# AN ANALYSIS AND DESIGN PROCEDURE FOR HIGHWAY-RAILROAD GRADE CROSSING FOUNDATIONS

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Structural and Geometric Design of
Highway-Railroad Grade Crossings

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16. Abstract

Both highway and railroad organizations are concerned with the maintenance problems of highway-railroad grade crossings. The Texas State Department of Highways and Public Transportation spends approximately one-half million dollars yearly for the maintenance of grade crossings. It has been the usual experience of engineers and it is a conviction implicit in this study that a major portion of such maintenance costs may be reduced by an improved knowledge of actual behavior of a railroad track under both railway and highway traffic and the influence of environmental factors. Up to the present time, no rational approach to analysis and design of a grade crossing structure has been available.

In this study a design system for a grade crossing is developed. A unique design criterion of permanent differential deformation between railroad track and adjacent highway pavements is established. This criterion is related to other existing criteria, available in pavement design literature, which are related to the rideability.

Polynomial stress equations are developed separately for railroad and highway pavements under their typical design wheel loads to predict stresses at different depths. Characteristic properties of all materials involved, such as resilient modulus and permanent deformation under repeated loading and considered. The influence of environmental factors such as temperature and moisture balance on subgrade material characteristics is also included. A computer program is developed to calculate the differential deformation (the design criterion) for the purpose of the design of a grade crossing. (continued on back side)

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The concept of differential deformation as a design criterion and the design system proposed in this report constitutes a new and rational approach to the design of highway and railroad grade crossings. Several example problems are presented to illustrate the whole design system. These examples also illustrate how these designs must change according to the variations in expected loading, temperature, climatic zone, and subgrade soil.

## PREFACE

This is the fourth and final report of a series of reports from the study entitled "Structural and Geometric Design of Highway-Railroad Grade Crossings." The study is sponsored by the State Department of Highways and Public Transportation in cooperation with the Federal Highway Administration. This report describes a comprehensive design method for the foundation of a grade crossing. A unique design criterion of permanent differential deformation between the grade crossing structure and the adjacent pavement due to an expected number of repetitions of wheel loads (both for railway and highway traffic) is established. This design criterion is related to two performance criteria: dynamic load profile and roughness index. A computer program is developed for the purpose of analyzing and designing a grade crossing structure. This program requires approximately 128 k memory core and has a very simple input data format.

## ACKNOWLEDGEMENTS

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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

## LISTS OF REPORTS

Report No. 164-1, "Structural and Geometric Characteristics of Highway-Railroad Grade Crossings," by Thomas M. Newton, Robert L. Lytton, and Robert M. Olson, describes the crossing distribution and geometric characteristics, crossing appraisals, drainage, dynamic loading, stabilization fabrics, and structural details for improved life and rideability.

Report No. 164-2, "Computation of Dynamic Loads at Grade Crossings; A User's Manual of the Computer Program," by Aziz Ahmad and Robert L. Lytton describes the revisions to the computer program DYMOL, including input formats of the program.

Report No. 164-3, "Dynamic Properties of Subgrade Soils, Including Environmental Effects," by Earl V. Edris, Jr. and Robert L. Lytton describes the work done in the repetitive load testing of subgrade soils and how the resilient modulus and residual strain of these soils are related to the climatic conditions.

Report No. 164-4F, "An Analysis and Design Procedure for Highway-Railroad Grade Crossing Foundations," by Aziz Ahmad, Robert L. Lytton and Robert M. Olson, describes a comprehensive design procedure for the foundation of a highway-railroad grade crossing involving a computerized design system.

## SUMMARY

This is the fourth and final report in a series from the study entitled "Structural and Geometric Design of Highway-Railroad Grade Crossings." The five chapters in this report describe a comprehensive design procedure for the foundation of a grade crossing involving a computerized design system.

In present day construction practice, the selection of materials and layer thicknesses for a railroad crossing structure is based on a trial and error approach. Although several improved design methods for highway pavements are available, prior to the work in this report, none of these methods had ever been applied to the design of highway-railroad grade crossings.

The design procedure developed herein is based on rideability, which mainly depends on the amount of permanent differential deformation between the railroad track and the adjacent highway pavement. Repetitions of wheel loads cause permanent differential deformation. Due to the difference in wheel loadings, material properties, and the track and pavement structures, each will deform differently after the passage of a number of repetitions of wheel loads (expected in a design period).

Layer thicknesses of the crossing structure and adjacent pavement, their wheel loadings and the properties of all the materials involved as they are affected by the local climate determine the level of stress that acts at different points in these foundation layers. The repetition of these stresses produces the permanent differential deformation which must remain within acceptable limits if the foundation layers are properly designed. The influence of the permanent differential deformation on increasing highway dynamic load and the increase in dynamic railway wheel loads due to higher train speed is considered while computing the stresses.

Characteristic properties of fine grained subgrade materials including the influence of environmental factors such as

temperature and suction on subgrade material properties are completely described.

Several example problems are presented to illustrate the whole design system. These examples also illustrate how these designs must change according to the variations in expected loading, temperature, climatic zone and subgrade soil.

## IMPLEMENTATION STATEMENT

In present day construction practice, the selection of materials and layer thicknesses for a railroad crossing structure is based on a trial and error approach. Although several improved design methods for highway pavements are available, prior to the work in this report, none of these methods had ever been applied to the design of a highway-railroad grade crossing.

In this study, a computerized design system for a highway-railroad grade crossing foundation is developed. A unique design criterion of permanent differential deformation between railroad track and adjacent highway pavements is established. This design criterion is related to two performance criteria: dynamic load profile and roughness index, which is a measure of the ride roughness experienced by a passenger vehicle passing over the grade crossing.

The influence of the permanent differential deformation on increasing highway dynamic load is included in the computer program. The increase in dynamic railway wheel loads due to higher train speed is also considered.

Characteristic properties of all materials involved including the influence of environmental factors such as temperature and suction on subgrade material properties are considered. The computer program calculates the permanent differential deformation (the design criterion) due to the passage of an expected number of repetitions of wheel loads (required to serve a design period) for both highway and railway traffic.

Temperature and climatic conditions at a particular location greatly influence the design of a grade crossing. However, the design system developed in this study, can be used very effectively in different regions with different temperature and climatic conditions. The suction level corresponding to good drainage conditions and low water table of a particular location is considered in this study. However, a designer may choose a

lower suction value to represent a poor drainage condition at a particular location.

The number of wheel load repetitions (to serve a design period) for highway and railway traffic are considered separately in the calculations, and therefore, this design system can handle any combination of high and light volumes in railway and highway traffic.

The computer program, developed in this design system, requires approximately 128 k memory core and a short computation time for a typical grade crossing design. This program can also be used:

1) to find the most effective ballast depth in different climatic and soil conditions
2) to predict the performance of presently available commercial crossing materials.

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# CHAPTER I
# INTRODUCTION

Highway-railroad grade crossings are the subject of continuing concern for both highway and railroad organizations because of maintenance problems caused by load-associated roughness. Steady increases in highway traffic along with their increasing load and speed have made these problems still worse. The Texas State Department of Highways and Public Transportation spends approximately one-half million dollars yearly for the maintenance of grade crossings. It is believed that knowledge of the behavior of the railroad track structure under both highway and railway traffic will lead to improved designs.

The magnitude of dynamic highway loads over the grade crossing increases with time as the pavement on each side of the crossing becomes distressed because of repeated loads. Distress such as pumping and potholes may result in loss of control of the vehicle at higher crossing speeds. However, it is the relative permanent deformation between railroad and pavement that determines, to a large extent, the degree of roughness experienced by passing traffic. Therefore, material properties such as resilient modulus and permanent strain of grade crossing materials are very important for design purposes.

Although rail traffic has declined in recent years, increasing length of trains, weight of rail cars and locomotives, and speed of trains have contributed to track structure and crossing failures. Railroads also are concerned with rideability and operation of trains at grade crossings.

## Present Status

It was found in 1967 that there were 2,442 highway-railroad grade crossings on the system of highways maintained by the Texas State Department of Highways and Public Transportation (SDHPT). Types of surface materials include: timber, bituminous, concrete

slabs, rubber panels, metal sections, and others. Timber surfacing panels are installed at approximately 75% of these crossings. A recent inspection of several of these crossings by members of the Texas Transportation Institute staff has revealed that there is a need for major modification in the present design of highway-railroad grade crossings. It is realized that, regardless of the type of surface materials, proper design of track structure, base and subgrade including adequate drainage determine the performance and life of a grade crossing (24)*.

Up to the present day, the construction and development of the railroad structure was based on a trial and error approach (29). However, several attempts were made to analyze the stresses in track components during the second half of the 19th century. The majority of these findings were published in the journals such as: Organ für die Fortshritte des Eisenbahnwesens, Proceedings of the American Society of Civil Engineers and the Institution of Civil Engineers of London. In the United States, between the two World Wars, a special committee under ASCE-AREA conducted an extensive research program on the stresses in railroad track (38, 39, 40, 41).

The American Railway Engineering Association (Committee 9 - Highways) has published reports on the merits and economics of various types of grade crossing surfaces in the AREA Proceedings. However, none of this literature has provided information and guidance adequate for the purpose of grade crossing foundation design. Some consideration has been given to new and improved prefabricated grade crossings by AREA Committee 9. In 1948, Owens (33) reported on the design of railroad crossings. He emphasized the need for right angle intersections at crossings. An installation of rubber at a highway-railroad crossing was reported in 1954 (2). Taylor (42)

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* Number in parenthesis refer to corresponding items in the list of references.

discussed in 1955 five different pavement types found at highway-railroad crossings in Texas. In an installation in Iowa, Hund (27) described a multiple crossing that consisted of several types of pavement: concrete, brick and asphalt.

Currently several commercial systems are available for crossings which claim to provide good rideability. However, many important characteristics such as: 1) the influence of crossing profile (roughness characteristics) upon highway vehicle speeds and dynamic loads at the crossing and its approaches, 2) interaction of individual physical and geometrical characteristics of grade crossings, 3) stresses and deformation in ballast, base and subgrade due to both highway and railway loadings, with their dynamic effects, are not yet well defined.

## Objective and Scope

The purpose of this research is to develop an analytical method of evaluating the performance of highway-railroad grade crossings that can be used for design. Therefore two specific objectives of this research are:

1. To establish performance criteria for highway-railroad grade crossings.
2. To develop a method for predicting the performance of the crossing with reasonable accuracy for the purpose of designing its foundation.

In this research the performance of a grade crossing is measured by the following three performance criteria:

1. Dynamic Load Profile
2. Roughness Index
3. Permanent Differential Deformation.

These criteria are inter-related to each other, i.e. increase in one will increase the other two. A detailed description of these criteria is given in Chapter II. Due to the application of loads on a grade crossing, the railway structure and its adjacent pavement deform differently producing a differential

deformation between them. This difference in deformation is due to the difference in their material properties, loading types and the thickness design of their structures. This differential deformation will increase under the action of a great many repetitions of wheel loads, over a number of years until the failure occurs. It will be demonstrated in Chapter II that the criterion of differential deformation controls the design system in this study. However, its critical value for design purposes is chosen to assure that the critical values of the other two criteria (dynamic load and roughness index) are met. Dynamic load and roughness index criteria are related to rideability.

It is clear from the foregoing discussion that emphasis will be placed on computing permanent deformations in the layers of the foundation structure. Permanent deformation is a function of the level of stresses at varying depths produced by

- Load applications
- Number of load applications
- Material properties
- Environmental factors such as temperature and moisture balance.

Polynomial stress equations derived in this study predict stresses in highway and railroad foundation materials. Material characterization in the form of mathematical models for resilient modulus and permanent deformation of foundation materials are presented in Chapter II. The influence of environmental factors such as soil suction and temperature is included in the characterizations of subgrade materials. A detailed description of material characterization is given in Chapter III. Although asphaltic concrete and asphalt treated base course layers are known to fail in fatigue, this was beyond the scope of this study.

A computer program was developed to calcuate the necessary parameters for design purposes. Several example problems were solved and presented in Chapter IV. Conclusions and recommendations are presented in Chapter V.

## Summary

This chapter introduced the necessity for a rational approach to grade crossing analysis and presented a detailed discussion on the present status of grade crossing design. Three criteria for measuring the performance characteristic of a grade crossing were introduced. Permanent differential deformation was considered as the main design criterion and other factors which influence the performance criteria such as stress level, number of load repetitions and environmental factors were discussed.

# CHAPTER II

## DEVELOPMENT OF DESIGN SYSTEM FOR HIGHWAY-RAILROAD GRADE CROSSING FOUNDATION

The design procedure for a highway-railroad grade crossing developed herein is based on rideability, which mainly depends on the amount of permanent differential deformation between railroad track and the adjacent highway pavements. The whole design system can be broadly divided into three phases. The first phase deals with fixing the required dimensions and geometry of a grade crossing based on the type and volume of both highway and railway traffic. This will require the knowledge of typical dimensions of parts of a grade crossing that are essential for train movements. The second phase involves selection of materials for various layers of the foundation, including subgrade, for the track and pavement structures; and the influence of special environmental factors such as temperature, moisture balance, drainage, etc. on the properties of these materials. The third and final phase is concerned with establishing design criteria and acceptable limits to control the design system.

Typical cross-sections of a grade crossing and a highway structure are shown in Figure 1 and 2 respectively. The important dimensions of a typical grade crossing profile are shown in Figure 3. Typical construction materials for highway and grade crossing structures and their material characteristics are discussed in Chapter III.

### DESIGN CRITERIA

Three design criteria were considered: 1) dynamic load profile, 2) roughness index, and 3) permanent differential deformation between railroad track and adjacent

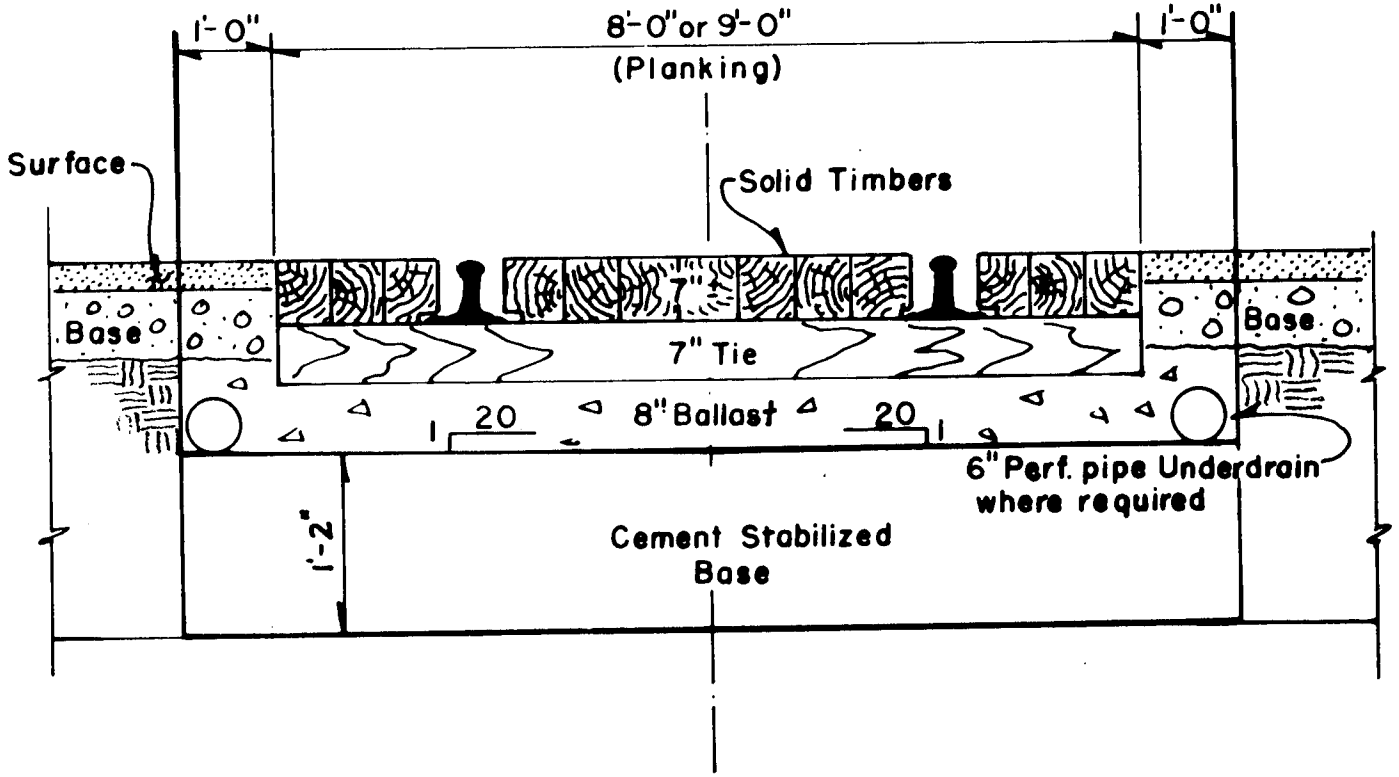


FIGURE 1.- TYPICAL CROSS-SECTION OF HIGHWAY-RAILROAD GRADE CROSSING (32)

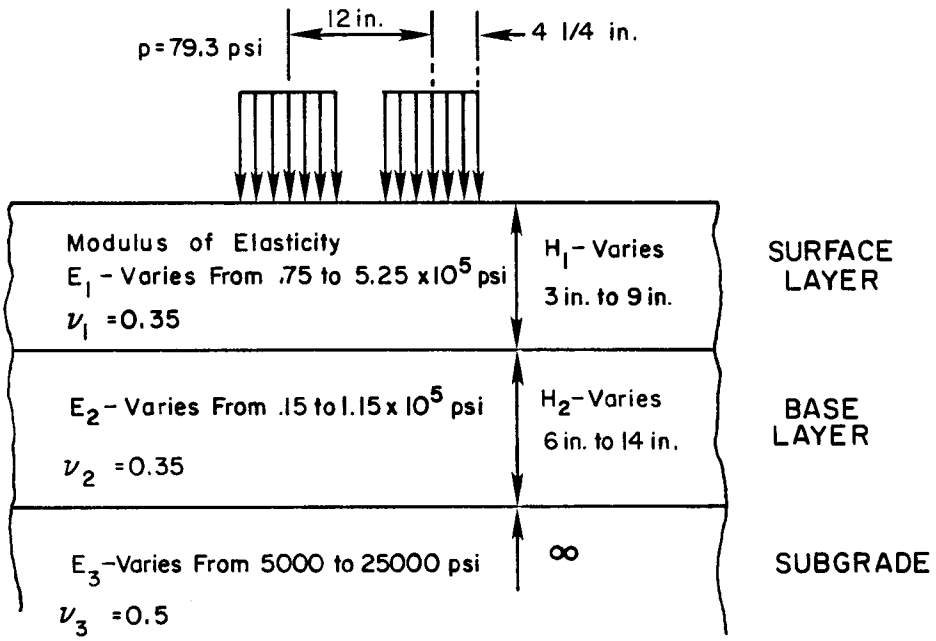


FIGURE 2.- TYPICAL CROSS-SECTION OF HIGHWAY PAVEMENT (21)

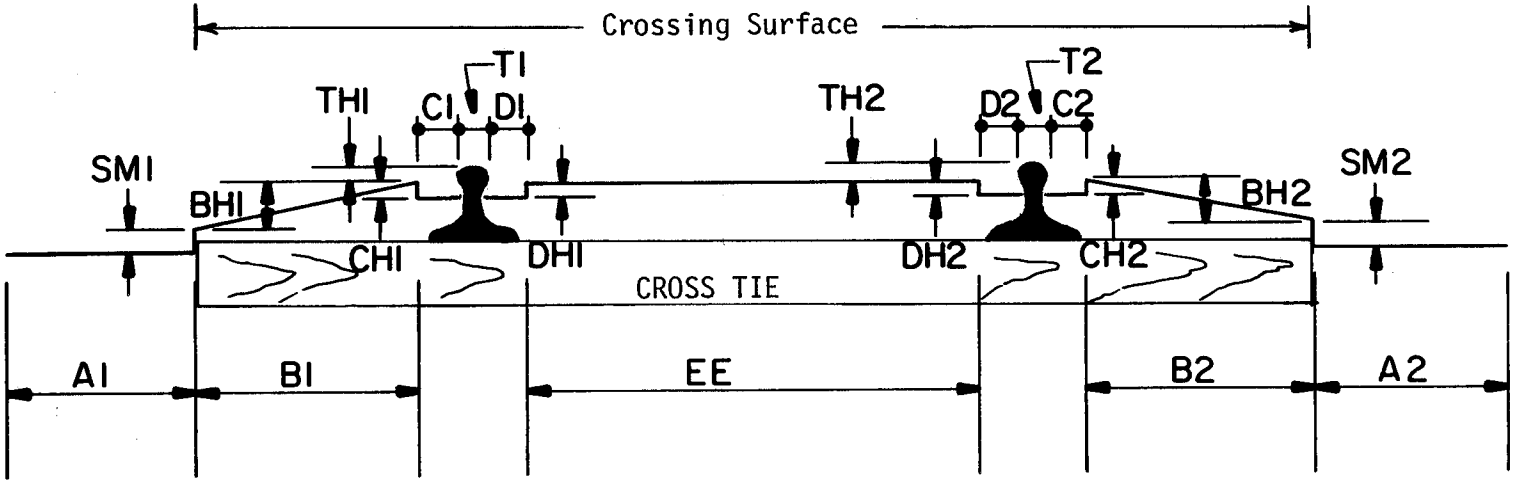


FIGURE 3.- IDEALIZED HIGHWAY-RAILROAD GRADE CROSSING PROFILE (1)

Note: See actual condition in Figure 8.

pavement, each of which is discussed below.

Dynamic Load Profile.- Dynamic load experienced by a vehicle depends upon the interaction of the roughness characteristics of the riding surface, the vehicle characteristics and vehicle speed. The roughness that develops with time and repeated traffic load is represented as dimensions SM1, SM2, BH1, and BH2 in Figure 3. These dimensions can be plus, minus, or zero (i.e. up, down, or flat). All of the dimensions shown in that figure are input data to computer program DYMOL (1), which can be used to predict dynamic loads on grade crossing profiles as a function of differential deformation caused by various highway vehicles and speeds. The dynamic load on the rear axle of a simulated dump truck travelling 55 mph (88.5 km/hr) is influenced by SM1, SM2, BH1 and BH2 of a grade crossing profile as shown in Figure 4. Finney (18) showed that for highway traffic the dynamic loads above static weight varies from 22 to 35 percent in a good pavement zone, 35 to 42 percent in an average zone, and from 42 to 65 percent in a poor zone.

Roughness Index.- This can be defined as the ratio of the summation of rear axle excursions of a vehicle in inches (as recorded by Mays Ride Meter) to the distance it travels in miles (32).

