) = SIG * PII
  400 CONTINUE
      S(NPS) = 0.
      RETURN
      END
      SUBROUTINE SLAE(AA,BB,N,ACC,N1,N2,CC,DET,KODE)
      DIMENSION AA(N1,N2),BB(N1),CC(N1)
      NM1=N-1
      DET=1.
C
      DO 150 K=1,NM1
      KP1=K+1
      L=K
      DO 100 I=KP1,N
      IF(ABS(AA(I,K)).LE.ABS(AA(L,K)))GO TO 100
      L=I
  100 CONTINUE
      IF(L.LE.K)GO TO 120
      DO 110 J=K,N
      TEMP=AA(K,J)
      AA(K,J)=AA(L,J)
  110 AA(L,J)=TEMP
      TEMP=BB(K)
      BB(K)=BB(L)
      BB(L)=TEMP
      DET=-DET
C
  120 PIVOT=AA(K,K)
      DET=DET*PIVOT
      IF(ABS(PIVOT).LE.ACC)GO TO 999
      DO 150 I=KP1,N
      FAC=AA(I,K)/PIVOT
      AA(I,K)=0.
      DO 130 J=KP1,N
  130 AA(I,J)=AA(I,J)-FAC*AA(K,J)
  150 BB(I)=BB(I)-FAC*BB(K)
C
```

```
C CHECK LAST ELEMENT. THEN BACK-SOLVE.
C
      PIVOT=AA(N,N)
      DET=DET*PIVOT
      IF(ABS(PIVOT).LE.ACC)GO TO 999
      CC(N)=BB(N)/PIVOT
      I=NM1
  200 IP1=I+1
      SUM=0.
      DO 210 J=IP1,N
  210 SUM=SUM+AA(I,J)*CC(J)
      CC(I)=(BB(I)-SUM)/AA(I,I)
      I=I-1
      IF(I.GT.0)GO TO 200
      RETURN
  999 KODE=1
      RETURN
      END
```

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