When RI for a crossing is calculated, x is taken as the effective crossing length. An effective crossing length of 150 feet was considered in this calculation (32). Mays Ride Meter readings are a measure of serviceability of pavement surface. A typical Mays Ride Meter chart for a grade crossing is shown in Figure 5. Simulation of the Mays Ride Meter reading is incorporated in the program DYMOL using a simulated passenger vehicle. Figure 6 shows a correlation between the actual and simulated values for pavement surfaces with various serviceability indexes for a 1972 Ford passenger vehicle which was calibrated on the dates shown. The shift of the curve to the right indicates a change in the

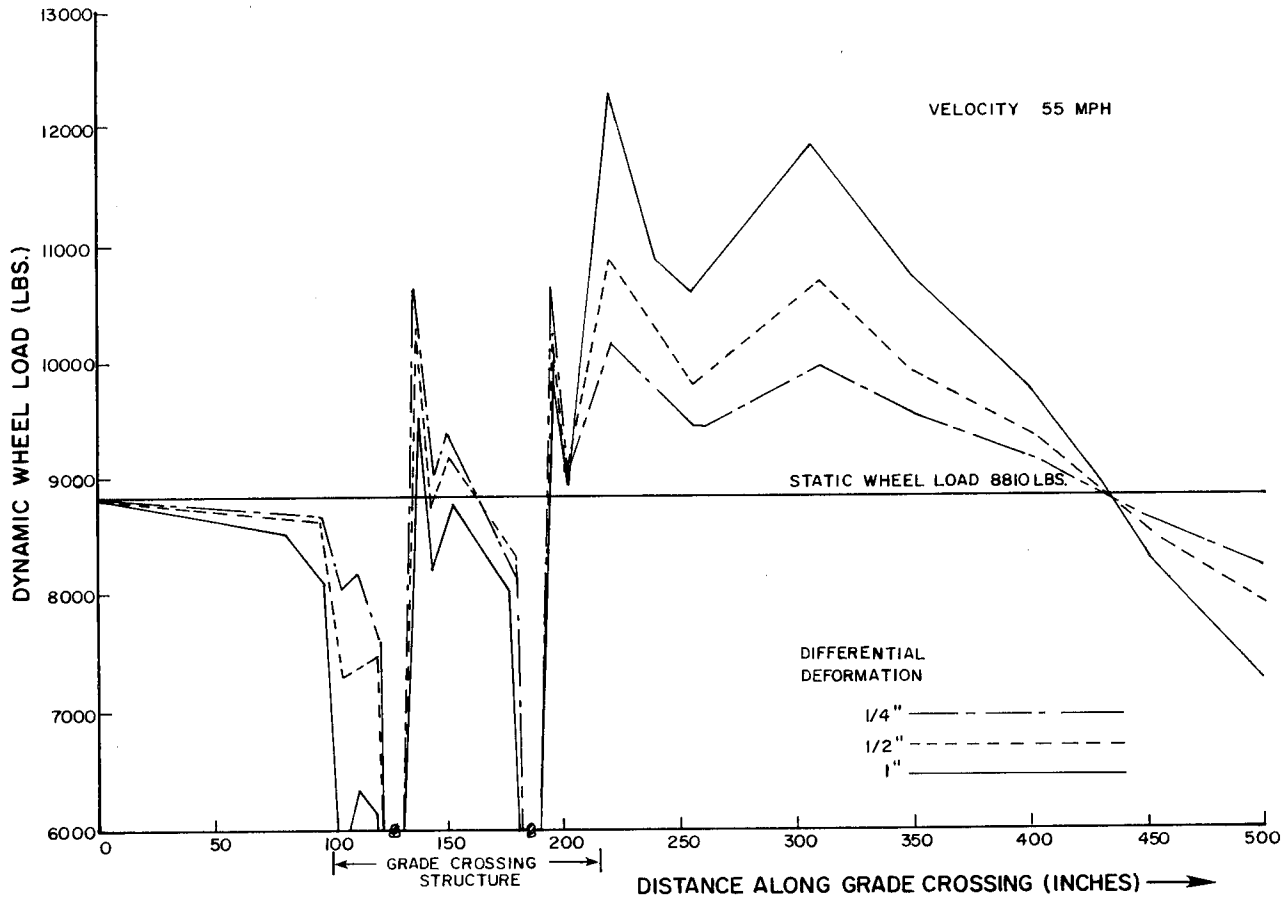


FIGURE 4.- VARIATION OF DYNAMIC WHEEL LOAD OF THE REAR AXLE OF DUMP TRUCK DUE TO THE GEOMETRY OF A GRADE CROSSING PROFILE WITH DIFFERENT AMOUNTS OF DIFFERENTIAL DEFORMATION

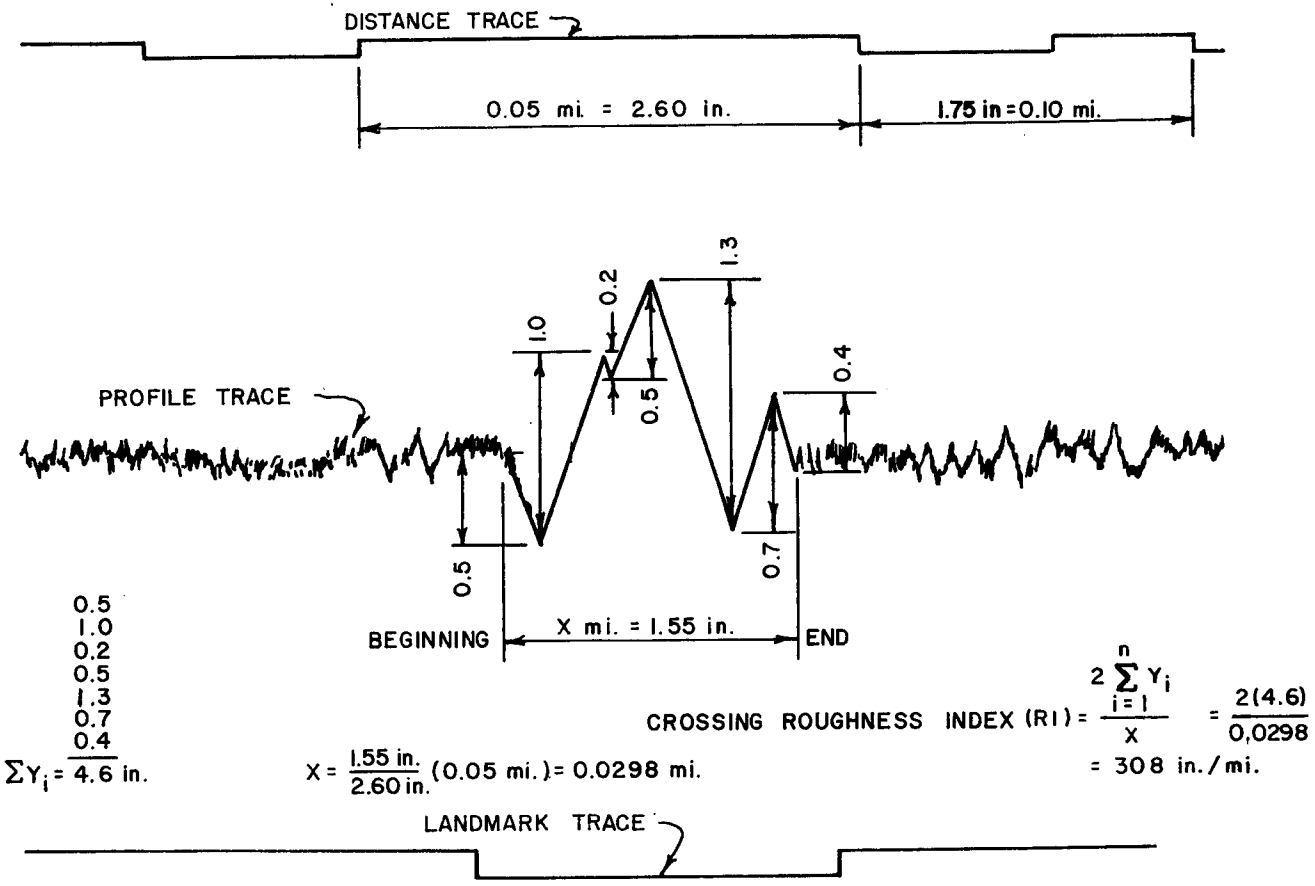


FIGURE 5.- TYPICAL MAYS RIDE METER CHART FOR A GRADE CROSSING PROFILE (32)

suspension characteristics of the vehicle. The computed relationship between the Roughness Index and the permanent differential deformation between railroad track and adjacent pavements is shown in Figure 7. Parametric values of differential deformations were input to DYMOL, which generated the values of roughness index plotted in the figure.

Permanent Differential Deformation.- Repetition of wheel loads causes permanent differnetial deformation between the railroad track and adjacent pavement structures. Due to the difference in wheel loadings, material properties, and the track and pavement structures, each will deform differently. Figure 8 shows a photograph of a grade crossing with differential deformation, note that permanent differential deformation occurs between the track structure and the adjacent pavement. It should also be noted that a vertical deformation occurs at the interface between the track and the approach pavement and that the deformation increases gradually sloping downward towards the rail. The surface between the two rails deforms uniformly. This differential deformation causes roughness on the surface and consequently highway traffic traversing a grade crossing produces a dynamic load effect. Figure 9 shows how dynamic load increases as the differential deformation increases. It may be seen in this figure that a dynamic load decreases with speed. That is a greater dynamic load is produced at a speed of 30 mph (48.3 km/hr) than at a higher speed of 55 mph (88.5 km/hr). This is because dynamic load depends on the frequency response of the vehicle which typically peaks at 1-2 Hz and again at frequencies higher than 10-12 Hz. Thus, within a certain range of vehicle speeds higher dynamic loads are produced with smaller frequency of excitation (speed/wave length). Lytton et al. (30) observed maximum dynamic loads on pavement surface with expansive clay roughness occurs at a vehicle speed of about 20 mph (32.2 km/hr) to 40 mph (64.4 km/hr). The design limit of this criterion

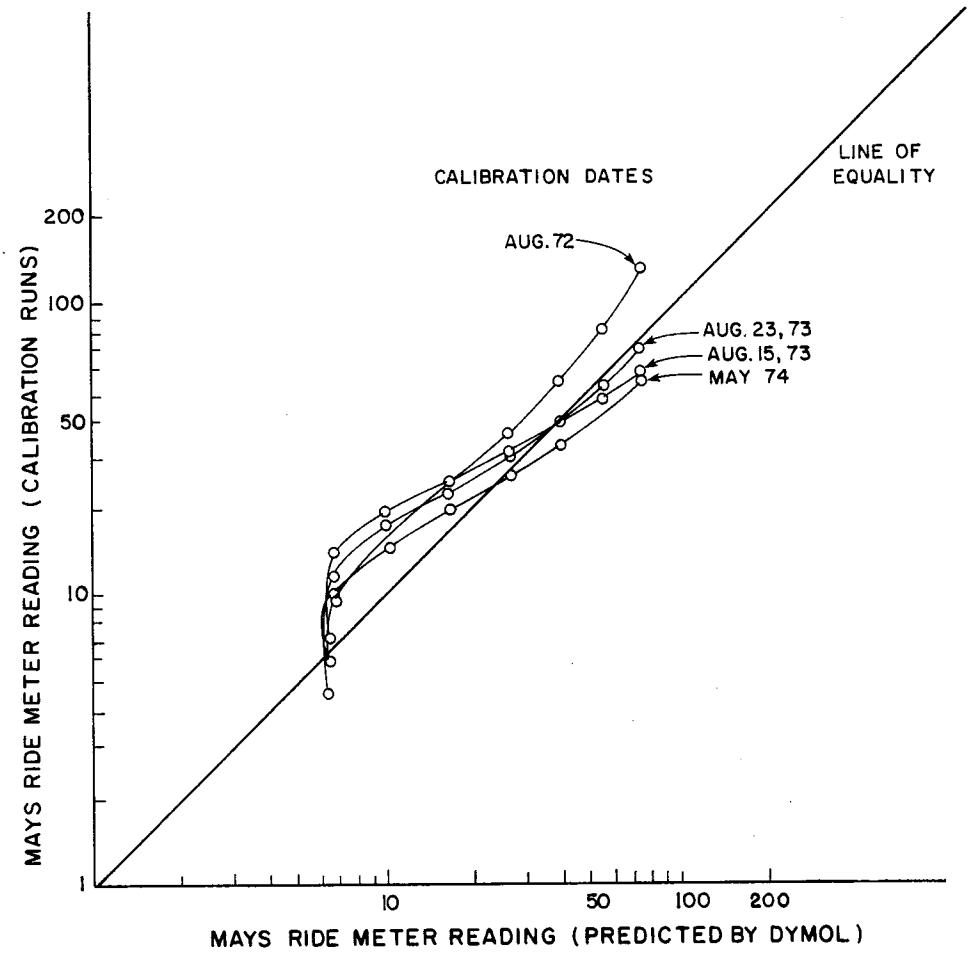


FIGURE 6.- CORRELATION BETWEEN ACTUAL AND PREDICTED MAYS RIDE METER READING

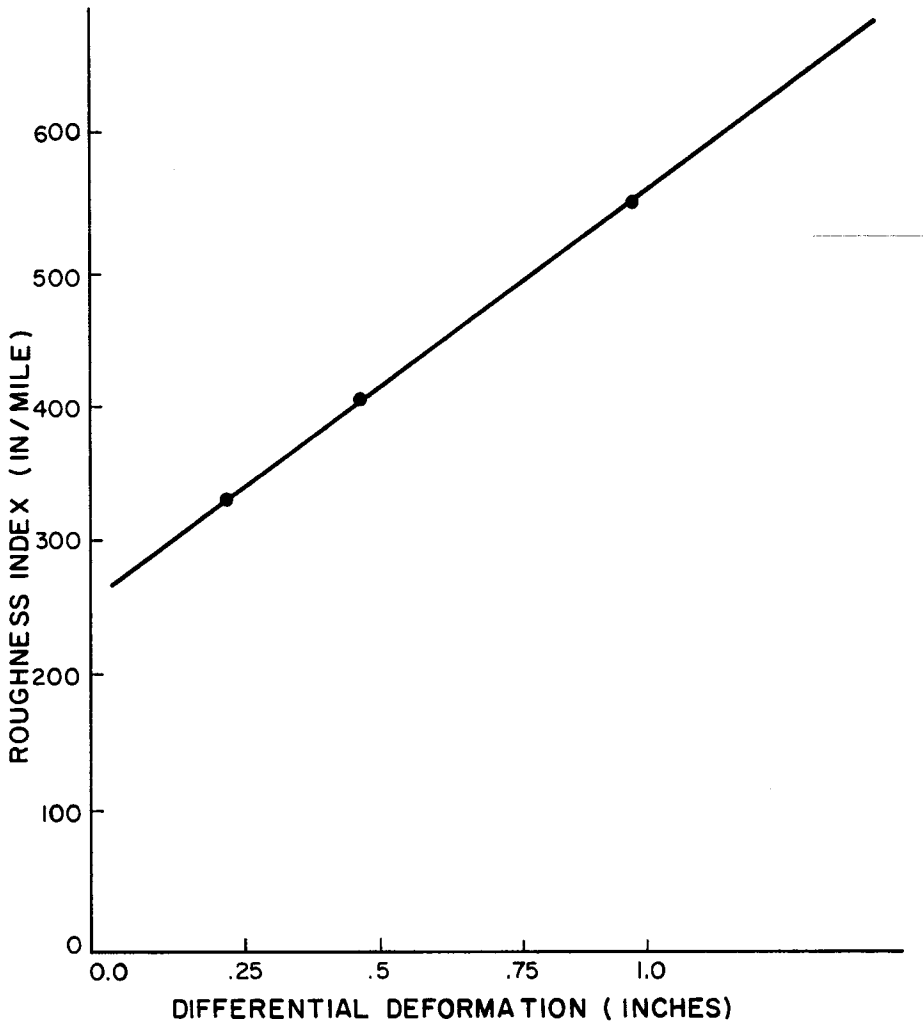


FIGURE 7.- RELATIONSHIP BETWEEN ROUGHNESS INDEX AND DIFFERENTIAL DEFORMATION OF A GRADE CROSSING

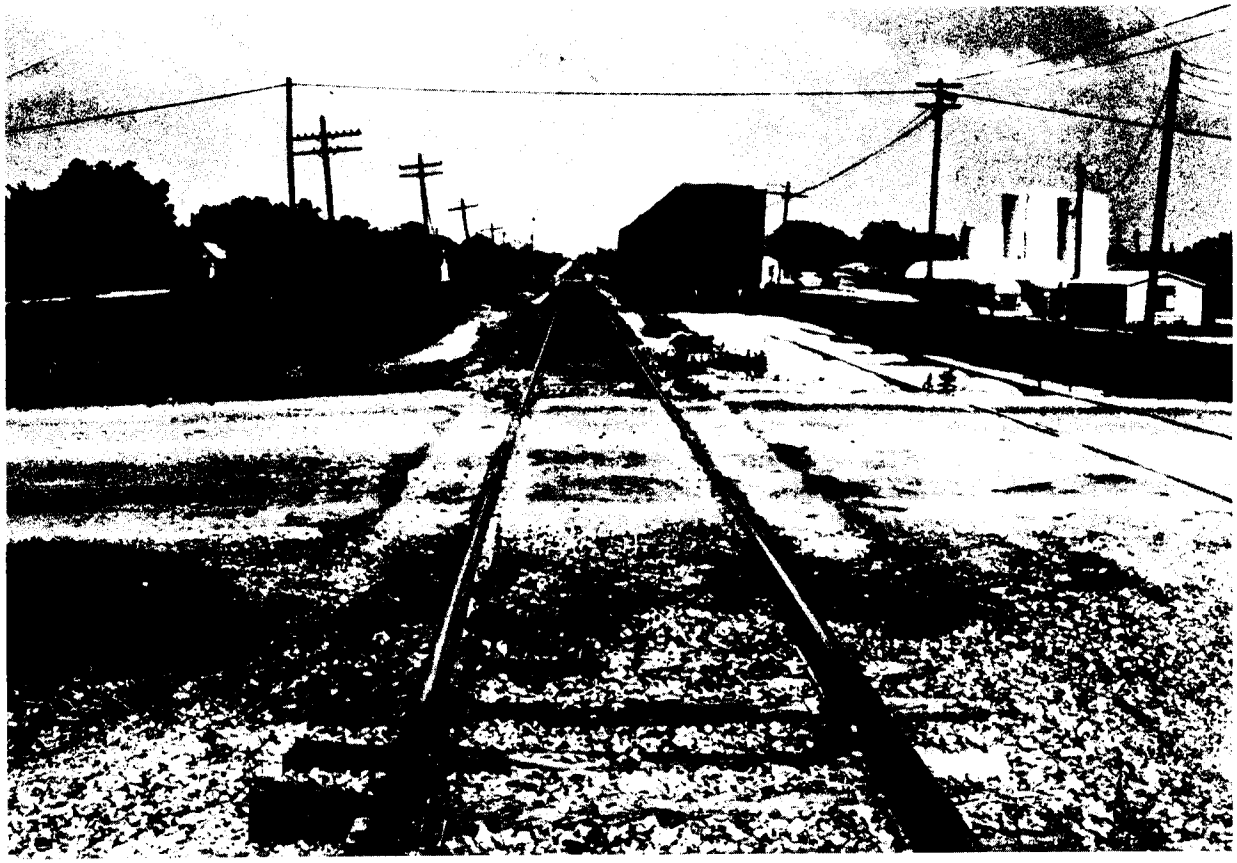

FIGURE 8.- A HIGHWAY-RAILROAD GRADE CROSSING IN HOLLAND, TEXAS WITH ITS TYPICAL DIFFERENTIAL PERMANENT DEFORMATION BETWEEN RAILROAD AND ADJACENT PAVEMENT STRUCTURES

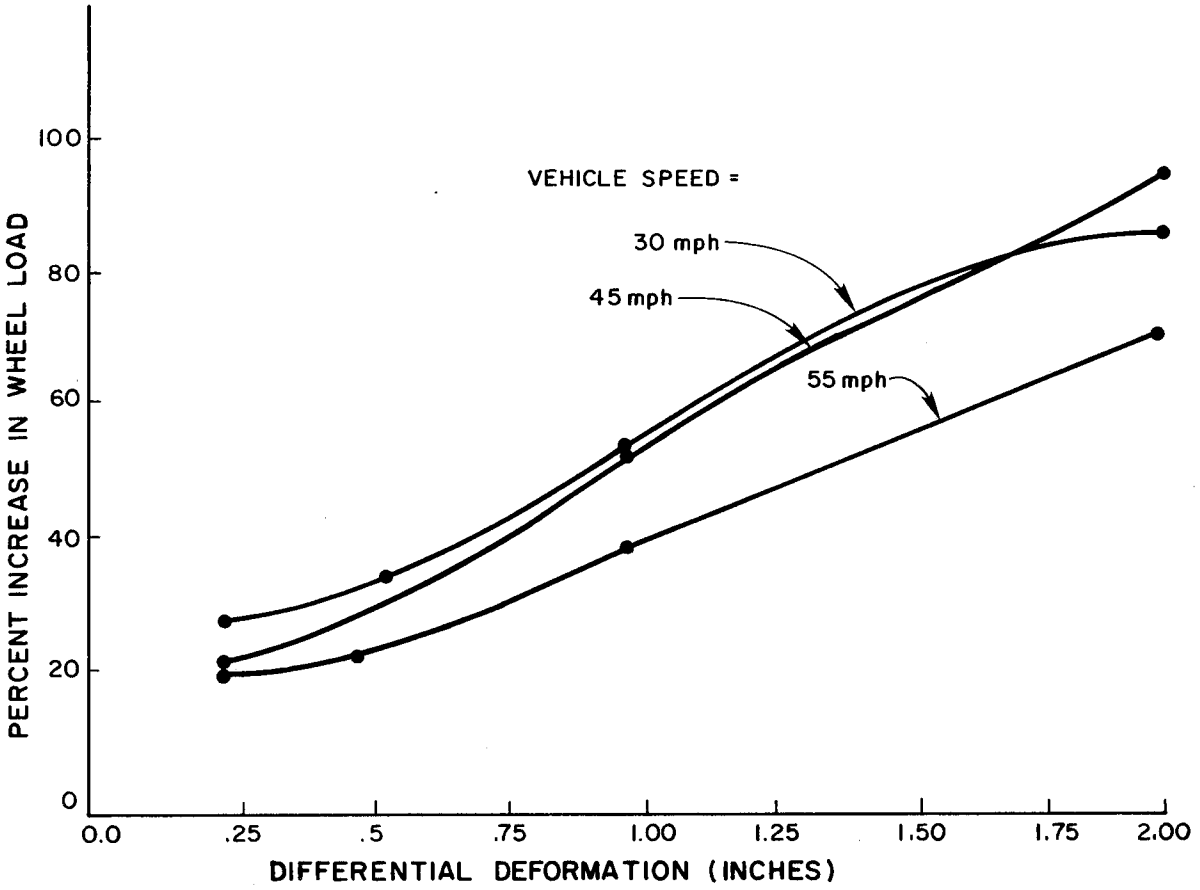


FIGURE 9.- INFLUENCE OF DIFFERENTIAL DEFORMATIONS OF A GRADE CROSSING AND VEHICLE SPEEDS ON DYNAMIC WHEEL LOAD

depends on the choice of the maximum allowable dynamic load experienced by highway traffic over a crossing which will vary with the rideability requirement of a particular crossing.

It was mentioned earlier in this chapter that for highway traffic the dynamic loads above static weight varies from 35 to 42 percent in an average pavement zone (18). Corresponding to this range of dynamic load and assuming a crossing speed of 30 mph (48.3 km/hr) the following limiting design criteria were established:

1. Dynamic Load - from 35% to 42% above static weight
2. Roughness Index - from 420 to 480 in/mile (from Figure 7)
3. Permanent Differential Deformation - from .55 in. to .75 in. (from Figure 9).

Development of Polynomial Stress Equations

Stress calculation in the track structure and the adjacent pavements is an important task in this design sytem, since the permanent differential deformation mainly depends on the magnitude of stresses at different depths and their number of repetitions. Stresses at any point, generally, depend on the geometry of the structure, material properties and the size of loadings. At present, several computer programs are available for the purpose of stress calculation in foundation layers. These programs require long computation times and large memory core and their repeated use in each design problem becomes cumbersome and expensive. In order to have the advantage of being able to calculate stresses at different depths rapidly, polynomial stress equations were developed in this study, separately for track and pavement foundations. These stress equations are of great help to the

design system, considering their simplicity and small computation time. The steps followed in developing these polynomial stress equations are:

1. The variables considered in developing the stress equations are thicknesses of surface and base layers and the moduli of surface, base and subgrade materials. The upper and lower limits of these variables were fixed so that a practical design system would be covered. Table 1 shows the upper and lower limits of these variables for which the equations are considered valid.
2. An experimental design approach was followed to determine the different combinations of these variables (7, 23).
3. Typical railroad track and highway structures as shown in Figures 1 and 2. Their respective wheel loadings and their equivalent representation for computer programs were determined in reference (55). In this study, a single axle 18 kip load for a typical highway loading, and axle loads and wheel spacing corresponding to GE-U-50 locomotive for a typical railway loading were used.
4. Computer program BISTRO (layered elastic pavement analysis program) by the Shell Laboratorium (Koninklijke) was used to calculate stresses in each of the designed structures.
5. The stresses thus obtained were regressed with the original independent variables (moduli and thicknesses) to obtain the polynomial stress equations. A computer program 'Select Regression' developed by Dubose (12) was used for this purpose.

TABLE 1.- UPPER AND LOWER LIMITS OF THE VARIABLES USED IN POLYNOMIAL STRESS EQUATIONS

| | Pavement Structure | | Track Structure | |
|---|---|---|---|---|
| | Upper Limit | Lower Limit | Upper Limit | Lower Limit |
| H1 = Thickness of surface layer, inches | 9 | 3 | 30 | 10 |
| H2 = Thickness of base layer, inches | 14 | 6 | 14 | 6 |
| E1 = Modulus of surface layer, psi | 525000.0 | 75000.0 | 300000.0 | 50000.0 |
| E2 = Modulus of base layer, psi | 115000.0 | 15000.0 | 115000.0 | 15000.0 |
| E3 = Modulus of subgrade, psi | 25000.0 | 5000.0 | 25000.0 | 5000.0 |

Note: Ties and ballast are considered to act together as a composite material in the surface layer of the track structures. The dominant modulus in this composite material is that of the ballast. Further discussion of this point is given on pages 35-37.

The polynomial stress equations for pavement structures are presented below. The coefficients of determination ($R^2$) of these equations vary with depth as shown in Figure 10. Compressive stresses are considered as positive.

Equation 1, for major principal stress in 2nd layer:

$$\sigma_1 = A0 + A1h + A2h^2 \qquad \ldots (1)$$

where:

h = depth in inches from the surface

$$A0 = 1.67219 + H1^{a^0*} E1^{c^0} E2^{d^0}(773.523\ H2^{b^0} - 295.904\ E3^{e^0}) + H1^{a^0} E2^{d^0}(-3.88815\ H2^{b^0} + 2.11753)$$

$$A1 = -.254090 + H1^{a^1} H2^{b^1} E2^{d^1} E3^{e^1}(-112.814\ E1^{c^1} + .197376) + H1^{a^1} E2^{d^1}(5.68131\ E1^{c^1} - .0206118)$$

$$A2 = .00835535 + H1^{a^2} H2^{b^2} E2^{d^2}(17.7337\ E1^{c^2} E3^{e^2} - .0117177) + H1^{a^2} E2^{d^2}(-.243798\ E1^{c^2} + .000723757)$$

Equation 2, for deviator stress in 2nd layer:

$$(\sigma_1 - \sigma_3) = A0 - A1h + A2h^2 \qquad \ldots (2)$$

where:

h = depth in inches from the surface

$$A0 = -2.40236 + H2^{b^0} E2^{d^0} E3^{e^0}(1946.78\ H1^{a^0} E1^{c^0} - 892.3982\ E1^{c^0} - 28.9221\ H1^{a^0}) + E1^{c^0} E2^{d^0}(-751.672\ H1^{a^0} + 396.368\ E3^{e^0}) + E2^{d^0}(11.4648\ H1^{a^0} + 12.0964\ H2^{b^0} - 5.38283)$$

$$A1 = .148485 + H1^{a^1} H2^{b^1} E2^{d^1}(-58.7292\ E1^{c^1} E3^{e^1} + 26.475\ E1^{c^1} - .0125271\ E3^{e^1} + H2^{b^1} E2^{d^1}(-.278655\ E1^{c^1} + .22897\ E3^{e^1} - .0850578)$$

$$A2 = .00790123 + H2^{b^2} E1^{c^2} E2^{d^2}(-1.29101\ H1^{a^2} E3^{e^2} + .39407\ H1^{a^2} + .504433\ E3^{e^2}) + E1^{c^2} E2^{d^2}(-.0685944\ H2^{b^2} - .00379577\ H1^{a^2} + .000558257)$$

* Values of exponents are shown in Table 2 .

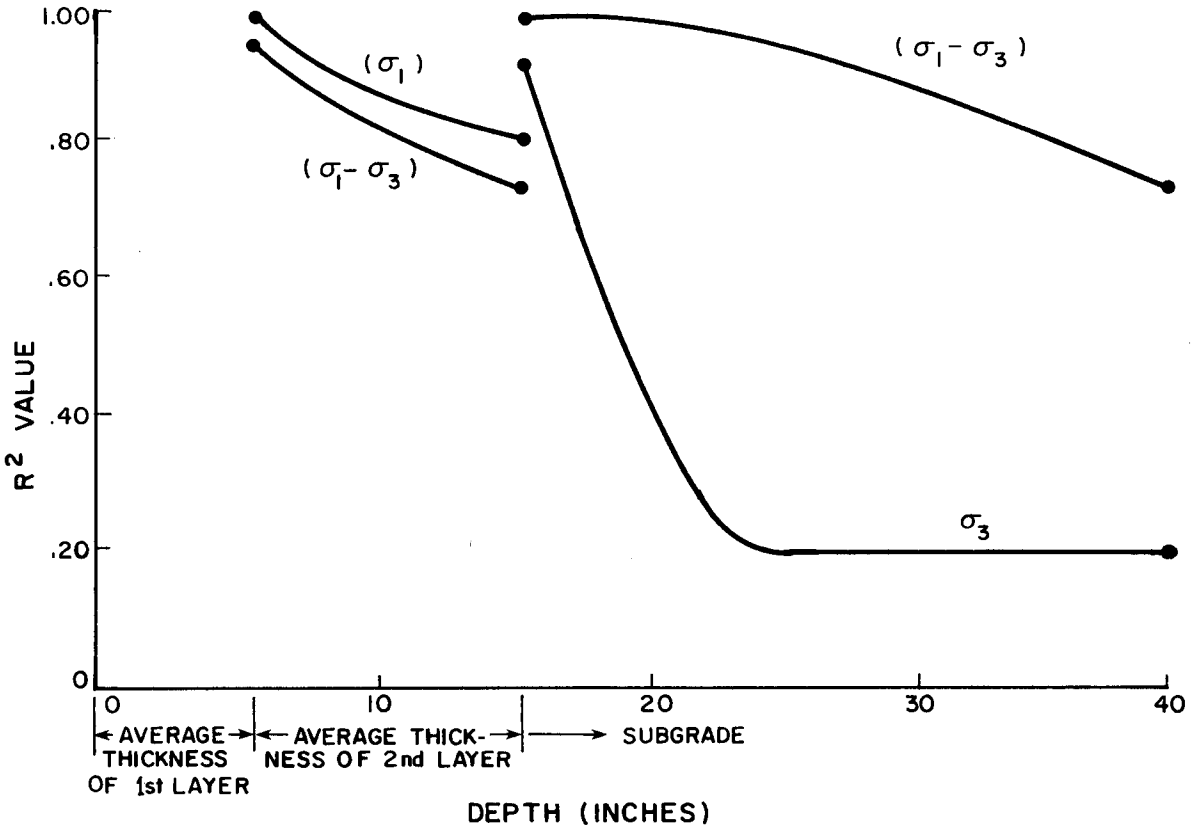


FIGURE 10.- COEFFICIENTS OF DETERMINATION ($R^2$) OF POLYNOMIAL STRESS EQUATIONS FOR HIGHWAY PAVEMENTS

TABLE 2.- VALUES OF EXPONENTS USED IN POLYNOMIAL STRESS EQUATIONS

| Equation No. | $a^0$ | $b^0$ | $c^0$ | $d^0$ | $e^0$ | $a^1$ | $b^1$ | $c^1$ | $d^1$ | $e^1$ | $a^2$ | $b^2$ | $c^2$ | $d^2$ | $e^2$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -.706 | -.271 | -.392 | .487 | .0076 | -1.068 | -.728 | -.428 | .642 | -.067 | -1.45 | -1.22 | -.446 | .685 | -.08 |
| 2 | -.383 | -.377 | -.327 | .529 | -.012 | -.61 | -.869 | -.369 | .733 | -.1 | -.98 | -1.521 | -.356 | .905 | -.239 |
| 3 | -.61 | -.706 | -.254 | -.184 | .25 | -.799 | -.905 | -.289 | -.24 | .34 | -.918 | -1.03 | -.309 | -.272 | .392 |
| 4 | -.545 | -.465 | -.222 | -.233 | .645 | -.666 | -.6 | -.255 | -.249 | .759 | -.738 | -.692 | -.272 | -.248 | .809 |
| 5 | -.221 | -.057 | -.131 | .046 | .094 | -.829 | -.049 | -.129 | .039 | .065 | -1.77 | -.075 | -.025 | -.013 | .025 |
| 6 | -5.135 | -1.83 | 1.01 | .655 | .23 | | | | | | | | | | |
| 7 | .522 | -1.385 | .250 | 1.155 | -.186 | -.434 | -1.363 | -.156 | 1.299 | -.325 | -1.59 | -1.368 | -.345 | 1.456 | -.484 |
| 8 | -.859 | -.504 | -.285 | .65 | -.08 | -1.482 | -.847 | .325 | .855 | -.168 | -2.152 | -1.153 | -.357 | 1.025 | -.277 |
| 9 | -.98 | -.433 | -.344 | -.028 | .151 | -1.367 | -.571 | -.432 | -.048 | .232 | | | | | |
| 10 | -.6 | -.145 | -.318 | -.106 | .684 | -.667 | -.198 | -.319 | -.115 | .817 | -.711 | -.26 | -.288 | -.11 | .855 |

From Equations 1 and 2, an equation for minor principal stress in the 2nd layer is calculated as follows:

$$\sigma_3 = \sigma_1 - (\sigma_1 - \sigma_3)$$

Equation 3, for minor principal stress in subgrade:

$$\sigma_3 = A0 - A1h + A2h^2 \qquad \ldots (3)$$

where:

h = depth in inches from the surface

$$A0 = .233285 + H1^{a^0} H2^{b^0} E3^{e^0} (41.011\ E1^{c^0} + 108.767\ E2^{d^0} - 14.6181) + H1^{a^0} H2^{b^0} E1^{c^0} (-24986.4\ E2^{d^0} + 3295.44)$$

$$A1 = .013372 + H1^{a^1} H2^{b^1} (-4421.43\ E1^{c^1} E2^{d^1} + 5.32534\ E2^{d^1} E3^{e^1} - .380767\ E3^{e^1} + 366.521\ E1^{c^1}) + H1^{a^1} (.00553385\ E3^{e^1} - .200216)$$

$$A2 = .00011311 + H1^{a^2} H2^{b^2} E3^{e^2} (-2.35541\ E1^{c^2} E2^{d^2} + .13664\ E1^{c^2} + .107941\ E2^{d^2} - .00546569) + H1^{a^2} H2^{b^2} (-2.14616\ E2^{d^2} + .110538) - .000540568\ H1^{a^2}$$

Equation 4, for deviator stress in subgrade:

$$(\sigma_1 - \sigma_3) = A0 - A1h + A2h^2 \qquad \ldots (4)$$

where:

h = depth in inches from the surface

$$A0 = .339135 + E1^{c^0} E3^{e^0} (52.3428\ H1^{a^0} E2^{d^0} - 21.6485\ H1^{a^0} H2^{b^0} + 57.9236\ H2^{b^0} E2^{d^0}) + E3^{e^0} (.711581\ H1^{a^0} H2^{b^0} - 2.29282\ E2^{d^0} + .0807088)$$

$$A1 = -.0013966 + H1^{a^1} H2^{b^1} E3^{e^1} (7.37209\ E1^{c^1} E2^{d^1} - .379944\ E2^{d^1} - .436305\ E1^{c^1}) + H1^{a^1} E3^{e^1} (.0178911\ H2^{b^1} + .0252902\ E2^{d^1}) + E3^{e^1} (.00192404\ H2^{b^1} - .000487605)$$

$$A2 = -.0000109923 + H1^{a^2} H2^{b^2} E3^{e^2} (.0635893\ E1^{c^2} E2^{d^2} - .00383969\ E1^{c^2} - .00256485\ E2^{d^2}) + H1^{a^2} E3^{e^2} (.000127151\ H2^{b^2} + .000122254\ E2^{d^2}) + E3^{e^2} (.00001019\ H2^{b^2} - .00000209092)$$

From Equations 3 and 4, an equation for the sum of principal stresses in the subgrade is calculated as follows assuming that $\sigma_2 = \sigma_3$:

$$(\sigma_1 + \sigma_2 + \sigma_3) = (\sigma_1 - \sigma_3) + 3\ \sigma_3$$

The polynomial stress equations for railroad structures are presented below. The coefficients of determination ($R^2$) for these equations vary with depth as shown in Figure 11. Compressive stresses are considered as positive.

Equation 5, for deviator stress in the top layer (ballast material):

$$(\sigma_1 - \sigma_3) = A0 + A1h + A2h^2 \qquad \ldots (5)$$

where:

h = depth in inches from ballast surface

$$A0 = 6.90067 + H1^{a^0} E1^{c^0} E2^{d^0} E3^{e^0} (53278.1\ H2^{b^0} - 46431.5) + H1^{a^0} E1^{c^0} E2^{d^0} (-130145.0\ H2^{b^0} + 113842.0) + 116.334\ H1^{a^0}$$

$$A1 = -.303037 + H1^{a^1} E1^{c^1} E2^{d^1} E3^{e^1} (-131815.0\ H2^{b^1} + 117753.0) + H1^{a^1} E1^{c^1} E2^{d^1} (246473.0\ H2^{b^1} - 220326.0) - 133.453\ H1^{a^1}$$

$$A2 = .0443057 + H1^{a^2} H2^{b^2} E2^{d^2} (-348002.0\ E1^{c^2} + 202822.0\ E3^{e^2}) + H1^{a^2} H2^{b^2} (301938.0\ E1^{c^2} - 175891.0\ E3^{e^2})$$

Equation 6, for minor principal stress at midpoint of top layer (ballast material):

$$\sigma_3 = C1 + C2 \qquad \ldots (6)$$

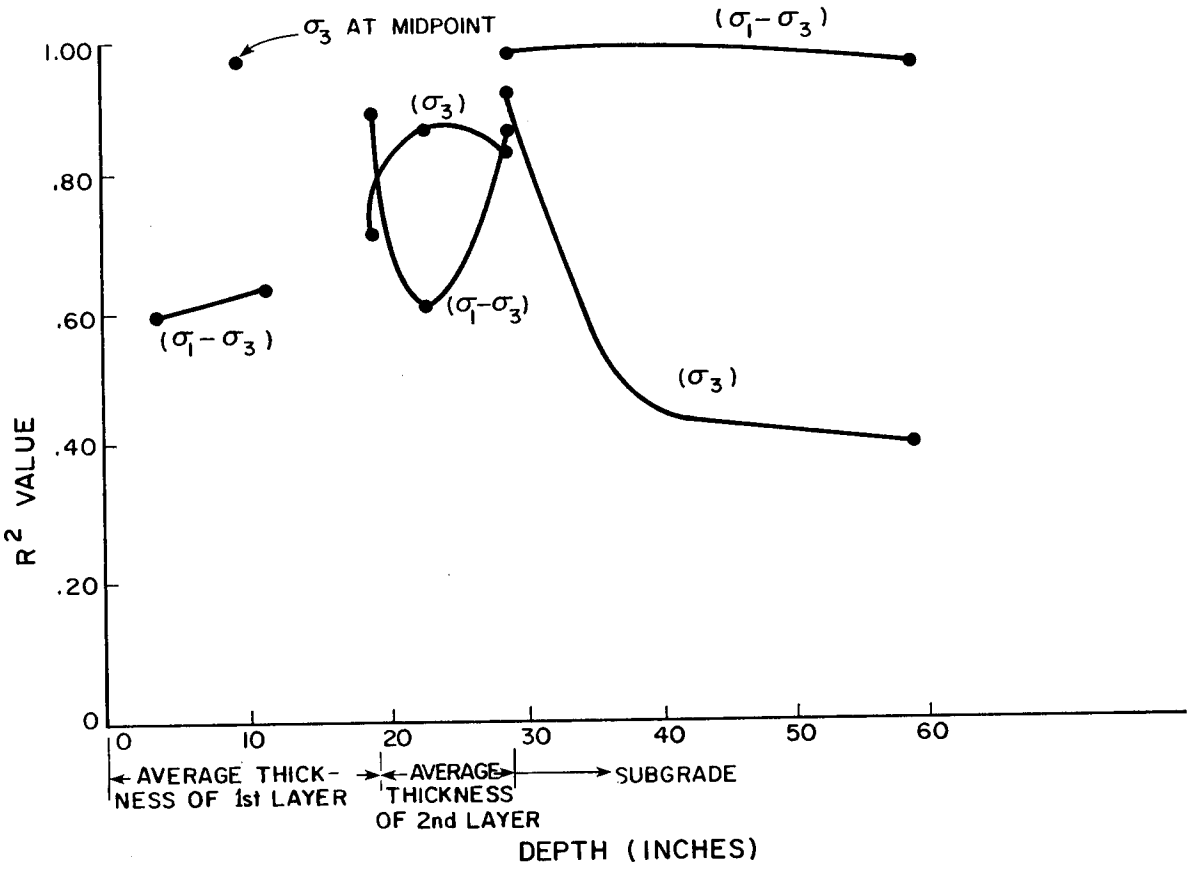


FIGURE 11.- COEFFICIENTS OF DETERMINATION ($R^2$) OF POLYNOMIAL STRESS EQUATIONS FOR RAILROAD STRUCTURE

where:

$$C1 = .664423 + H1^{a^0} H2^{b^0} E1^{c^0} E2^{d^0} (-12.8779\ E3^{e^0} + 77.5752) + H1^{a^0} E1^{c^0} E3^{e^0} (7724.05\ H2^{b^0} + .0988775\ E2^{d^0})$$

$$C2 = H1^{a^0} H2^{b^0} (-31632.5\ E1^{c^0} + 3824180.0\ E2^{d^0} - 1974900000.0) + H1^{a^0} E1^{c^0} (-.544612\ E2^{d^0} - 256.957) - 46242800.0\ H1^{a^0}$$

Equation 7, for minor principal stress in 2nd layer:

$$\sigma_3 = -(A0 + A1h + A2h^2) \qquad \ldots (7)$$

where:

h = depth in inches from ballast surface

$$A0 = 1.78920 + H1^{a^0} H2^{b^0} E2^{d^0} (-.00536975\ E1^{c^0} E3^{e^0} + .000830849\ E1^{c^0} + .128764\ E3^{e^0} - .0200543) + H2^{b^0} E2^{d^0} (.0243088\ E1^{c^0} E3^{e^0} - .00372796\ E1^{c^0} - .580973\ E3^{e^0} + .090371)$$

$$A1 = -.111305 + H2^{b^1} E2^{d^1} E3^{e^1} (.0774579\ H1^{a^1} E1^{c^1} - .000703219\ H1^{a^1} - .0214913\ E1^{c^1}) + H2^{b^1} E2^{d^1} (-.000551022\ H1^{a^1} + .000150856)$$

$$A2 = .00714227 + H1^{a^2} E2^{d^2} E3^{e^2} (.163560\ H2^{b^2} E1^{c^2} - .00176703\ H2^{b^2} - .00626615\ E1^{c^2}) + H1^{a^2} E2^{d^2} (.0000921984\ E3^{e^2} + .00000532471\ E1^{c^2}) - .00000000107315\ E2^{d^2}$$

Equation 8, for deviator stress in 2nd layer:

$$(\sigma_1 - \sigma_3) = A0 + A1h + A2h^2 \qquad \ldots (8)$$

where:

h = depth in inches from ballast surface

$$A0 = 5.23725 + H1^{a^0} E2^{d^0} E3^{e^0} (516.472\ H2^{b^0} E1^{c^0} - 137.697\ E1^{c^0} - 13.0602\ H2^{b^0} + 4.41353) - .00706606\ E2^{d^0}$$

$$A1 = -.389912 + H1^{a^1} E2^{d^1} E3^{e^1} (.0592021\ H2^{b^1} E1^{c^1} - 3.75471\ H2^{b^1} - .0067074\ E1^{c^1} + .343391) + .0000497097\ E2^{d^1}$$

$$A2 = -.00379223 + H1^{a^2} E2^{d^2} E3^{e^2} (17.8758\ H2^{b^2} E1^{c^2} - .14552\ H2^{b^2} - .88369\ E1^{c^2} + .0108234) + 5.27254\ H1^{a^2}$$

From Equations 7 and 8, an equation for the sum of principal stresses in the 2nd layer is calculated as follows assuming that $\sigma_2 = \sigma_3$:

$$(\sigma_1 + \sigma_2 + \sigma_3) = (\sigma_1 - \sigma_3) + 3\ \sigma_3$$

Equation 9, for minor principal stress in subgrade:

$$\sigma_3 = A0 + A1h \qquad \ldots (9)$$

where:

h = depth in inches from ballast surface

$$A0 = .229435 + H1^{a^0} E1^{c^0} E2^{d^0} (-331576.0\ H2^{b^0} + 29371.8\ E3^{e^0}) + H1^{a^0} E1^{c^0} (245758.\ H2^{b^0} - 21436.1\ E3^{e^0}) - 7.97399\ H1^{a^0}$$

$$A1 = -.00315258 + H1^{a^1} E1^{c^1} E2^{d^1} (37214.9\ H2^{b^1} - 1118.31\ E3^{e^1}) + H1^{a^1} E1^{c^1} (-22232.2\ H2^{b^1} + 651.612\ E3^{e^1}) + .342981\ H1^{a^1}$$

Equation 10, for deviator stress in subgrade:

$$(\sigma_1 - \sigma_3) = A0 + A1h + A2h^2 \qquad \ldots (10)$$

where:

h = depth in inches from ballast surface

$$A0 = -.135338 + H1^{a^0} H2^{b^0} E3^{e^0} (134.041\ E1^{c^0} E2^{d^0} - 45.5381\ E1^{c^0} - 2.71228\ E2^{d^0} + 1.00933) + E3^{e^0} (.85696\ E1^{c^0} - .0195183)$$

$$A1 = .00375682 + H1^{a^1} H2^{b^1} E3^{e^1} (-1.46644\ E1^{c^1} E2^{d^1} + .437029\ E1^{c^1} + .0301024\ E2^{d^1} - .00963337) + E3^{e^1} (-.00412002\ E1^{c^1} + .0000975604)$$

$$A2 = -.000019561 + H1^{a^2} H2^{b^2} E3^{e^2} (.00566798\ E1^{c^2} E2^{d^2} - .00160626\ E1^{c^2} - .000170594\ E2^{d^2} + .0000518243) - .0000000489157\ E3^{e^2}$$

From Equations 9 and 10, an equation for the sum of principal stresses in the subgrade is calculated as follows assuming $\sigma_2 = \sigma_3$:

$$(\sigma_1 + \sigma_2 + \sigma_3) = (\sigma_1 - \sigma_3) + 3\,\sigma_3$$

Influence of Dynamic Effect on Stresses

Irregularities in the riding surface interact with vehicle characteristics and vehicle speeds to induce dynamic effects in vehicle loadings which may increase or decrease their static weight at a particular location. For highway traffic a grade crossing is a source of surface irregularities because of geometrics and construction complexities. It has been observed that dynamic loads are produced by train movements at a higher speed (41), even though the rail surface is essentially smooth under train wheel loads. Therefore, it is important to include the dynamic effect for both highway and railway traffic in the stress calculations.

Highway Traffic.- It was stated earlier in this chapter that for highway traffic the dynamic load above static weight varies from 35 to 42 percent in an average pavement. This range was considered as the limiting value in this design. A newly constructed grade crossing surface provides a smoother riding surface, and hence, lower dynamic effects. The limiting value of the dynamic load will be reached with time and number of load applications. Therefore, an average increase of 20% above static stresses due to the dynamic effect on the pavements adjacent to the grade crossing was used in this design. Also, as illustrated in Figure 4, it was found that the increase in the dyanmic load over a grade crossing was approximately half of that on the adjacent pavement surface. Therefore, a 10% increase of static stresses due to the dynamic effect of highway loads on the grade

crossing surface was incorporated in this design.

Railway Traffic.- Slight imperfection in the rail surface or wheel roundness and lateral movement of the train, etc. create dynamic loads which increase with train speed. Centrifugal forces, superelevation, turning forces, etc. increase the dynamic effect on curves. Talbot, et al. (41) measured rail stresses for various locomotives and trains at varying speeds. The ratios of stresses at several speeds with respect to stress at 5 mph are shown in Figure 12. The effect of curvature is apparent. A 15 percent increase in rail stress (straight track, 60 mph) was selected from the figure to account for dynamic effects. This value was used in design computations.

## Design Technique

The stress equations presented in this chapter and the material characterization described in the following chapter involve many terms, therefore, a computer program was written to calculate the stresses at different depths of railroad track and adjacent pavement structures. Permanent deformations in each structure are calculated as a function of these stresses, the deformation characteristics of the materials, and the number of repetitions of wheel loads applied in a design period are calculated. The difference in these deformations (permanent differential deformation) serves as the design criterion. If this difference in deformations exceeds the permissible maximum limit (established earlier considering rideability need), layer thicknesses and their material properties are revised and the whole analysis is repeated to estimate the new values of the differential deformation. This is again compared with the maximum permissible limit. This process

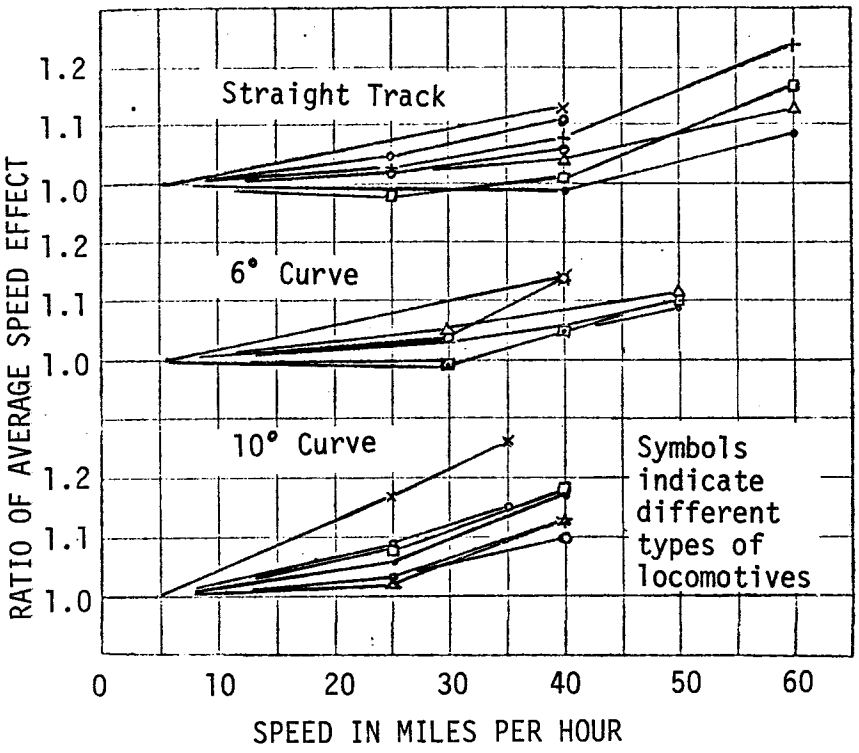


FIGURE 12.- RATE OF INCREASE IN STRESS IN RAIL IN TERMS OF THE STRESS AT 5 MILES PER HOUR (41)

continues until a suitable design is obtained. Figure 13 shows the flow chart of the design system.

## Summary

This chapter contained a detailed description of the design system for highway-railroad grade crossings developed in this study. Three design criteria were considered: 1) dynamic load profile, 2) roughness index and 3) permanent differential deformation. Critical values of these design criteria were presented, and development of polynomial stress equations was described. The dynamic effect on stress calculations is included and finally the design technique is presented including a flow chart of the total design system.

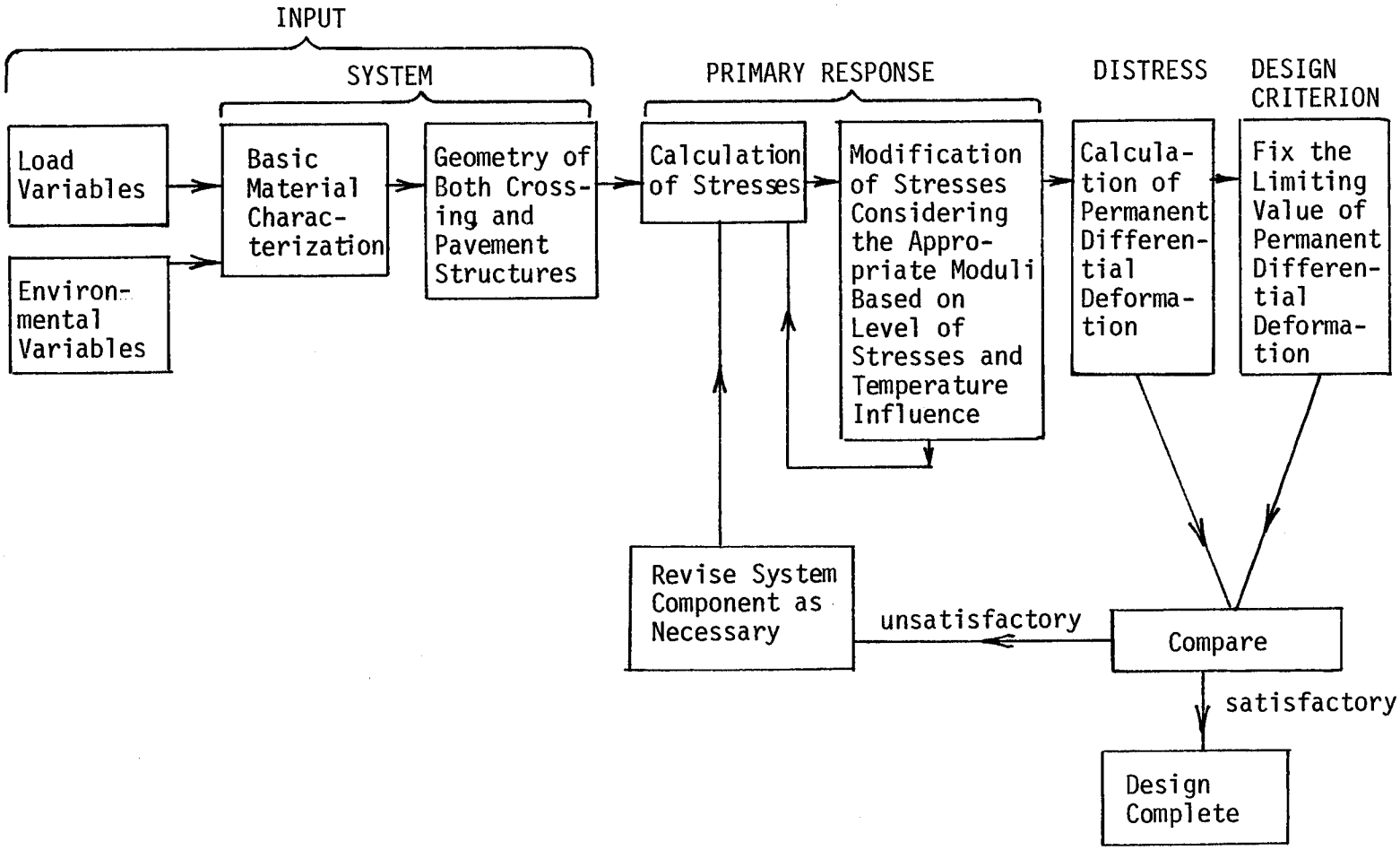


FIGURE 13.- FLOW CHART OF GRADE CROSSING DESIGN SYSTEM

# CHAPTER III

## MATERIAL CHARACTERIZATION FOR DESIGN APPLICATION

The analysis and design procedures for pavements as well as for grade crossings are fundamentally based on the determination of primary response variables such as stresses, strains and deflections at different locations in these structures. These variables are determined through the formulation and solution of boundary value problems. In formulating the governing differential equation, properties of the various materials which comprise the structure are considered in the form of constitutive equations. Generally these constitutive equations describe the stress-strain relationship of the materials. In a complete design system, however, determination of primary response variables is not sufficient in itself, it is also necessary to establish limiting (failure) criteria in terms of these variables for the loading and environmental conditions. In this study, excessive permanent deformation due to repetition of loads control the failure criterion as was discussed in the previous chapter. In this chapter, material characterization will mean both the selection of the constitutive properties and failure criteria for a material. The material characterizations that are used in this design procedure are discussed in the rest of this chapter.

### Surface Layer Materials

A surface layer in a pavement or in a railroad structure is used mainly to 1) withstand the displacement and abrasive forces associated with contact wheel loads, 2) deliver wheel loads over an area larger than the contact area, thereby decreasing the intensity of stress in the base course and subgrade, 3) withstand deformations caused by wheel loads and temperature changes without developing serious cracks and 4) withstand fatigue stresses

caused by wheel load repetitions. In a typical railroad track ties and ballast are combined to form a surface layer which rests under the rails. In a flexible pavement system asphalt concrete is used as the surface layer. The detailed description of ties, ballast and asphalt concrete layers and their material characteristics are given below.

Ties.- Ties perform two main functions in a railroad track:

1) hold two lines of rails transversely to correct gage and

2) transmit the train load to the ballast material with a diminishing stress intensity. Different kinds of ties such as timber, concrete, steel, etc. are used in this country; among them timber ties are used more frequently. Typical dimensions of timber ties are 9 in. x 7 in. in cross-section and 8½ to 9 ft. in length. These are usually placed at 19 in. - 22 in. apart from center to center.

In early analyses, it was assumed that the pressure distribution due to train loads is uniform across the length and width of the tie. However, experiments conducted first by Cuenot in France and more recently by Talbot (39) in this country proved this assumption to be erroneous. Actually, a loaded tie in a freshly tamped ballast material behaves like a beam with the supporting reactions concentrated under the points of application of loads (the points under the rails) with a portion overhanging the support on both sides. Figure 14 shows the composite depression curves as developed by Talbot from his tests of tie pressure in freshly tamped high-grade ballast. From this figure it can be seen that the maximum downward deflection of the tie occurs right below the outer edge of the rails which indicates that the concentration of pressure distribution is also developed in the same region. Pressure intensity along the length of the tie decreases as the distance from the rail increases and becomes a minimum at the midpoint and at both the ends. However, in this study, it was assumed that the total wheel load is transmitted by the tie on to the ballast through an area directly under the rail and the dimension of the area is equal to that of a tie plate.

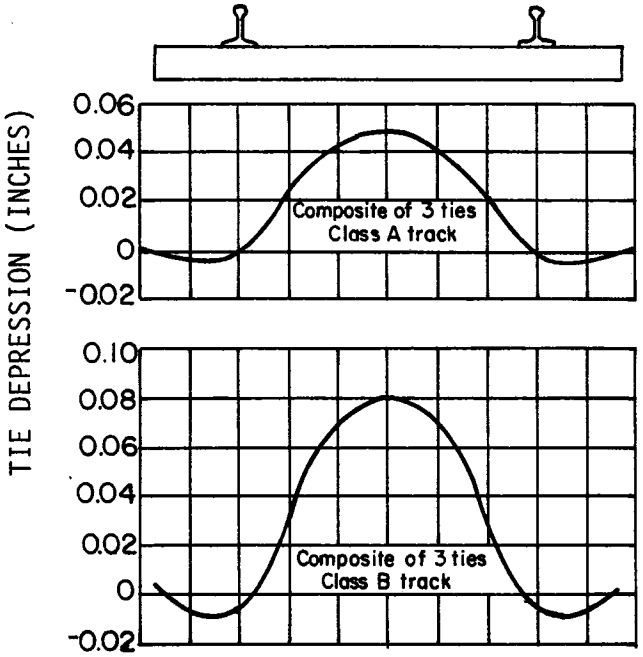


FIGURE 14.- COMPOSITE CURVES OF TIE DEPRESSION (39)

The modulus of elasticity of timber ties is generally very high (1,500,000 to 2,000,000 psi) compared to those of the other layers such as ballast, base course and subgrade. It is believed that the deformation within the tie itself is very small compared to that of other layers and hence it was not accounted for in this study. Therefore no material characterization of ties is considered here.

Ballast.- In a railroad track ballast is used to perform the following main functions:

1) transfers the applied load from the ties and uniformly distributes the load over the road bed
2) anchors the track both laterally and longitudinally
3) provides immediate drainage in the track structure
4) reduces the frost heaving of the track
5) facilitates the maintenance operations
6) retards the growth of vegetation in the track
7) provides some degree of resilience which absorbs some of the shock from dynamic impact.

A variety of ballast materials are available, among the most commonly used ones are crushed stone (limestone or granite), crushed slag, prepared gravel, pit run gravel, cinders, etc. The sizes of the ballast material generally depend on the types of loading that a track carries. Larger sizes up to 3 in. offer more resistance against crushing and hold the line and surface better. Large size ballast materials are used in heavily loaded freight lines. However, the flatter edges and angles on large particles do not grip the ties as well as smaller sizes. With large size ballast, it is also difficult to lift the rail by a small amount for surfacing work. Considering all these, the most preferred sizes of ballast materials seem to range from 3/4 in. to 1½ in. This allows a small lift and gives a more finished line and surface.

No adequate research to characterize ballast material has been conducted up to the present time. However, most recently Gaskin et al. published some work on the selection of railroad

ballast (19). In arriving at their conclusions they performed a limited number of repetitive load tests in an attempt to characterize the material. However, more work is needed in this area to develop a representative mathematical model. Hargis (22) studied the strain characteristics of limestone gravel, typically used as base course material in Texas. Since this material falls quite close to the ballast material, it was decided to use this as the representative sample for ballast and its characteristic properties were used in this study.

Hargis (22) tested several limestone gravel specimens and obtained data to calculate the modulus of elasticity of the material. He plotted modulus values versus confining stresses on a linear scale. He found that the modulus of limestone gravel increases with the increase of confining stress. It was also observed from his data that the modulus value generally increases with the increase in the number of load repetitions and the magnitude of such load. He fitted the data on each plot with the best-fit straight line. A typical plot relating modulus of elasticity with the confining stress for limestone gravel is shown in Figure 15. In this study the constitutive relation for ballast material is considered as linearly elastic, i.e., modulus value is independent of stress. The magnitude of modulus of elasticity varies from 50,000 psi to 300,000 psi depending upon the condition and degree of compaction of the ballast layer.

Again from the repetitive load test data, Hargis developed a regression model to predict the permanent deformation in the material caused by an arbitrary stress level and number of repetition of such stresses. The regression model is shown below:

$$\log_{10}\varepsilon^{p} = -1.8688 + 0.1666 \log_{10}N + 2.4048\ R$$

where

$\varepsilon^{p}$ = permanent deformation, in percent

$$R = \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f}$$

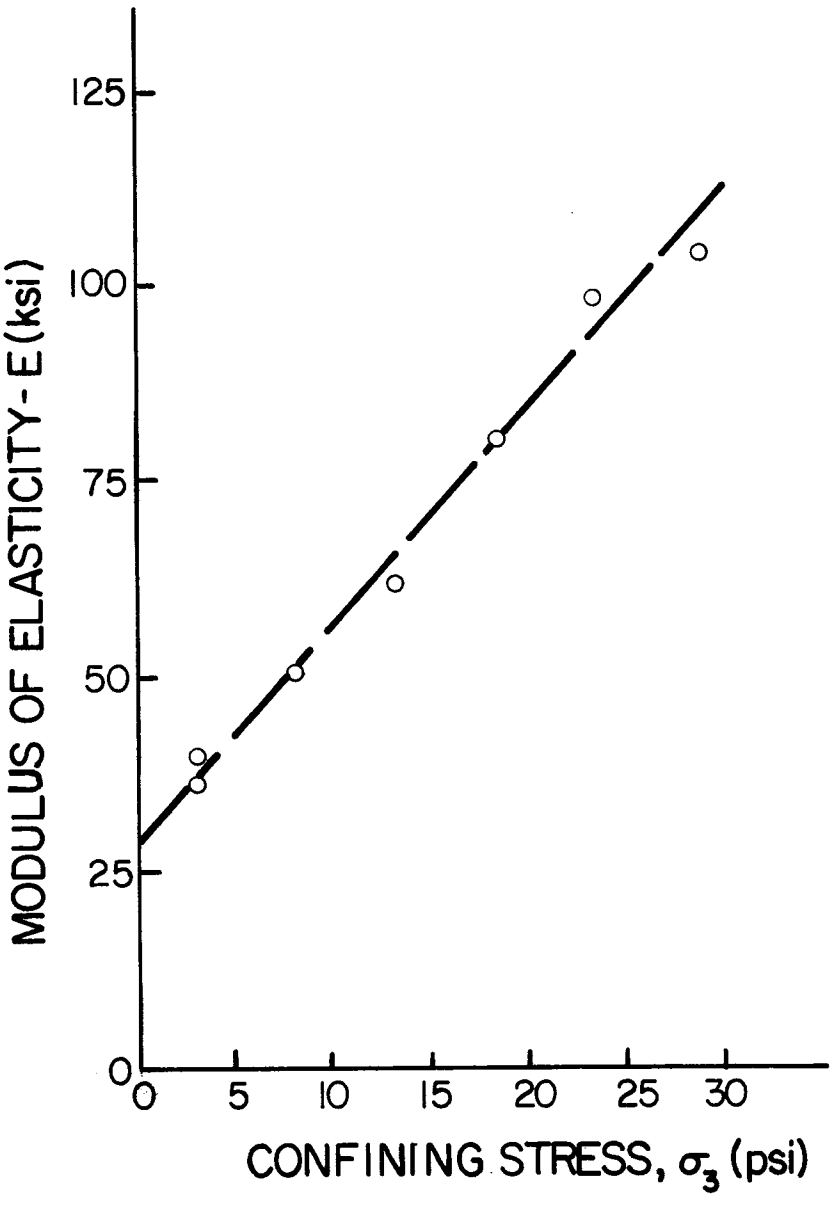


FIGURE 15.- TYPICAL PLOT OF MODULUS OF ELASTICITY VERSUS CONFINING STRESS FOR LIMESTONE GRAVEL (22)

$(\sigma_1-\sigma_3)$ = applied deviator stress

$(\sigma_1-\sigma_3)_f$= deviator stress at failure

N = number of load repetitions.

Figure 16 shows a comparison between predicted and measured permanent strain in limestone gravel. Figure 17 shows a relationship between deviator stress at failure and the confining stress.

Asphalt Concrete.- This is a mixture of mineral aggregates and asphalt used for surface layer in a flexible pavement. The aggregate is generally crushed stone, crushed slag or crushed gravel with sand and filler added.

The modulus of asphalt concrete is temperature dependent. A typical plot of temperature versus modulus of asphalt concrete is shown in Figure 18. It is important to use the modulus value which corresponds with the actual temperature, since a variation in modulus will greatly influence the level of stresses in the structure. The permanent deformation in the asphalt concrete layer of the adjacent pavement is believed to be very small compared to that in other layers and therefore this is not included in the design procedure. A typical failure in an asphalt concrete layer is caused by fatigue. However, only permanent deformation due to repetition of load is considered in this study as the design criterion.

## Base Course Materials

In pavements as well as in railroad tracks granular materials (treated or untreated) are most commonly used in the base course layer. Various kinds of such materials are in use at present and they vary widely in their aggregate types, gradation and in their constitutive representations. Comprehensive studies on these materials were conducted by Smith et al. (37), Hicks (26), Barksdale (4) and many others. Typical granular (treated and untreated) materials are discussed below.

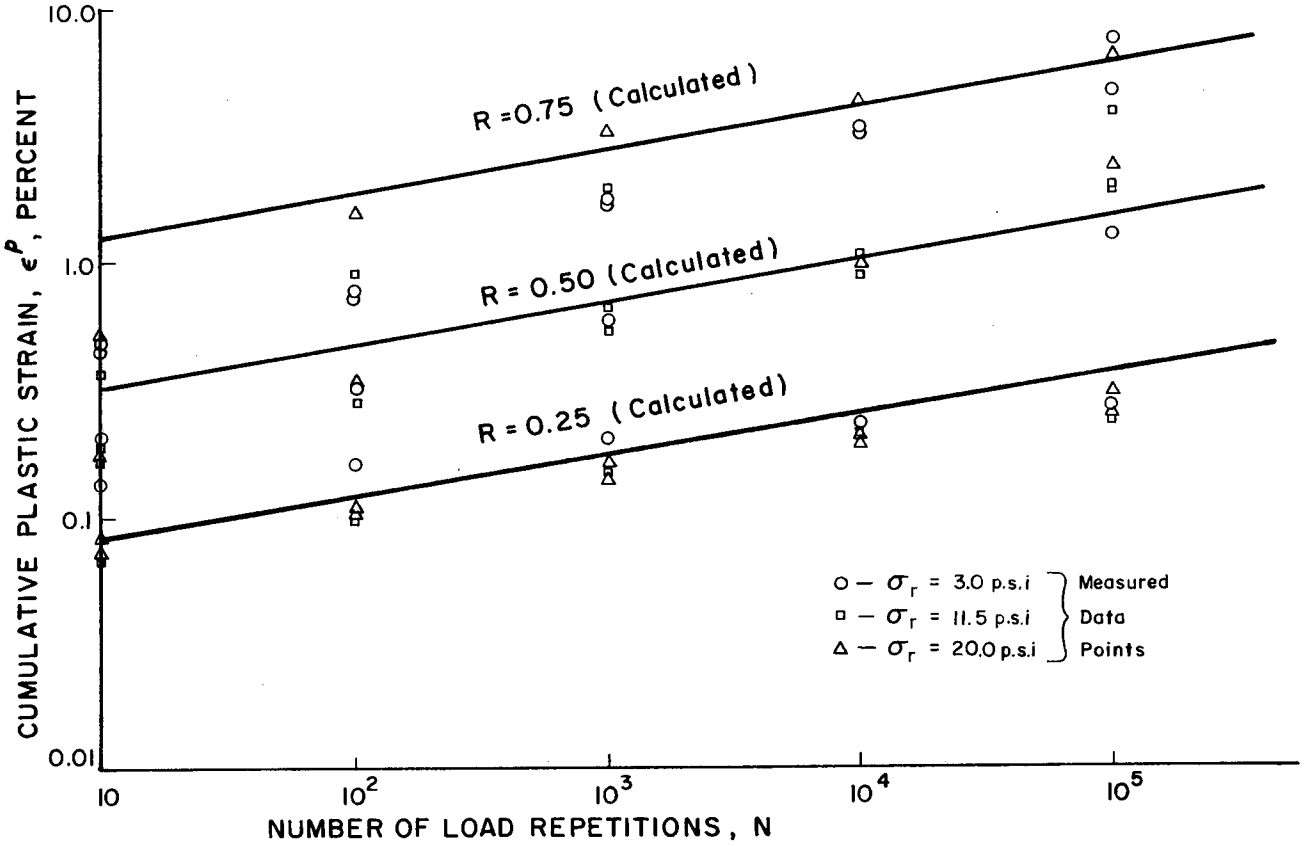


FIGURE 16.- COMPARISON BETWEEN PREDICTED AND MEASURED PERMANENT STRAIN IN LIMESTONE GRAVEL (22)

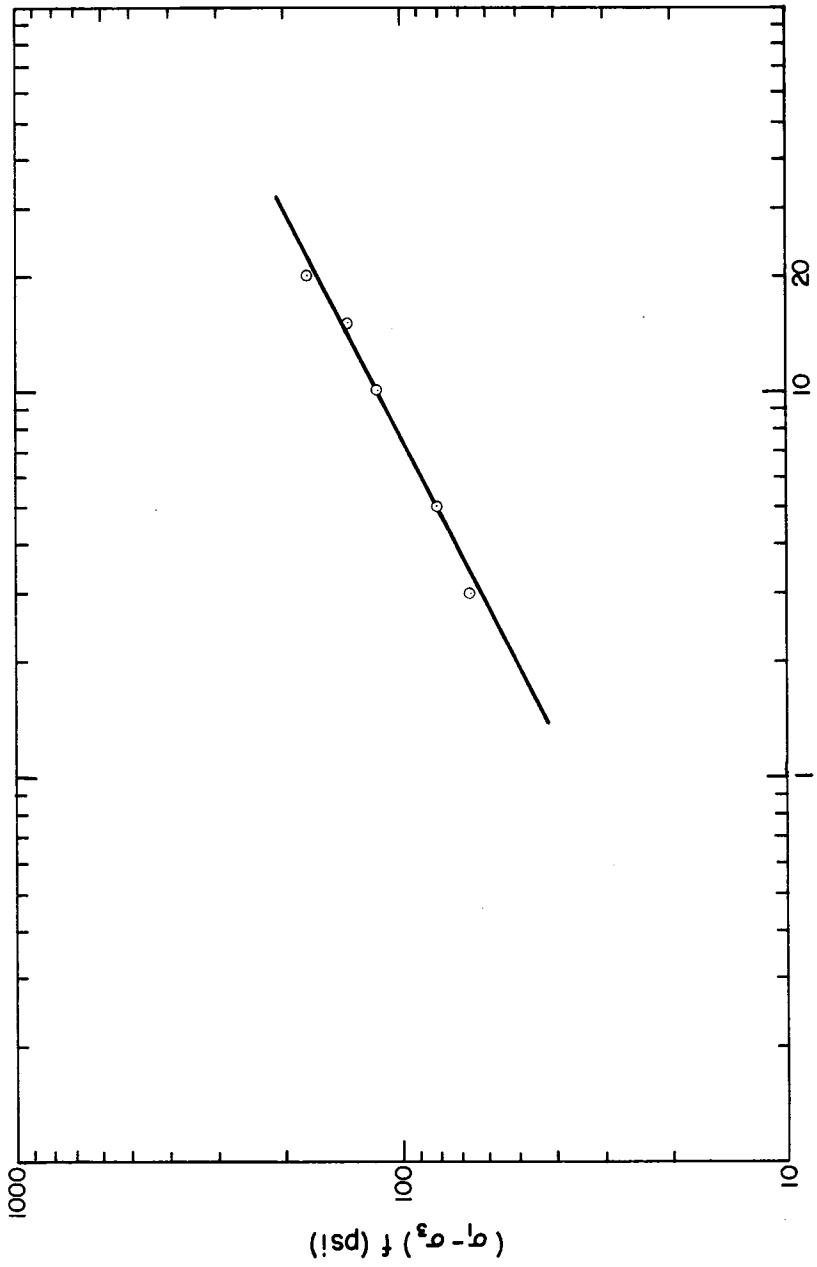


FIGURE 17.- RELATIONSHIP BETWEEN DEVIATOR STRESS AT FAILURE AND CONFINING STRESS OF LIMESTONE GRAVEL (22)

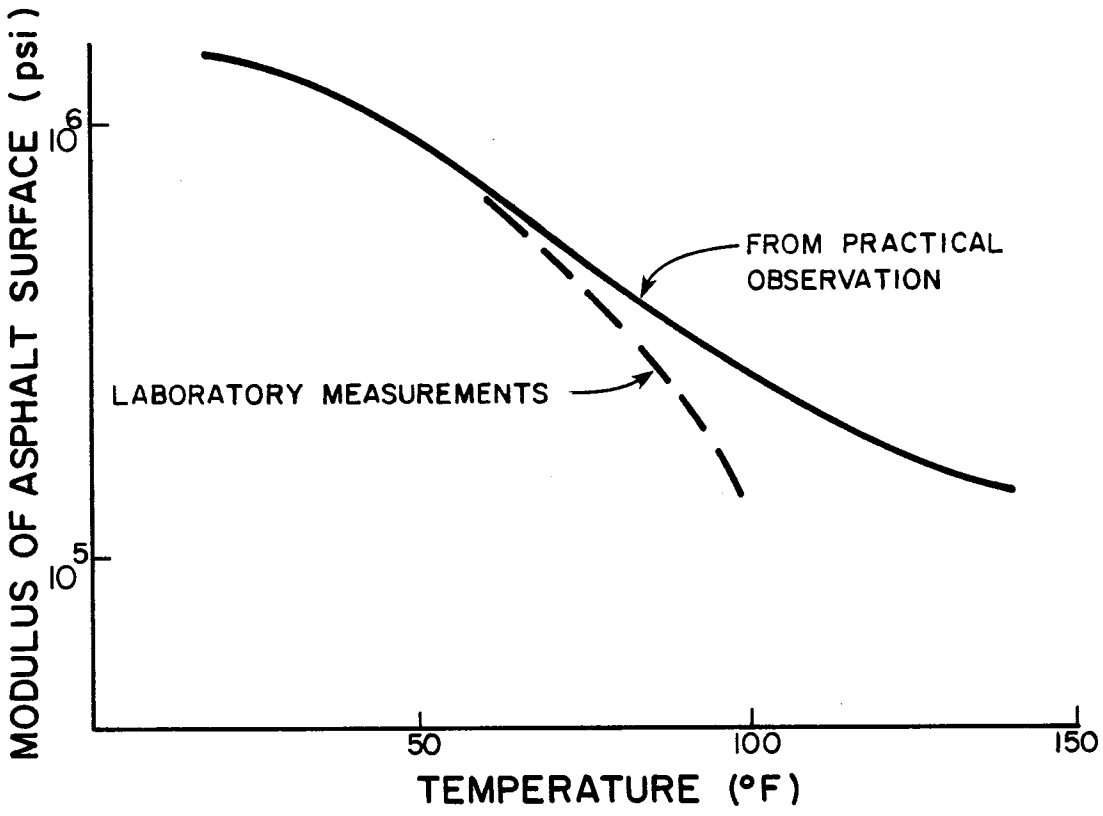


FIGURE 18.- TYPICAL PLOT OF MODULUS OF ASPHALT CONCRETE VERSUS TEMPERATURE (37)

Untreated Granular Base Course Material.- In order to examine whether untreated granular material should be characterized by elastic or viscoelastic theory, Hicks (26) studied the time dependency of these materials. He imposed stresses on these materials over a variety of times of duration and found no significant influence on their resilient modulus values and Poisson's ratios. Creep test results on untreated granular base and subbase materials used at the AASHO Road Test reported by Coffman et al. (8) showed no indication of time dependency in their characteristic properties. All the above studies clearly indicate that constitutive representations of granular base course (untreated) materials can be adequately expressed by elastic theory. These studies also indicate that the resilient modulus of these materials are stress dependent, especially on confining stress. Dunlap (16), Mitry (31), Seed et al. (36) and Barksdale (4) reported significant increases in resilient modulus in granular materials with an increase in confining stress. This nonlinear relationship between confining stress and resilient modulus is commonly represented in the following form:

$$M_R = K \sigma_3^n$$

where

$M_R$ = resilient modulus

$\sigma_3$ = confining stress

K and n = regression constants.

Figure 19 shows the variation of K and n for different granular materials with their relative densities (26). Others related the resilient modulus to the sum of principal stresses $\theta=(\sigma_1+\sigma_2+\sigma_3)$ as follows

$$M_R = K' \theta^{n'}$$

where K' and n' = regression constants.

Figures 20 and 21 show the influence of confining stress and sum of principal stresses respectively on the resilient modulus for a typical untreated granular material. Values of

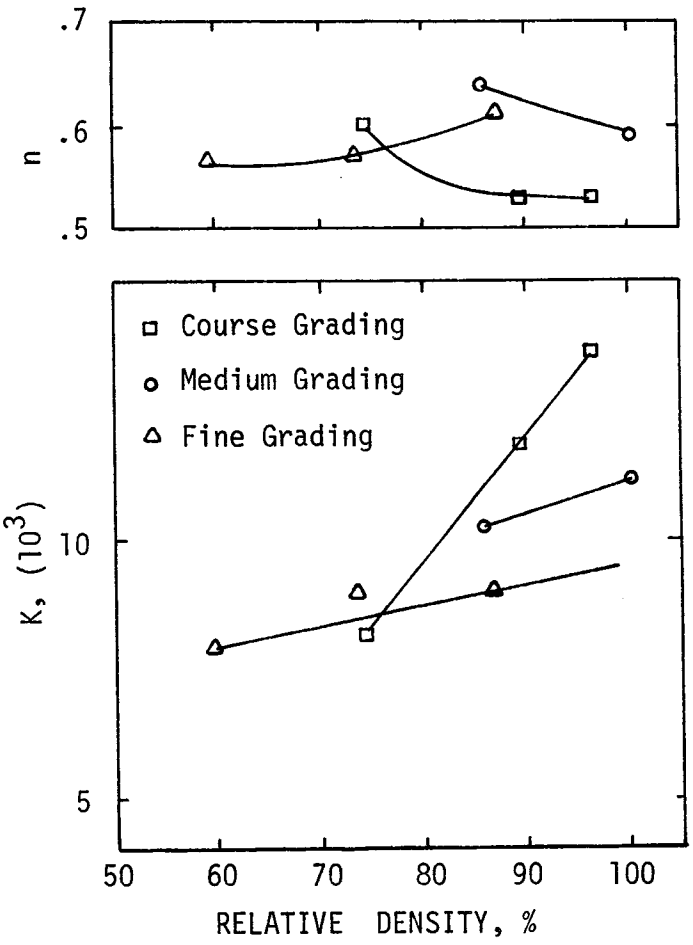


FIGURE 19.- VARIATION OF CONSTANTS K AND n WITH RELATIVE DENSITY (PARTIALLY CRUSHED AGGREGATE) (26)

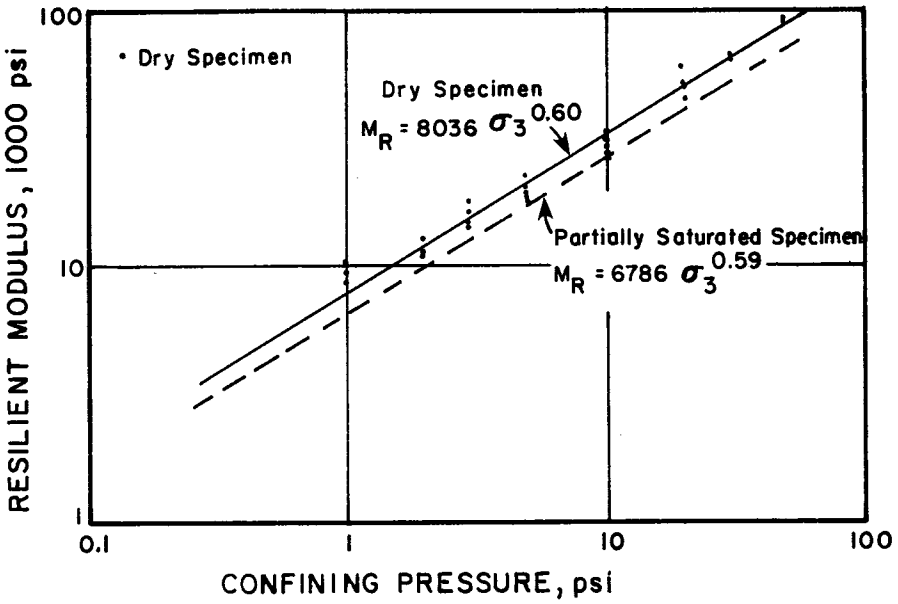


FIGURE 20.- VARIATION OF RESILIENT MODULUS WITH CONFINING PRESSURE (PARTIALLY CRUSHED AGGREGATE) (26)

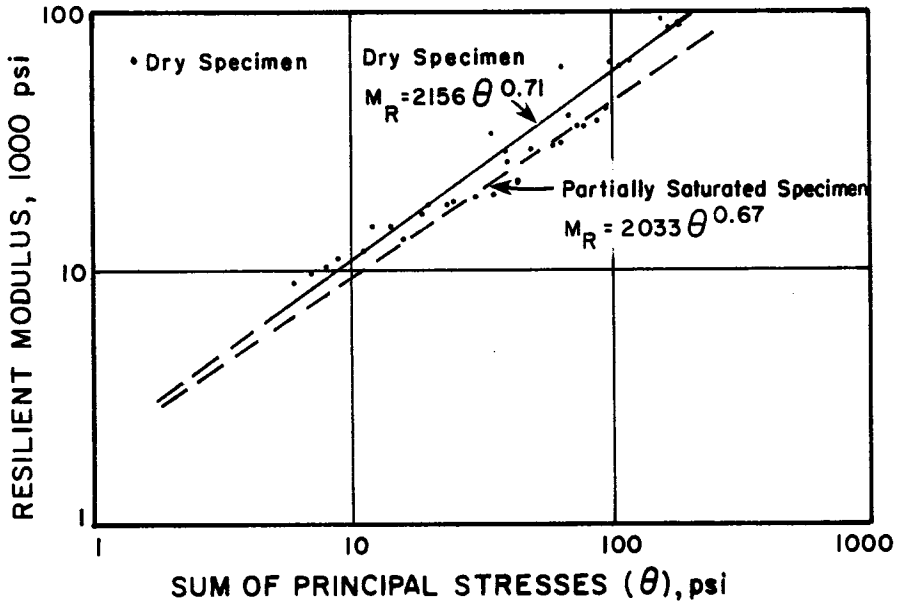


FIGURE 21.- VARIATION OF RESILIENT MODULUS WITH SUM OF PRINCIPAL STRESSES (PARTIALLY CRUSHED AGGREGATE) (26)

regression constants are also shown in these figures.

Stress analyses in typical pavement systems show that small tensile confining stresses can be developed in the base course layer. It is believed that the behavior of untreated granular base course material with tensile confining stresses will be different from that with compressive confining stresses, although no test results are available at the present time. Heukelom and Klomp (25) suggested that the action of tensile confining stress causes local decompaction of the granular base which results in a reduction of modulus. They also stated that due to granular interlock and frictional resistance induced by the vertical compressive stress, granular materials can sustain a small tensile stress without failing. However, a reduction in modulus value in the zone where a tensile confining stress is present can be accounted for by assigning a low value for that region corresponding to an unconfined condition. Hicks (26) found that an untreated granular material in an unconfined condition can have a vertical modulus of elasticity of 5000 psi or more. In this study, 5000 psi was considered as the lowest modulus value for a base course material when subjected to a small tensile stress.

Deformations of granular base course materials under one application of load representative of in-service stress conditions are completely recoverable. However, a large number of such load applications causes permanent deformation in the material. In this study, such permanent deformation constitutes the design criterion. Besides the magnitude of stresses and the number of load applications, permanent deformation in granular material is also influenced by the degree of saturation. Permanent deformation increases with an increase in degree of saturation, which indicates the importance of proper drainage in the structure. Barksdale (4) studied the permanent deformation on several untreated granular materials. He plotted the permanent axial strain for 100,000 load repetition as a function of deviator stress for a series of confining stresses.

From these results he found that a hyperbolic stress-strain representation, analogous to that used by Duncan and Chang (14) to describe static triaxial tests, can be used to fit the cycled stress versus permanent strain data. The hyperbolic equation can be written as follows:

$$\varepsilon_a^p = \frac{(\sigma_1-\sigma_3)/K\,\sigma_3^n}{1 - \left\{\frac{(\sigma_1-\sigma_3)\cdot R_f}{\frac{2(C\cdot\cos\phi+\sigma_3\,\sin\phi)}{(1-\sin\phi)}}\right\}}$$

where

$\varepsilon_a^p$ = permanent axial strain (after a particular number of load applications)

$(\sigma_1-\sigma_3)$ = deviator stress

$K\,\sigma_3^n$ = initial tangent modulus as a function of confining stress (K and n are constants)

$C$ = cohesion

$\phi$ = angle of internal friction

$R_f$ = ratio of compressive strength to an asymptotic stress difference $\frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)_{ult}}$ ; for granular base course materials the value of $R_f$ generally lies between .75 to 1.00.

Assuming that failure in the material will occur with no change in the value of $\sigma_3$, the relationship between compressive strength and confining stress may be expressed in terms of the Mohr-Coulomb failure criterion as:

$$(\sigma_1-\sigma_3)_f = \frac{2\,C\,\cos\phi + 2\,\sigma_3\,\sin\phi}{1-\sin\phi}$$

Now the above hyperbolic stress strain equation can be rewritten as:

$$\varepsilon_a^p = \frac{(\sigma_1-\sigma_3)}{E_i\left(1 - \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_{ult}}\right)}$$

where

$E_i = K \sigma_3^n$ (as defined earlier)

$(\sigma_1-\sigma_3)_{ult}$ = ultimate value of deviator stress.

Figures 22 and 23 show how to obtain the values of $E_i$ and $(\sigma_1-\sigma_3)_{ult}$ from the stress-strain relationship.

In the design procedure, it is required to obtain the magnitude of permanent deformation corresponding to a very large number of load applications that will occur in a design period. It is very difficult and time consuming to generate stress-strain curves for such a large number of load applications. Therefore the author developed an equation, from the data produced by Barksdale ( 4 ), to predict the growth of such deformation for a large number of load applications from known data corresponding to a lower number of load applications. The permanent deformation accumulates approximately logarithmically with the number of load applications and the rate of accumulation of such deformation is increased by an increase in magnitude of deviator stress. Figure 24 shows the influence of the number of load applications and the magnitude of deviator stress on the permanent deformation. The equation which predicts the permanent deformation at a desired large number of load applications is shown below:

$$\varepsilon^p_{ND} = \varepsilon^p_{NK} + \{\log_{10}(ND) - \log_{10}(NK)\} \cdot (SLOPE)$$

where

$\varepsilon^p_{ND}$ = permanent strain at a desired large number of load application, percent

$\varepsilon^p_{NK}$ = permanent strain at a known number of load application

ND = desired large number of load application at which $\varepsilon^p_{ND}$ is required

NK = number of load application which produces $\varepsilon^p_{NK}$.

SLOPE = $1.3507 \times 10^{-3} (\sigma_1-\sigma_3)^{1.2623}$ for $0 < (\sigma_1-\sigma_3) < 30$ psi

and SLOPE = $1.0543 \times 10^{-4} (\sigma_1-\sigma_3)^{2.0191}$ for $(\sigma_1-\sigma_3) > 30$ psi.

Asphalt-Treated Granular Base Course Material (ATB).- Asphalt-treated granular base course materials are most commonly used as

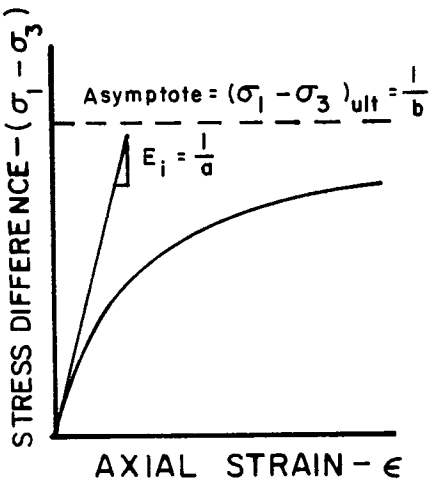


FIGURE 22.- HYPERBOLIC STESS-STRAIN CURVE (14)

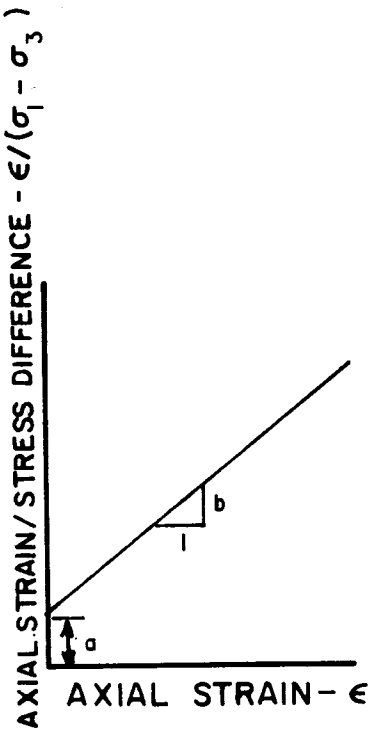


FIGURE 23.- TRANSFORMED HYPERBOLIC STRESS-STRAIN CURVE (14)

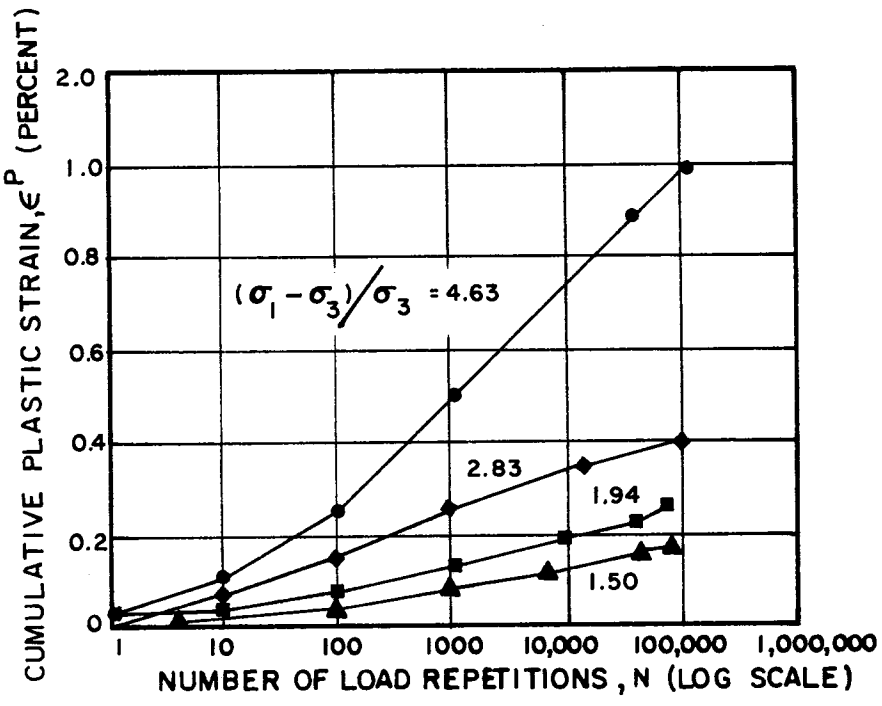


FIGURE 24.- INFLUENCE OF NUMBER OF LOAD APPLICATIONS AND MAGNITUDE OF DEVIATOR STRESS ON PERMANENT DEFORMATION (PORPHYRITIC GRANITE GNEISS -3% FINES) (4)

flexible pavement base. There is a very limited amount of published information available regarding the constitutive properties of ATB materials. Smith et al. (37) found that the constitutive properties of ATB materials are dependent on both temperature and time (duration of load). This dependency requires the use of viscoelastic theory to represent the constitutive properties of these materials. However, elastic constitutive representations for these materials may be used and temperature and time (duration of load) effects can be accounted for through the selection of appropriate constitutive values. The elastic material properties must first be defined over the range of temperatures and load durations of interest and then the values of such parameters must be selected for applicable temperature and load duration. Figure 25 shows the influence of temperature on modulus values of a typical ATB material for a load duration of 0.1 sec. (typical in-service load duration).

As with asphalt concrete, ATB materials generally fail in fatigue. However, a thick layer of ATB material will also accumulate some amount of permanent deformation due to the repetition of loads. It is believed that these materials will have very small permanent deformation compared to that in untreated granular base course materials due to their higher modulus value and compressive strength. Due to asphalt treatment, granular base course materials develop a cohesive force which prevents any sharp rupture or a sudden change under ordinary conditions of loading. However, Goetz (20) found that with an increase in asphalt content, the angle of internal friction decreased sharply and at an almost constant rate; and the cohesive value increased until a maximum value was reached at about 4% asphalt and then decreased as the asphalt content increased. Figure 26 shows the variation of internal friction angle and cohesion due to increase in asphalt content in an ATB material. At the present time no mathematical model for predicting the permanent deformation characteristics of asphalt treated material is available. In such a situation it was

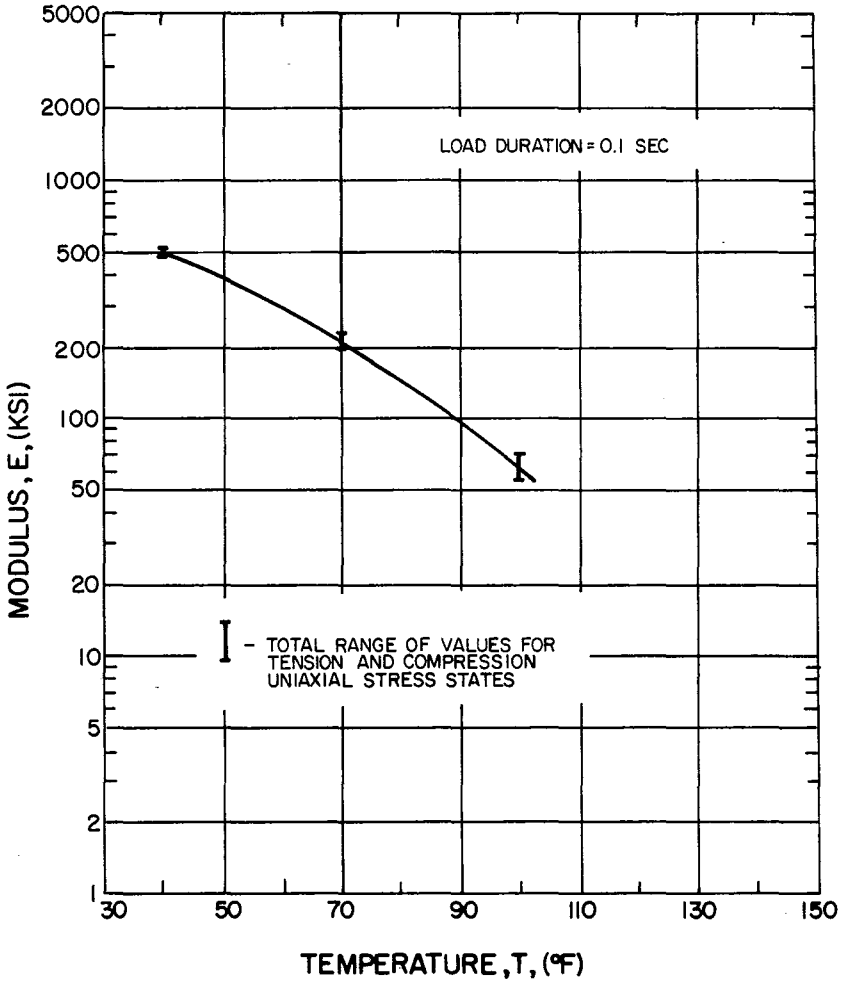


FIGURE 25.- TEMPERATURE INFLUENCE ON ATB MODULUS (ULTIMATE CURE CONDITION) (37)

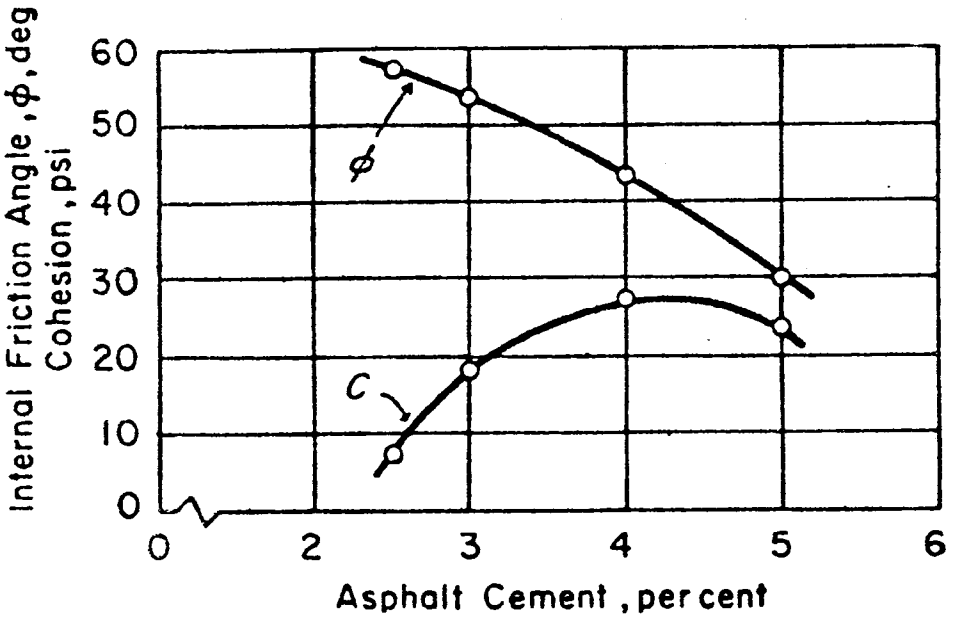


FIGURE 26.- VARIATION OF TEST PROPERTIES WITH PERCENT ASPHALT IN ATB MATERIALS (20)

assumed that the same deformation law as was used for untreated base course material can also be used for ATB materials. The deformation equation was presented earlier in this chapter and is again shown below:

$$\varepsilon_a^p = \frac{(\sigma_1-\sigma_3)}{E_i\ (1 - \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_{ult}})}$$

All the above terms were defined earlier. It can be observed in the above equation that due to an increase in compressive strength and modulus value in an ATB material, the permanent deformation $\varepsilon_a^p$ will be very small compared to that in untreated base course material.

Subgrade Materials

In any pavement design system, the properties of subgrade materials are very important factors. The capacity of subgrade support influences the structural design of pavements and other structures. Usually subgrade materials are composed of fine-grained sand and silt and clay fractions with high plasticity index. The strength and performance of subgrade materials are greatly influenced by environmental factors such as temperature, moisture balance, drainage, etc. Edris (17) conducted a comprehensive study on subgrade materials from different climatic zones in Texas. These were composed of different clay contents ranging from 20% to 70%. He developed regression equations for resilient modulus and permanent deformation for these materials with their temperature correction factors. In these equations two factors, number of load applications and soil suction, were most important. Other factors which enter these equations are degree of saturation, volumetric moisture content, volumetric soil content, deviator stress, confining stress and mean principal stress. It was decided to use these equations for the design purpose of this study. The equations of resilient modulus and permanent

deformation corresponding to clay contents of 20%, 39% and 70% are presented below. Their values corresponding to any intermediate clay content are calculated by quadratic interpolation of these results.

Resilient Modulus.-

Soil with 20% clay content

$$M_R = -1827.72 + 171705.0[(\frac{h_f}{h_i})^{0.20} N^{0.081}][1 + 0.6566(1-n)^{1.4} \{1-4.4849(\sigma_1-\sigma_3)^{-0.16}\} + 64.6522(S)^{-0.26}\{1-1.6108 (\sigma_1-\sigma_3)^{-0.16} - 0.001155(\sigma_m)^{0.063}\}-14.8816(nS)^{-0.30} \{1-1.5899(\sigma_1-\sigma_3)^{-0.16}\}]$$

Soil with 39% clay content

$$M_R = 7980.89 + 2981.64[(\frac{h_f}{h_i})^{0.20} N^{0.145}][1 + 64.397(1-n)^{3.3} \{1-4.2008(\sigma_1-\sigma_3)^{-0.60}\} - 2.002 \times 10^{-3}(S)^{2.0}\{1-3.7228 (\sigma_1-\sigma_3)^{-0.60} - 0.1639(\sigma_m)^{-0.23}\}-0.1974(nS)^{-2.25} \{1-4.2766(\sigma_1-\sigma_3)^{-0.60}\}]$$

Soil with 70% clay content

$$M_R = -4791.99 - 27272.4[(\frac{h_f}{h_i})^{0.20} N^{0.084}][1 - 45.0169(1-n)^{3.6} \{1-3.733(\sigma_1-\sigma_3)^{-0.60}\} + 1.706 \times 10^{-7}(S)^{3.6}\{1-5.0763 (\sigma_1-\sigma_3)^{-0.60} - .1288(\sigma_m)^{-0.27}\}+0.05999(nS)^{-3.3} \{1-5.8416(\sigma_1-\sigma_3)^{-0.60}\}]$$

where: $M_R$ = resilient modulus, psi
$h_i$ = initial suction, psi
$h_f$ = final suction, psi
$(1-n)$ = volumetric soil content, decimal form
$nS$ = volumetric moisture content, decimal form

$S$ = saturation, %

$(\sigma_1-\sigma_3)$ = deviator stress, psi

$\sigma_m$ = mean stress, psi.

Permanent Strain.-

Soil with 20% clay content

$$\varepsilon^p = 0.04076 - 1.2679[(\frac{1}{h_t})^{0.65} N^{0.395}][1-1.2067 \times 10^{-15} \{(\frac{(S)^{5.35}}{(nS)^{5.10}})(\frac{1}{1-n})^{10.4}\}]$$

Soil with 39% clay content

$$\varepsilon^p = 0.01519 - 0.000254\ N^{0.63}[1-24.62205(\frac{h_t}{h_f})^{0.50}\{(\frac{1}{\sigma_3\ h_f})^{.38} -0.01297(\sigma_1-\sigma_3)^{1.58}(\frac{1}{h_f})^{0.54}\} + 61.1811(\frac{1}{\sigma_m\ h_f})^{0.60} -0.52205(\sigma_1-\sigma_3)^{1.58}(\frac{1}{h_f})^{0.54}]$$

Soil with 70% clay content

$$\varepsilon^p = -0.000186 - 0.000443\ N^{0.45}[1-63.0264(\frac{h_t}{h_f})^{0.61}\ (\frac{1}{\sigma_3\ h_f})^{0.25} -0.09398(\sigma_1-\sigma_3)^{0.24}(\frac{1}{h_f})^{0.24}\ + 123.8399(\frac{1}{\sigma_m\ h_f})^{0.40} -5.9323(\sigma_1-\sigma_3)^{0.24}(\frac{1}{h_f})^{0.24}\ ]$$

where: $\varepsilon^p$ = permanent strain, in percent

$h_t$ = test suction, psi

$\sigma_3$ = confining stress, psi.

The other terms are defined earlier.

Prediction of Temperature Correction Factors.- To determine the influence of temperature on resilient modulus and permanent strain, factors other than temperature such as number of load cycles, deviator stress and soil suction were also considered. A reference state: 72°F (22.2°C) temperature, 10000 load cycles, 13.7 psi (94.5 kN/m$^2$) deviator stress and soil suction

corresponding to moisture content 2% dry of optimum moisture content was used to determine the ratios of the above factors. These ratios were used to determine the temperature correction factors for resilient modulus and permanent strain. For a climatic region where the subgrade experiences a temperature other than 72°F (22.2°C), these factors are calculated. The actual values of resilient modulus and permanent deformation of a subgrade at a particular temperature can be obtained by multiplying their predicted values at 72°F (22.2°C) with their corresponding temperature correction factor.

The equation developed for the temperature correction factor for the resilient modulus is:

$$f_{M_R} = a_0 - a_1\left(\frac{D}{D_0}\right)^b + a_2\left(\frac{h}{h_0}\right)^c + a_3\left(\frac{T}{T_0}\right)^d \{1 - a_4\left(\frac{h}{h_0}\right)^c \left(\frac{D}{D_0}\right)^b + a_5\left(\frac{N}{N_0}\right)^e [1 - a_6\left(\frac{h}{h_0}\right)^c + a_8\left(\frac{h}{h_0}\right)^c \left(\frac{D}{D_0}\right)^b - a_7\left(\frac{D}{D_0}\right)^b]\}$$

where: $\left(\frac{D}{D_0}\right)$ = deviator stress ratio

$\left(\frac{h}{h_0}\right)$ = soil suction ratio

$\left(\frac{T}{T_0}\right)$ = temperature ratio

$\left(\frac{N}{N_0}\right)$ = number of load cycle ratio

$b = -1.7013 + 6.2014\ (PL)$

$c = 0.0271 - 0.2873 \log\ (clay)$

$d = 0.0697 - 0.9846\ (clay)$

$e = 0.0582 - 0.00226\ (1/clay)$

$a_0 = -125.574(SL) - 2764.13(PL) + 21234.1\ (SL \times PL)$

$a_1 = -465.052(SL) - 2890.01(PL) + 23642.5\ (SL \times PL)$

$a_2 = -37.6644 + 279.813\ (SL + PL)^2$

$a_3 = -15.0184 + 13786.434\ (SL \times PL)^2$

$a_4 = 0.8088 + 0.3006\ (clay)$

$a_5 = 30.8763 - 306.7167\ (LL)^2$

$a_6 = 7.5058(SL) - 6.0135(PL) + 41.1548\ (SL \times PL)$

$a_7 = 3.6476(PL) + 2.0336(LL) - 7.3402\ (PL \times LL)$

$a_8$ = 4.370(SL) - 6.1516(PL) + 53.4137 (SL X PL)

clay = clay fraction of soil in decimal form

LL = liquid limit

PL = plastic limit

SL = shrinkage limit

However, temperature correction for resilient modulus was not included in this study since this was found to be insignificant in the total design.

The equation to determine the temperature correction factor for permanent strain is:

$$f_{\varepsilon p} = a_0+a_1(\frac{h}{h_0})^c+a_2(\frac{T}{T_0})^d(\frac{N}{N_0})^e\{1-a_3(\frac{h}{h_0})^c+a_4(\frac{h}{h_0})^c(\frac{D}{D_0}) -a_5(\frac{D}{D_0})^b\}$$

where:

b = 0.6761 - 0.2384 (1/clay)

c = -1.7043 + 1.9130 (200 sieve)

d = 2.3620 - 0.4128 (1/clay)

e = 0.3716 + 0.1700 (clay)

$a_0$ = -114.111 + 159.212 (200 sieve)

$a_1$ = 119.823 - 166.053 (200 sieve)

$a_2$ = -81.345 - 41.866 (1/log clay)

$a_3$ = 0.7882 + 1.4700 (SL)

$a_4$ = -0.0663 + 1.5214 (200 sieve)

$a_5$ = -0.2791 + 1.7426 (200 sieve)

200 sieve = the amount of soil that passed the #200 sieve in decimal form.

## Summary

This chapter summarizes the material characteristics such as the constitutive relations and deformation characteristics of different materials that were used in this study. Typical materials for surface layer, base course and subgrade were discussed in detail.

Although the resilient moduli of asphalt concrete and treated base course material are basically time and temperature

dependent, it is suggested here that a proper selection of elastic parameters corresponding to the actual temperature and load duration can be used in the analysis and design purposes. Nonlinearity in the constitutive relations was shown in ballast material, untreated base course material and subgrade material. The influence of temperature and environmental effects on constitutive relation and deformation characteristics of subgrade materials were incorporated. Permanent deformations in an asphalt concrete layer in a flexible pavement structure and timber ties in a railroad structure were assumed to be very small and neglected in this study.

# CHAPTER IV

## APPLICATION OF DESIGN TECHNIQUE

Several design examples are presented in this chapter to illustrate the overall design procedure. The first step of the design procedure is to determine the amount of traffic that is to be served by the grade crossing. The railroad track in a grade crossing must serve both the railway and highway wheel loadings, whereas, the adjacent highway pavement is required to serve only the highway wheel loading. The standard highway and railway wheel loadings which were used in this design, are described by Ahmad (55). The second step involves fixing the layer thicknesses of each structure (railroad track and adjacent pavement). The third step is the selection of materials for each layer. Material selection involves careful consideration of its characteristics and particularly for this case, resilient modulus and permanent deformation characteristics. The characterization of these material properties are equations which include the influence of environmental factors such as temperature and suction levels of subgrade material. As a fourth step, temperature, suction and clay content information is input into the computer program developed in this study, to calculate the differential deformation between the railroad track and the adjacent pavement structure. While developing the computer program limits were placed on the values of some of the variables in order to make sure that all calculated results stay within a reasonable range of values. These limits are documented in the program in Appendix C with comment statements.

### Selection of Environmental Data

The temperature influences the behavior of surface layer, base course and subgrade materials. Proper selection of modulus values for surface layer and base course materials is very

important for the purpose of analysis and design. These modulus values can be selected from Figures 18 and 25 in Chapter III. As shown earlier in Chapter III, Edris (17) developed equations to include the influence of suction and temperature on subgrade materials. Barber (3) showed how to calculate the pavement temperature from weather reports. When temperature influence is not considered the input data for temperature is $72^{o}F$.

Besides temperature, three levels of suction values such as initial suction $h_i$, test suction $h_t$, and final suction $h_f$ corresponding to subgrade materials with 20%, 39% and 70% clay content are required as input in the program. These values will be different for different climatic zones. The procedure explained below gives details of how each of these values of suction are determined.

Russam (35) developed the relationship between Thornthwaite moisture index, which is a function of climatic conditions, and equilibrium (initial) suction level for different types of subgrade materials as shown in Figure 27. This relationship is not valid in areas where there is a high water table. Carpenter (6) used Russam's curves and calculated the initial suction level in different climatic zones of Texas considering the subgrade to be composed of heavy clay and considering no influence of the water table on suction level. His calculated values of initial suction and the corresponding Thornthwaite moisture index are presented on the map of Texas in Figure 28. Figure 29 shows the relationship between the clay content and the ratio of suction at any desired clay content to that at 70% clay content. Using this figure and Carpenter's calculated values of initial suction for heavy clay (assumed to be composed of 70% clay content) in different climatic zones, corresponding suction values at 20% and 39% clay content can be obtained. Figure 30 shows the relationship between the ratio of final suction $h_f$ to initial suction $h_i$ and clay content. Figure 31 shows the relationship between the ratio of test suction $h_t$ to final suction $h_f$ and number of load cycles. Using these three figures, final and

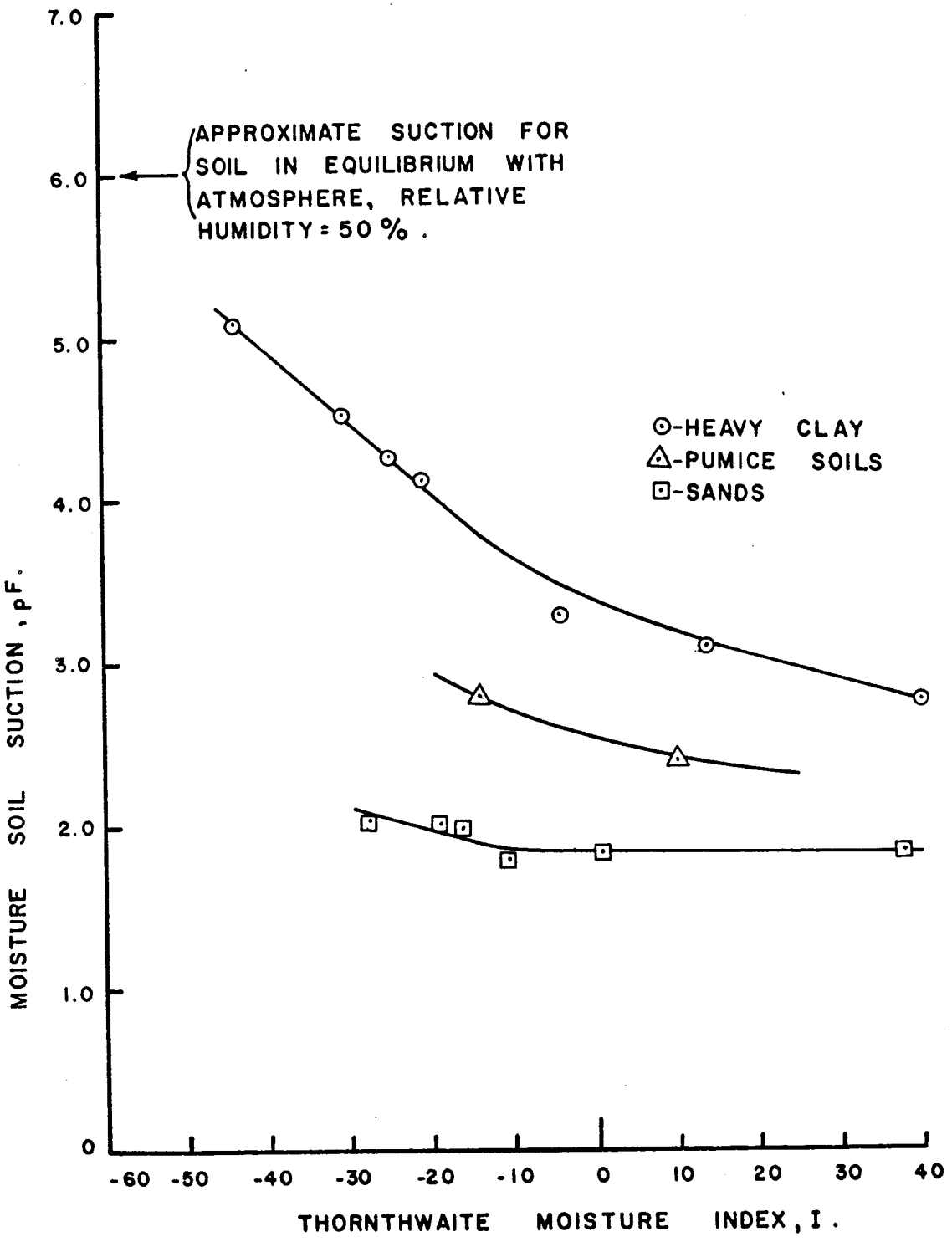


FIGURE 27.- SUBGRADE SOIL SUCTION AS A FUNCTION OF THE THORNTHWAITE MOISTURE INDEX (35)

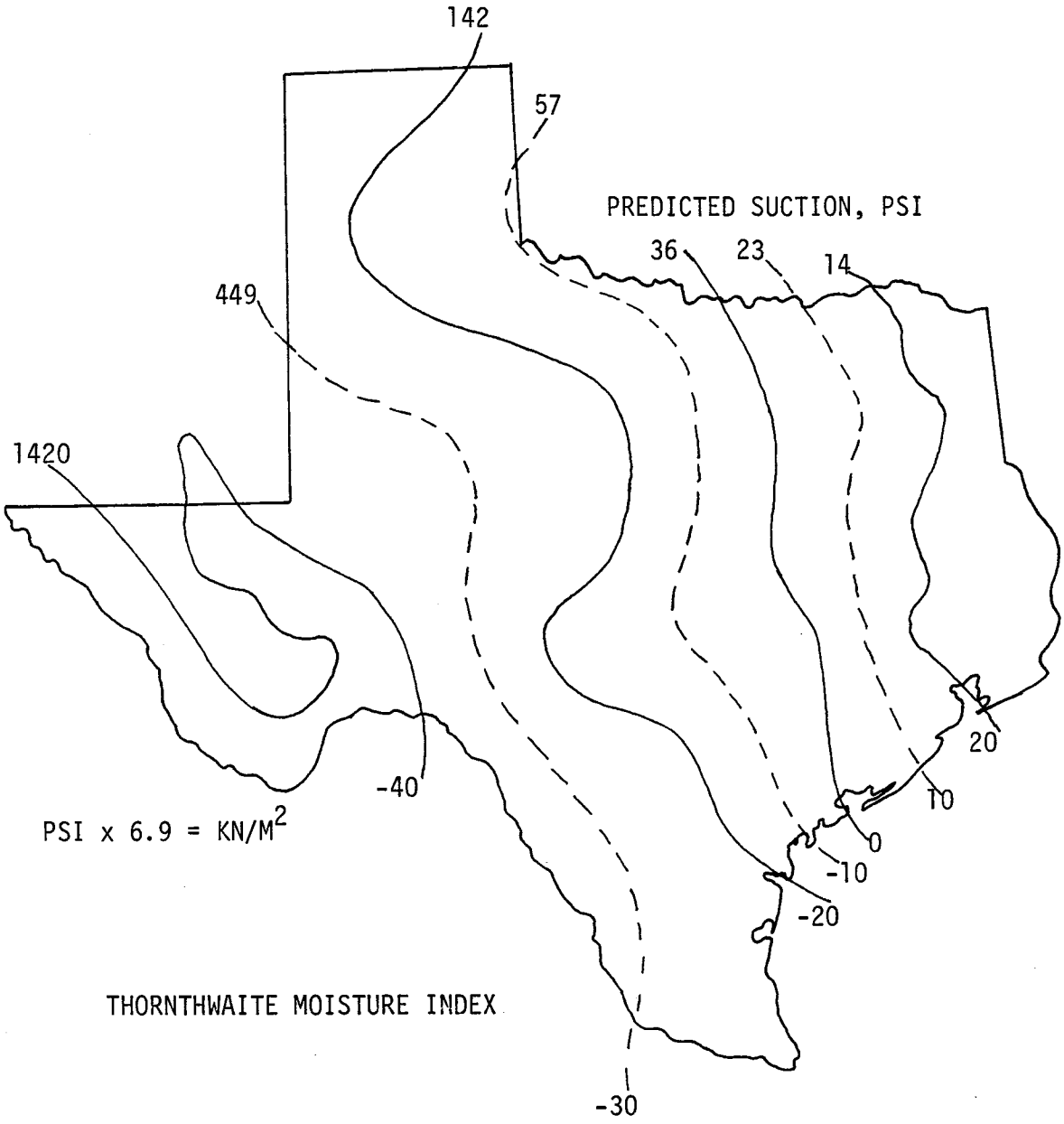


FIGURE 28.- CALCULATED VALUES OF INITIAL SUCTION IN SUBGRADE (HEAVY CLAY) AND CORRESPONDING THORNTHWAITE MOISTURE INDEX IN TEXAS (6)

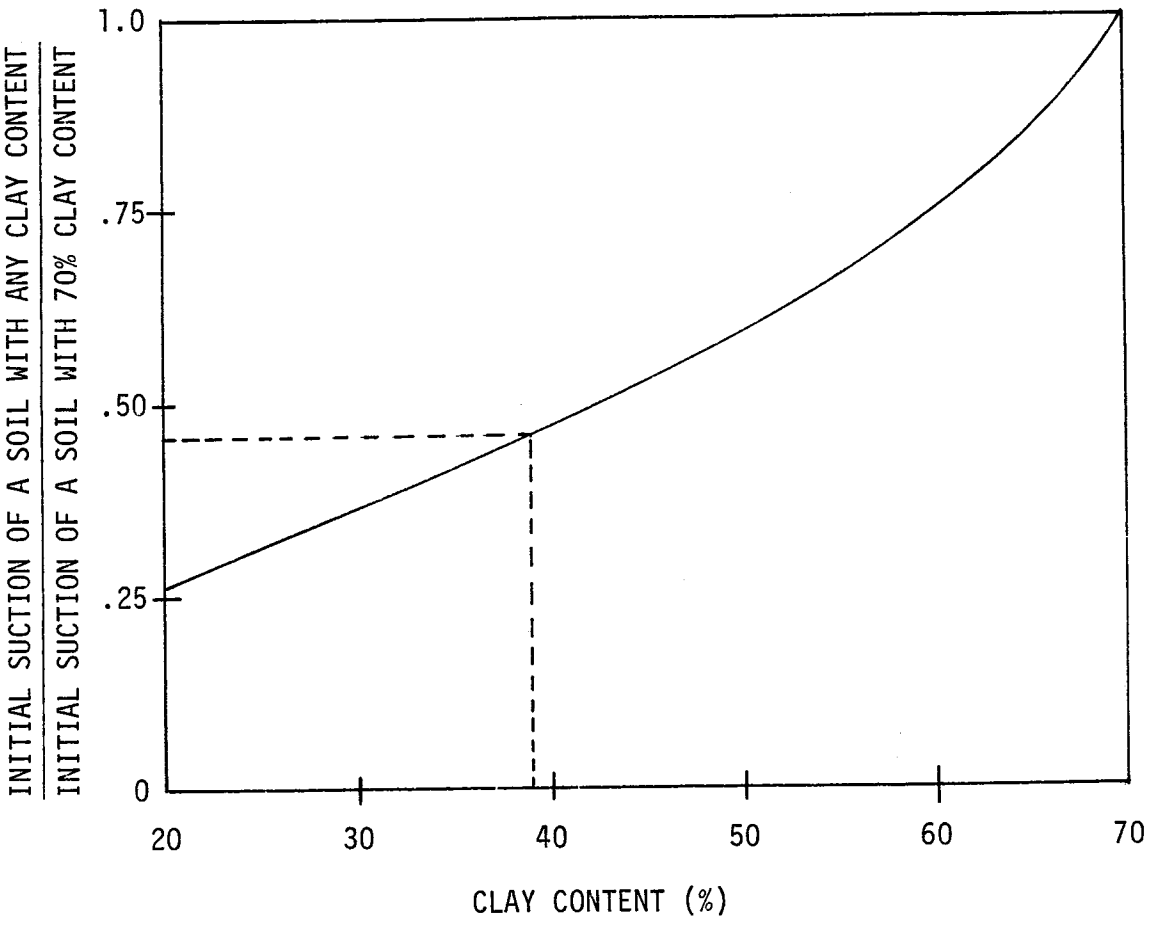


FIGURE 29.- THE RATIO OF INITIAL SUCTION OF A SOIL WITH ANY CLAY CONTENT TO THAT OF A SOIL WITH 70% CLAY CONTENT AS A FUNCTION OF THE CLAY CONTENT

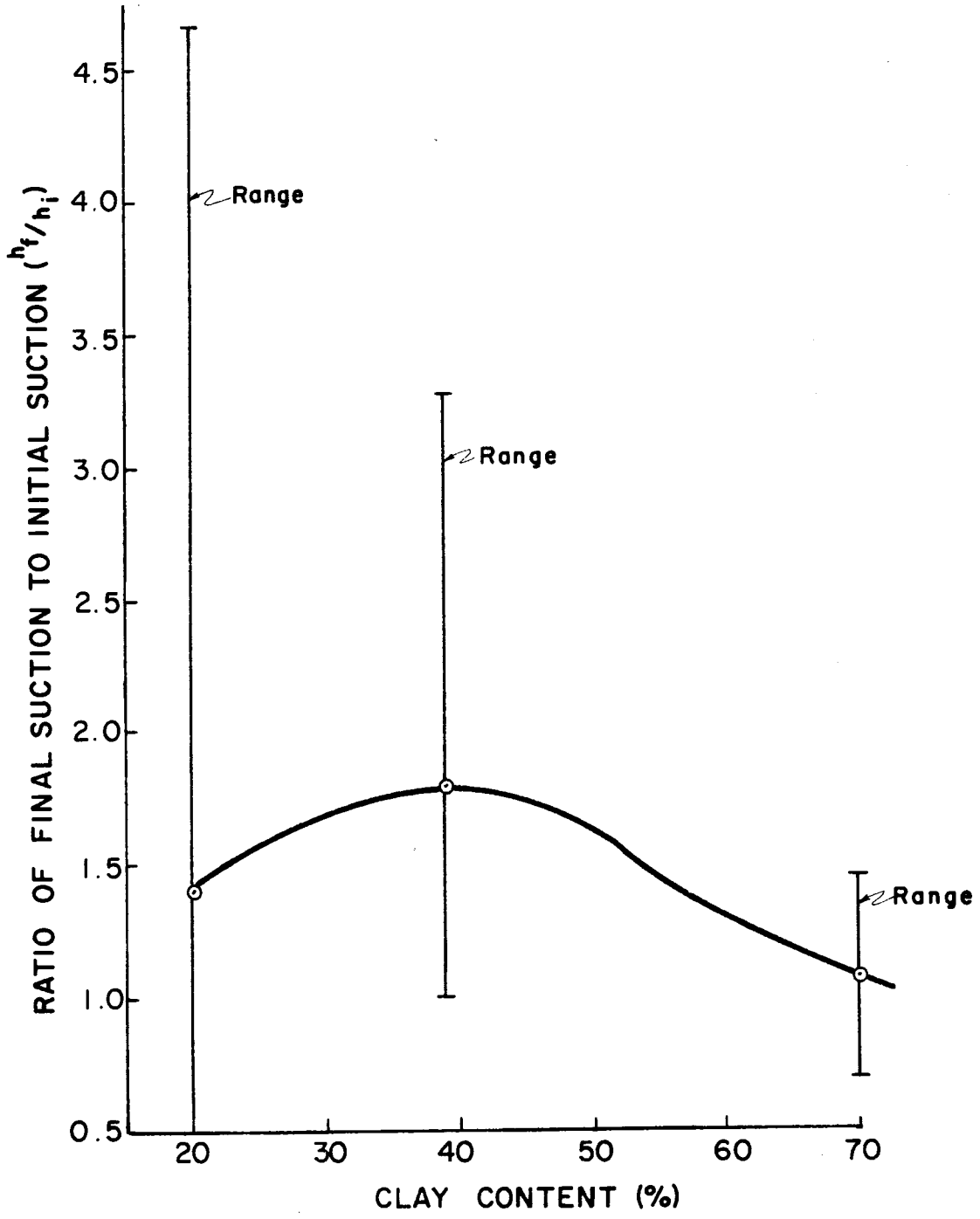


FIGURE 30.- THE RATIO OF FINAL SUCTION TO INITIAL SUCTION AS A FUNCTION OF THE CLAY CONTENT (17)

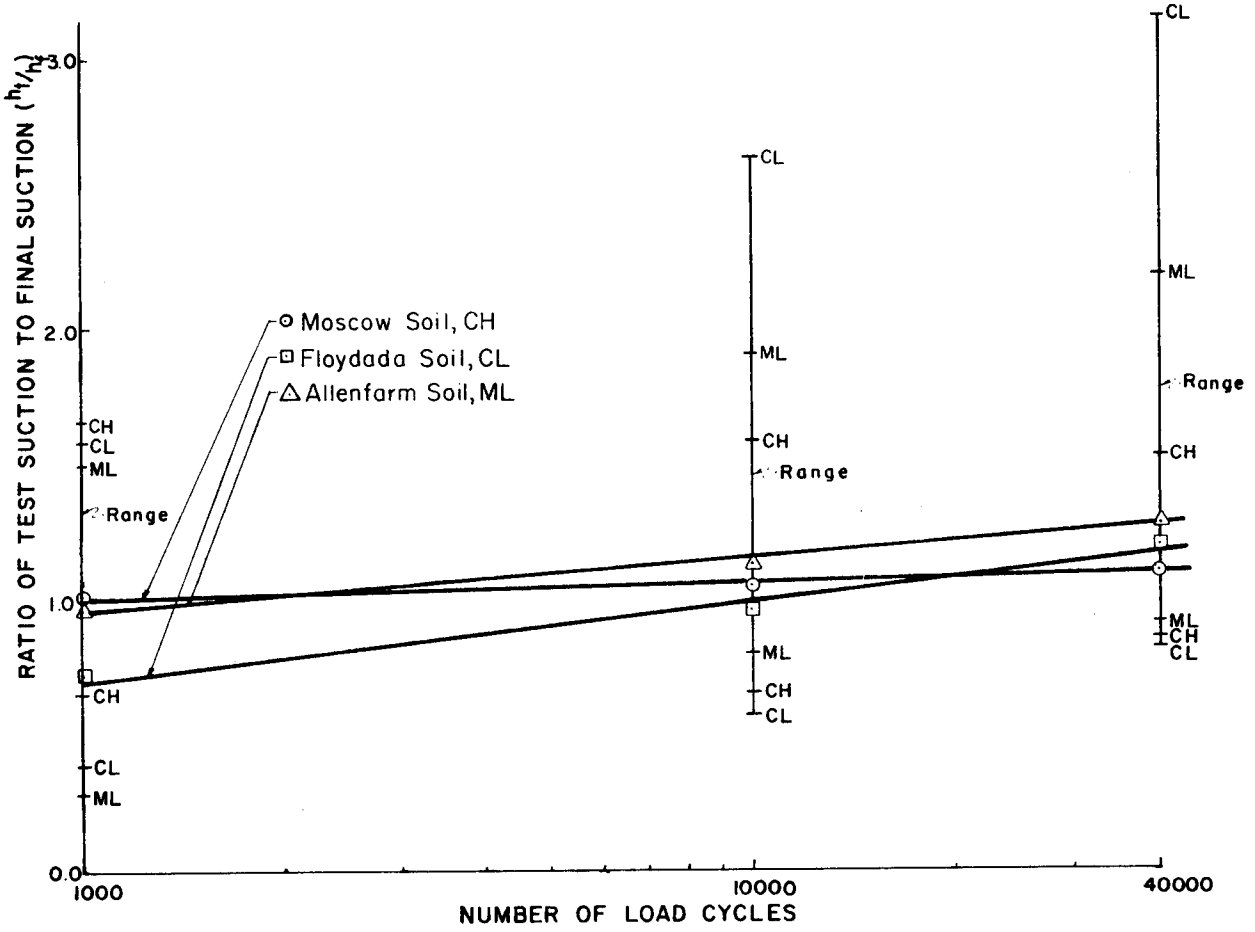


FIGURE 31.- THE RATIO OF TEST SUCTION TO FINAL SUCTION AS A FUNCTION OF THE NUMBER OF LOAD CYCLES (17)

test suctions corresponding to each initial suction can be obtained as follows:

$h_o$ - initial suction of heavy clay (70% clay content) from Figure 28

$h_i$ - initial suction of any soil

Figure 29 gives the suction ratio, $\gamma_1$

Initial suction, $h_i = \gamma_1 h_o$

$h_f$ - Final suction of any soil

Figure 29 gives the suction ratio, $\gamma_1$

Figure 30 gives the final-to-initial suction ratio, $\gamma_2$

Final suction = $h_f = \gamma_2 h_i = \gamma_1 \gamma_2 h_o$

$h_t$ - Test suction of any soil

Figure 29 gives the suction ratio, $\gamma_1$

Figure 30 gives the final-to-initial suction ratio, $\gamma_2$

Figure 31 gives the test-to-final suction ratio, $\gamma_3$

Test suction = $h_t = \gamma_3 h_f = \gamma_2 \gamma_3 h_i = \gamma_1 \gamma_2 \gamma_3 h_o$

In the computer program, it was assumed that the ratio of test suction to final suction would remain nearly the same after 40,000 load cycles. The value of $h_o$ is read in as input data and other values of $h_i$, $h_f$ and $h_t$ are internally computed in the program.

The following example problems will illustrate the design process using the computer program developed in this study.

Example Problem No. 1.

Input Information:

It is assumed that the average temperatures in top layer, base course and subgrade are $90^oF$, $85^oF$ and $72^oF$ respectively. The location of the grade crossing lies in a climatic zone having a Thornthwaite moisture index of -10. The number of repetition of wheel loads (required to serve a design period) is considered to be 1,000,000 for both highway and railway traffic. The base course under both highway and railroad are asphalt treated.

A. Railroad Track:

i) Top layer (Ballast)

| | |
|---|---|
| Thickness | 18 inches |
| Resilient Modulus (Not temperature dependent) (Fig. 15) | 50,000 psi |

ii) 2nd Layer (Base Course)

| | |
|---|---|
| Thickness | 12 inches |
| Resilient Modulus (Asphalt treated), $85^{o}F$ (Fig. 25) | 120,000 psi |
| C, cohesion (Fig. 26) | 25 psi |
| $\phi$, angle of internal friction, (Fig. 26) | $45^{o}$ |

iii) Subgrade

The nonlinear equations developed for resilient modulus and deformation characteristics of subgrade material require the following information (17):

| | |
|---|---|
| Initial Guess of Resilient Modulus | 15,000 psi |
| Actual Clay Content | 30% |
| Suction Level for 70% Clay Content (Fig. 28) | 57.00 psi |

B. Highway Pavement Structure:

i) Top Layer (Asphalt Concrete)

| | |
|---|---|
| Thickness | 4 inches |
| Resilient Modulus, $90^{o}F$, (Fig. 18) | 350,000 psi |

ii) 2nd Layer (Base Course)

Same as used in railroad track structure

iii) Subgrade

Same as used in railroad track structure.

Deformations in each layer in the railroad track and highway pavement including the differential deformation as calculated in this example are shown in Table 3. It should be noted that the railroad track deformed more than the highway pavement. Major

TABLE 3.- DEFORMATIONS CALCULATED IN EXAMPLE PROBLEM NO. 1

| Type of Structure | Type of Loading | No. of Load Application | Deformations (inch) | | | Total Deformations (inch) | Differential Deformation (inch) |
|---|---|---|---|---|---|---|---|
| | | | Top Layer | 2nd Layer | Subgrade | | |
| Highway Pavement | Highway Traffic | 1,000,000 | | 0.015 | 0.176 | 0.191 | |
| Railroad Track | Railway Traffic | 1,000,000 | 0.979 | 0.025 | 0.212 | 1.216 | |
| Railroad Track | Highway Traffic | 1,000,000 | 0.272 | 0.007 | 0.090 | 0.368 | 1.393 |

portions of these deformations were in ballast and subgrade materials as expected. The differential deformation is 1.393 inch which is an unacceptable (Fig. 9) amount according to the criterion of differential deformation established in Chapter II.

Example Problem No. 2

The input information is identical to Example Problem No. 1 except in this problem unstabilized bases were used in both railroad track and highway pavement structures. The following are the input information for unstabilized base:

2nd Layer (Base Course)

Thickness 12 inch

Resilient Modulus, nonlinear and independent of temperature, expressed in the following equation:

$$M_R = 15{,}000\ \sigma_3^{.5}$$

where

$M_R$ = resilient modulus, psi

$\sigma_3$ = confining stress, psi

C, cohesion (Fig. 26) 0

$\phi$, angle of internal friction (Fig. 26) $50^o$

The calculated deformations in this example are shown in Table 4. From these calculated results it can be seen that the ballast and the subgrade under the railroad deformed more than previously and the highway pavement, due to higher stresses in the subgrade, also deformed more than previously. The differential deformation is increased to 2.015 inch which is still larger than the acceptable limit established in Chapter II.

Example Problem No. 3

From the results of the first two example problems, it can be easily seen that an unstabilized base is required in the highway pavement structure. This allows the highway pavement

TABLE 4.- DEFORMATIONS CALCULATED IN EXAMPLE PROBLEM NO. 2

| Type of Structure | Type of Loading | No. of Load Application | Deformations (inch) | | | Total Deformations (inch) | Differential Deformation (inch) |
|---|---|---|---|---|---|---|---|
| | | | Top Layer | 2nd Layer | Subgrade | | |
| Highway Pavement | Highway Traffic | 1,000,000 | | 0.083 | 0.370 | 0.453 | |
| Railroad Track | Railway Traffic | 1,000,000 | 1.548 | 0.118 | 0.218 | 1.884 | |
| Railroad Track | Highway Traffic | 1,000,000 | .422 | 0.045 | 0.118 | 0.584 | 2.015 |

to deform uniformly with the railroad track by which the differential deformation is reduced. It is expected that the use of a stabilized base in the railroad track along with an unstabilized base in the highway pavement would reduce the differential deformation even more and make the design more acceptable. Therefore, in this example a combination of an unstabilized base in the highway pavement and a stabilized base in the railroad track was used. The basic input data are shown in the previous two examples. Calculated results are shown in Table 5. The differential deformation is reduced to 1.131 which is closest of all designs yet considered to being within the acceptable limit (Fig. 9). The design in this example is accepted as standard for the remaining example problems in this cahpter, some of thwich will investigate the influence of changes in eemperature and climatic zones (suction levels).

Example Problem No. 4

The basic input data as used in Example Problem No. 3 are used in this example except for the change in temperature. It is assumed that the average temperatures in the top layer, base course and subgrade are $95^{o}F$, $90^{o}F$ and $80^{o}F$ respectively. The change in resilient modulus values due to the temperature changes are as follows:

| | |
|---|---|
| Resilient Modulus of Asphalt Concrete, $95^{o}F$ (Fig. 18) | 250,000 psi |
| Resilient Modulus of Asphalt Treated Base, $90^{o}F$ (Fig. 25) | 90,000 psi |

The calculated deformations are shown in Table 6. Due to the changes in the resilient modulus values the stresses and consequently the deformations are changed in each layer. The deformations are increased in every layer compared to those in Example Problem No. 3. The differential deformation is also increased to 2.181 inch, making the design clearly unacceptable. This example clearly indicates that a design that is nearly acceptable in a particular temperature zone may be completely unacceptable in

TABLE 5.- DEFORMATIONS CALCULATED IN EXAMPLE PROBLEM NO. 3

| Type of Structure | Type of Loading | No. of Load Application | Deformations (inch) | | | Total Deformations (inch) | Differential Deformation (inch) |
|---|---|---|---|---|---|---|---|
| | | | Top Layer | 2nd Layer | Subgrade | | |
| Highway Pavement | Highway Traffic | 1,000,000 | | 0.083 | 0.370 | 0.453 | |
| Railroad Track | Railway Traffic | 1,000,000 | 0.979 | 0.025 | 0.212 | 1.216 | |
| Railroad Track | Highway Traffic | 1,000,000 | 0.272 | 0.007 | 0.090 | 0.368 | 1.131 |

TABLE 6.- DEFORMATIONS CALCULATED IN EXAMPLE PROBLEM NO. 4

| Type of Structure | Type of Loading | No. of Load Application | Deformations (inch) | | | Total Deformations (inch) | Differential Deformation (inch) |
|---|---|---|---|---|---|---|---|
| | | | Top Layer | 2nd Layer | Subgrade | | |
| Highway Structure | Highway Traffic | 1,000,000 | | 0.080 | 0.427 | 0.507 | |
| Railroad Structure | Railway Traffic | 1,000,000 | 1.548 | 0.017 | 0.229 | 1.794 | |
| Railroad Structure | Highway Traffic | 1,000,000 | 0.793 | 0.005 | 0.096 | 0.894 | 2.181 |

another temperature zone.

The influence of different climatic zones and different subgrade clay contents on deformations in highway pavement and railroad track were studied using the design of example 3 as a standard. Four different climatic zones with Thornthwaite moisture index of -30, -10, +10, +20 and two subgrades with 30% and 70% clay contents were used in this study. The calculated deformations are shown in Table 7. From these results it can be seen that the deformations in the highway pavement increase with an increase in Thornthwaite moisture index, i.e., the deformation is larger in a wetter zone, as would be expected.

A consistent difference in relative displacement is maintained throughout these example problems. The difference is due to the deformation of the ballast under railroad and highway loadings, a total permanent displacement of 1.251 inches. Any improvement in this difference must come as a result of differential deformation between the highway and railroad sublayers. If it is assumed that track resurfacing will be done periodically so that the accumulated permanent deformation in the ballast will never be more than half of this value, the performance of the crossings in each location may be compared as in Table 8.

This table shows that when the crossing is built on silt (% clay = 30%), it will have an unacceptable amount of permanent deformation in all except the driest climate. On the other hand, well drained crossings built on clay (% clay = 70%) will approach an unacceptable amount of deformation in the wet zones where the Thornthwaite Index is 10 or above.

The information in Tables 7 and 8 illustrate several important points about railroad grade crossing design.

- few crossings of the sort considered here can be expected to perform satisfactorily without a regular track resurfacing program.

TABLE 7.- DEFORMATIONS CALCULATED FOR DIFFERENT CLIMATIC ZONES AND DIFFERENT CLAY CONTENTS USING THE DESIGN OF EXAMPLE PROBLEM NO. 3

| Thornthwaite Moisture Index | Clay Content % | Total Highway Deformation (inch) | Total Railroad Deformation (inch) | Differential Deformation (inch) |
|---|---|---|---|---|
| -30 | 30 | 0.138 | 1.343 | 1.204 |
| -10 | 30 | 0.453 | 1.584 | 1.131 |
| +10 | 30 | 0.515 | 1.602 | 1.086 |
| +20 | 30 | 0.487 | 1.586 | 1.099 |
| -30 | 70 | 0.475 | 1.860 | 1.385 |
| -10 | 70 | 0.437 | 1.676 | 1.239 |
| +10 | 70 | 0.293 | 1.429 | 1.137 |
| +20 | 70 | 0.156 | 1.327 | 1.171 |

TABLE 8.- DEFORMATIONS CALCULATED FOR DIFFERENT CLIMATIC ZONES AND CLAY CONTENTS ASSUMSING PERIODIC TRACK RELEVELING

| Thornthwaite Moisture Index | Clay Content % | Deformation of Sublayers (inch) | | Differential Deformation +0.025 inch |
|---|---|---|---|---|
| | | Highway | Railroad | |
| -30 | 30 | 0.138 | 0.092 | 0.671 |
| -10 | 30 | 0.453 | 0.334 | 0.744 |
| +10 | 30 | 0.515 | 0.352 | 0.788 |
| +20 | 30 | 0.487 | 0.336 | 0.776 |
| -30 | 70 | 0.475 | 0.610 | 0.490 |
| -10 | 70 | 0.437 | 0.525 | 0.537 |
| +10 | 70 | 0.293 | 0.179 | 0.739 |
| +20 | 70 | 0.156 | 0.077 | 0.694 |

- it is important for the designer of a crossing to know how frequently the track is releveled since the behavior of the ballast is the most important single feature of grade crossing performance.
- the type of subgrade soil and the climate in which it is found is the next most important feature of grade crossing design. In general, clays are more resilient than silts. Wet climates and warm temperatures require more frequent track releveling and pavement maintenance, as well as initially thicker or stiffer pavements.
- because the ballast settles so much, it is worthwhile to design the adjacent pavement to have a large enough permanent deformation that the differential will not become unacceptable in the period between track releveling programs. This can be done by a careful selection of the thickness of the unstabilized base course used in the pavement.

Because a railroad and its adjacent highway pavement deform at different rates, it is important to emphasize the need to carefully design the crossings for structural compatibility.

Lighter traffic than used in this study will require lighter crossing designs, but the same attention must be given to subgrade soil and the climate in which it is found.

In all of these studies, it is assumed that adequate drainage has been provided. If one wishes to consider the effect of poor drainage, an input suction value of around 10 psi for a heavy clay subgrade should produce the desired effect, since, the suction value of heavy clay at 2% dry of optimum moisture content was found to be 110 psi (17).

# CHAPTER V

## CONCLUSIONS AND RECOMMENDATIONS

Conclusions.- This study establishes a unique design criterion of differential deformation between the railroad pavement and adjacent pavement structures for the design of a highway-railroad grade crossing. Actually, the differential deformation produces the dynamic load in highway traffic which gradually causes the loss of rideability and total grade crossing failure. Different structural dimensions in railroad track and adjacent highway pavements and selection of different construction materials will increase or decrease the differential deformation. This is explained in example problems in Chapter IV. However, individual deformations in each layer are also important design parameters. A design may look promising from the point of view of differential deformation criterion ; but it should be rejected if there is large deformation in an individual layer.

The influence of environmental factors on subgrade materials is included in this study which made the whole design system more general. A study of the influence of environmental factors on the magnitudes of the differential deformation is presented in Chapter IV. There it is illustrated that for a heavy traffic situation in a climatic zone with a Thornthwaite moisture index of -10 and an average subgrade temperature of 72°F, an unstabilized base should be used in the highway pavement. This will allow the highway pavement to deform uniformly with the railroad track. However, in a climatic zone with a higher Thornthwaite moisture index (wetter area) and with higher subgrade temperature, the deformation in the highway pavement would be significantly large, causing a situation that may require the use of a stabilized base or a thicker base in the highway pavement.

When good drainage condition and low water tables are expected at a given location, the suction level in the subgrade will be controlled by the climate. The equivalent suction level

value corresponding to the Thornthwaite moisture index of a particular area is obtained from Figure 28 (Chapter IV). However, a designer can change this value as he wishes in accordance with the expected drainage or water table condition of a particular area of interest. Poor drainage will decrease the suction level, and an input of 10 psi should represent fairly poorly drained conditions.

If some slight descrepancies are observed in the calculated results, they can be explained by the relative inaccuracy of the equations developed in Chapter II to predict the stresses in the lower layers and the deformation in the subgrade. The inaccuracy is especially noticeable in the equations predicting the confining stresses at different depths of the subgrade layer ($R^2$ ranged between 0.2 - 0.6)

Recommendations.- The following recommendations concerning further work to improve on this design system are:

1) Laboratory tests should be designed and conducted to improve the material characterizations of ballast and base course materials and particularly stabilized base course materials.

2) It is necessary to improve the polynomial stress equations for the lower layers. It is believed that improvement of these equations can be achieved by generating more equations with smaller upper and lower limits of the variables.

3) The equations of resilient modulus, permanent strain and temperature effect of subgrade material should be made simpler. It is believed that higher accuracy can be achieved in these equations by using only the most important variables and using constitutive relations that are indicated as important by mixture theory and rate process theory.

4) An iterative scheme should be added to the computer program to automatically search for the optimum design thicknesses under a given condition of traffic, climate and soil type.

5) A technique to achieve the effect of gradual stress built up as the dynamic load increases with the increase in differential deformation should be added to the computer program.

6) Recommended uses of the design system:

   a) This design system can be very effectively used to find the most effective ballast depth in different climatic and soil conditions

   b) Using this design system, performances of presently available commercial crossing materials can be predicted.

# APPENDIX A

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## APPENDIX B

## NOTATION

$A0, A1, A2$ = coefficients of polynomial stress equations

$a_0$-$a_8$ = coefficients of the equation for temperature correction factors

$a^0, a^1, a^2$ = exponents in polynomial stress equations

$b, c, d, e$ = exponents in temperature correction factor equation

$b^0, b^1, b^2$ = exponents in polynomial stress equations

$C$ = cohesion, psi

$CLAY$ = clay fraction in soil in decimal form

$C1, C2$ = coefficients of polynomial stress equations

$c^0, c^1, c^2$ = exponents in polynomial stress equations

$d^0, d^1, d^2$ = exponents in polynomial stress equations

$\frac{D}{D_0}$ = deviator stress ratio

$E_i$ = initial tangent modulus, psi

$E1, E2, E3$ = modulus of elasticity, psi

$e^0, e^1, e^2$ = exponents in polynomial stress equations

$f_{M_R}$ = temperature correction factor for resilient modulus

$f_{\varepsilon_p}$ = temperature correction factor for permanent strain

$H1, H2$ = thickness of layers, inch

$h$ = depth in inches from surface

$h_0, h_i$ = initial suction, psi

$h_f$ = final suction, psi

$h_t$ = test suction, psi

$\frac{h}{h_0}$ = soil suction ratio

$K, K', K1, K2, K3, K4$ = regression constants

$LL$ = liquid limit

$M_R$ = resilient modulus

$N, ND, NK$ = number of load repetition

$\frac{N}{N_0}$ = number of load cycle ratio

$n, n'$ = regression constant

$nS$ = volumetric moisture content

$PL$ = plastic limit

$R$ = ratio of applied deviator stress to deviator stress at failure

$R^2$ = coefficient of determination

$RI$ = roughness index inch/mile

$R_f$ = ratio of compressive strength to an asymptotic stress difference

$S$ = saturation

$SL$ = shrinkage limit

$SLOP$ = rate of increase in permanent strain

$\frac{T}{T_0}$ = temperature ratio

$X_1$-$X_5$ = variables used in statistical experiment design

$x$ = the distance a vehicle travels, miles

$Y$ = quadratic response surface

$y_i$ = measured excursion of rear axle, inches

$(1-n)$ = volumetric soil content

200 sieve = the amount of soil passed the #200 sieve in decimal form

$\alpha$ = level of variables used in statistical experiment design

$\beta_o, \beta_i, \beta_{ii}, \beta_{ij}$ = coefficient of response surface equation

$\gamma_1, \gamma_2, \gamma_3$ = suction ratios

$\varepsilon^p, \varepsilon^p_a, \varepsilon^p_{ND}, \varepsilon^p_{NK}$ = permanent strain

$\theta$ = sum of principal stresses, psi

$\nu_1, \nu_2, \nu_3$ = Poisson's ratio

$\sigma_1$ = major principal stress, psi

$\sigma_m$ = mean stress, psi

$(\sigma_1-\sigma_3)$ = deviator stress, psi

$(\sigma_1-\sigma_3)_f$ = deviator stress at failure, psi

$(\sigma_1-\sigma_3)_{ult}$ = ultimate deviator stress, psi

$\phi$ = angle of internal friction.

# APPENDIX C

## FORTRAN LISTING FOR COMPUTER PROGRAM WITH INPUT AND OUTPUT INFORMATION

---------- INPUT FORMAT ----------

CARD NO 1.  FORMAT(5F10.2)

XE1  IS MODULUS OF TOP LAYER OF HIGHWAY PAVEMENT,PSI
XE2  IS MODULUS OF 2ND LAYER OF HIGHWAY PAVEMENT,PSI *
XE3  IS MODULUS OF SUBGRADE OF HIGHWAY PAVEMENT,PSI *
XH1  IS THICKNESS OF TOP LAYER OF HIGHWAY PAVEMENT,INCH
XH2  IS THICKNESS OF 2ND LAYER OF HIGHWAY PAVEMENT,INCH

CARD NO 2.  FORMAT(6F10.2)

HBASE  IS 0 FOR UNSTABILIZED AND 1.0 FOR STABILIZED BASE IN
       HIGHWAY PAVEMENT
HXC  IS COHESION FOR HIGHWAY BASE,PSI
XPHI  IS ANGLE OF INTERNAL FRICTION FOR HIGHWAY BASE,DEGREES
HRF  IS FACTOR (0.7 - 1.0) FOR BASE MATERIALS
HXK  IS REGRESSION CONSTANT OF MODULUS OF BASE MATERIALS
HXN  IS REGRESSION CONSTANT OF MODULUS OF BASE MATERIALS

CARD NO 3.  FORMAT(3I10,F10.2)

NLDATA IS NUMBER OF LOAD REPETITION FROM WHICH HXK AND HXN IS
       OBTAINED. THIS IS GENERALLY 10000
NLHWAY IS NUMBER OF HIGHWAY LOAD REPETITION
NLRWAY IS NUMBER OF RAILWAY LOAD REPETITION
TEMP  IS AVERAGE SUBGRADE TEMPERATURE,DEG.F.

CARD NO 4.  FORMAT(2F10.2)

ACCLAY IS CLAY CONTENT OF SUBGRADE  IN PERCENT
SUCI70 IS EQ. SUCTION LEVEL FOR SOIL WITH 70 PERCENT CLAY,PSI

CARD NO 5.  FORMAT(5F10.2)

RE1  IS MODULUS OF TOP LAYER OF RAILROAD TRACK,PSI
RE2  IS MODULUS OF 2ND LAYER OF RAILROAD TRACK,PSI *
RE3  IS MODULUS OF SUBGRADE OF  RAILROAD TRACK,PSI *
RH1  IS THICKNESS OF TOP LAYER OF RAILROAD TRACK,INCH
RH2  IS THICKNESS OF 2ND LAYER OF RAILROAD TRACK,INCH

CARD NO 6.  FORMAT(6F10.2)

RBASE  IS 0 FOR UNSTABILIZED AND 1.0 FOR STABILIZED BASE IN
       RAILROAD TRACK
RXC  IS COHESION FOR RAILROAD BASE ,PSI
RPHI  IS ANGLE OF INTERNAL FRICTION FOR RAILROAD BASE,DEGREES
RRF  IS FACTOR (0.7 - 1.0) FOR BASE MATERIAL
RXC  IS REGRESSION CONSTANT OF MODULUS OF BASE MATERIAL
RXN  IS REGRESSION CONSTANT OF MODULUS OF BASE MATERIALS

* INITIAL GUESS IF MATERIAL IS NONLINEAR

INPUT INFORMATION

HIGHWAY PAVEMENT

| THICKNESS (INCH) | | MODULUS VALUES (PSI) | | |
|---|---|---|---|---|
| TOP LAYER | 2ND LAYER | TOP LAYER | 2ND LAYER* | SUBGRADE* |
| 4.00 | 12.00 | 350000.00 | 25000.00 | 15000.00 |

| UNSTABILIZED BASE | | | | |
|---|---|---|---|---|
| COHESION (PSI) | PHI DEG. | RF | K | N |
| 0.00 | 50.00 | 0.705 | 15000.00 | 0.50 |

SUBGRADE
CLAY CONT.= 30.00 PERCENT
EQ.INT.SUC AT 70 CLAY CONT= 57.00 PSI
SUBGRADE TEMP= 72.00 DEG.F

RAILROAD TRACK

| THICKNESS (INCH) | | MODULUS VALUES (PSI) | | |
|---|---|---|---|---|
| TOP LAYER | 2ND LAYER | TOP LAYER | 2ND LAYER* | SUBGRADE* |
| 18.00 | 12.00 | 50000.00 | 120000.00 | 15000.00 |

| STABILIZED BASE | | | | |
|---|---|---|---|---|
| COHESION (PSI) | PHI DEG. | RF | K | N |
| 25.00 | 45.00 | 0.705 | 15000.00 | 0.50 |

* INITIAL GUESS IF MATERIAL IS NONLINEAR

OUTPUT INFORMATION

| STRUCTURE TYPE | LOADIND TYPE | NO OF LOAD REPETITION | DEFORMATIONS (INCH) TOP LAYER | 2ND LAYER | SUBG RADE | TOTAL | DIFFERENTIAL |
|---|---|---|---|---|---|---|---|
| HIGHWAY | HIGHWAY | 1000000 | | 0.100 | 3.779 | 3.879 | |
| RAILROAD | RAILWAY | 1000000 | 0.979 | 0.030 | 1.425 | 2.433 | |
| RAILROAD | HIGHWAY | 1000000 | 0.272 | 0.008 | 1.380 | 1.660 | 0.214 |

```
C     ************************************
C     ********** MAIN PROGRAM ********
C
C     THIS PROGRAM DESIGNS A HIGHWAY-RAILROAD GRADE CROSSING
C     THIS PROGRAM CALCULATES THE DEFORMATIONS OF EACH LAYER OF HIGH-
C     WAY PAVEMENTS AND RAILROAD TRACK SEPERATELY
C     THE DIFFERENCE IN DEFORMATIONS BETWEEN HIGHWAY AND RAILROAD
C     CONTROLS THE DESIGN SYSTEM
C     POSITIVE STRESS MEANS COMPRESSIVE
C     FOLLOWING ARE SOME ASSUMPTIONS REQUIRED IN THE PROGRAM
C     TO OBTAIN THE CALCULATED RESULTS WITHIN REASONABLE LIMITS
C
C    1 AT MID-POINT OF BALLAST LAYER, MINIMUM VALUES OF CONFINING STRESS
C      IS 0.5 PSI AND DEVIATOR STRESS IS 20.0 PSI
C
C    2 MAX. VALUE OF RATIO OF APPLIED DEVIATOR STRESS TO DEVIATOR
C      STRESS AT FAILURE IS 0.75  IN BALLAST LAYER AND
C      MAX. VALUE OF RATIO OF APPLIED DEVIATOR STRESS TO DEVIATOR
C      STRESS AT FAILURE IS 0.999 IN 2ND LAYER
C
C    3 MINIMUM VALUE OF CONFINING STRESS IN 2ND LAYER IS .01 PSI AND
C      MAXIMUM VALUE IS 1.0 PSI  FOR BOTH HIGHWAY AND RAILROAD
C
C    4 MINIMUM VALUES OF DEVIATOR STRESSES IN 2ND LAYER ARE 10.0 PSI
C      AND 5.0 PSI FOR HIGHWAY AND RAILROAD RESPECTIVELY
C
C    5 CONFINING STRESS IN SUBGRADE IS 0.1 PSI MINIMUM AND DEVIATOR
C      STRESS IS 1.0 PSI MINIMUM
C     ************************************************************************
C
C
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,PE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ5 / HXC,HPHI,HRF,HXK,HXN,RXC,RPHI,RRF,RXK,RXN
      COMMON /AZ6/  RCS2(5),RDS2(5),RTS2(5),RCS3(5),RDS3(5),RTS3(5)
     1,RD2(5),RD3(5),RX,RY
      COMMON /AZ7/ CS2(5),DS2(5),TS2(5),CS3(5),DS3(5),TS3(5),D2(5),D3(5)
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      COMMON /AZ10 /DVSTR1,CONF1,DVSTFL,DELP1,XDELP1,PLDEF1,XPLDF1,TOTDF1
      CALL READIN
      CALL OUTPUT
      CALL HWSTRS
      PHI=3.141592654*HPHI/180.
      XC=HXC
      RF=HRF
      XK=HXK
      XN=HXN
      TOTDF2=0.0
      TOTDF3=0.0
      NLANAL=NLHWAY
      DO 1 I=1,3
      SIGMA3=CS2(I)*1.2
      DEVSTR=DS2(I)*1.2
      BASE=HBASE
      IF(BASE.EQ.1.0) EI=XE2
      CALL PRDEF2
```

```
      TOTDF2=TOTDF2 + PLDEF2
      S3=CS3(I)*1.2
      SD=DS3(I)*1.2
      CALL PRDEF3
      TOTDF3=TOTDF3+PLDEF3
    1 CONTINUE
      CALL BALAST
      CALL RWSTRS
      PHI=3.141592654*RPHI/180.
      XC=RXC
      RF=RRF
      XK=RXK
      XN=RXN
      X=RX
      Y=RY
      RTDF2=0.0
      RTDF3=0.0
      NLANAL=NLRWAY
      DO 2 I=1,3
      SIGMA3=RCS2(I)*1.15
      DEVSTR=RDS2(I)*1.15
      BASE=RBASE
      IF(BASE.EQ.1.0) EI=RE2
      CALL PRDEF2
      RTDF2=RTDF2+PLDEF2
      S3=RCS3(I)*1.15
      SD=RDS3(I)*1.15
      CALL PRDEF3
      RTDF3=RTDF3+PLDEF3
    2 CONTINUE
      XXDF2=0.0
      XXDF3=0.0
      NLANAL=NLHWAY
      DO 3 I=1,3
      SIGMA3=RCS2(I)*.44
      DEVSTR=RDS2(I)*.44
      BASE=RBASE
      IF(BASE.EQ.1.0) EI=RE2
      CALL PRDEF2
      XXDF2=XXDF2+PLDEF2
      S3=RCS3(I)*.44
      SD=RDS3(I)*.44
      CALL PRDEF3
      XXDF3=XXDF3+PLDEF3
    3 CONTINUE
      RLW1=PLDEF1+RTDF2+RTDF3
      RLW2=XPLDF1+XXDF2+XXDF3
      HWYTOT=TOTDF2+TOTDF3
      RLWTOT=TOTDF1+RTDF2+RTDF3+XXDF2+XXDF3
      DIFFER=RLWTOT-HWYTOT
      WRITE(6,110) NLHWAY,TOTDF2,TOTDF3,HWYTOT
  110 FORMAT(10X,'HIGHWAY',3X,'HIGHWAY',1X,I10,10X,F6.3,2X,F6.3,1X,F6.3
     1,//)
      WRITE(6,111) NLRWAY,PLDEF1,RTDF2,RTDF3,RLW1
  111 FORMAT(10X,'RAILROAD',2X,'RAILWAY',1X,I10,1X,F7.3,2X,F6.3,2X,
     1 F6.3,1X,F6.3,//)
      WRITE(6,112) NLRWAY,XPLDF1,XXDF2,XXDF3,RLW2,DIFFER
  112 FORMAT(10X,'RAILROAD',2X,'HIGHWAY',1X,I10,1X,F7.3,2X,F6.3,2X,F6.3
     1 ,1X,F6.3,2X,F7.3,//)
      RETURN
      END
```

```
      SUBROUTINE READIN
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ5 / HXC,HPHI,HRF,HXK,HXN,RXC,RPHI,RRF,RXK,RXN
      READ(5,1) XE1,XE2,XE3,XH1,XH2
    1 FORMAT(5F10.2)
      READ(5,2) HBASE,HXC,HPHI,HRF,HXK,HXN
    2 FORMAT(6F10.2)
      READ(5,3)NLDATA,NLHWAY,NLRWAY,TEMP
    3 FORMAT(3I10, F10.2)
      READ(5,4)ACCLAY,SUCI70
    4 FORMAT(3F10.2)
      READ(5,1) RE1,RE2,RE3,RH1,RH2
      READ(5,2) RBASE,RXC,RPHI,RRF,RXK,RXN
      CLAY=ACCLAY/100.0
      SUCI(1)=SUCI70
      SUCF(1)=1.05*SUCI70
      SUCT(1)=1.0*SUCF(1)
      SUCI(2)=0.4545*SUCI70
      SUCF(2)=1.8*SUCI(2)
      SUCT(2)=1.1*SUCF(2)
      SUCI(3)=0.2545*SUCI70
      SUCF(3)=1.4*SUCI(3)
      SUCT(3)=1.3*SUCF(3)
      RETURN
      END



      SUBROUTINE PRDEF2
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      IF(BASE.EQ.1.0) GO TO 15
      EI=XK*SIGMA3**XN
      IF (EI.LT.5000.0) EI=5000.0
   15 CONTINUE
      DVFAIL=(2.0*XC*COS(PHI)+2.0*SIGMA3*SIN(PHI))/(1-SIN(PHI))
      ULTDEV=DVFAIL/RF
      SRATIO=DEVSTR/ULTDEV
      IF(SRATIO.GE.1.0) SRATIO=.999
      DELP=DEVSTR/(EI*(1-SRATIO))
      IF(NLDATA.EQ.NLANAL) GO TO 1
C ***** EQ. THAT FINDS THE VALUE OF DELP FOR NANAL *****
C     EQ. FOR DELP DUE TO NO. OF LOAD APPLICATION DURING DESIGN PERIOD
      SLOPE=.0013507*DEVSTR**1.2623
      IF(DEVSTR.GT.30.0) SLOPE=.00010543*DEVSTR**2.0191
      FNDELP=DELP+(ALOG10(FLOAT(NLANAL))-ALOG10(FLOAT(NLDATA)))*SLOPE
      DELP=FNDELP
    1 DELP=DELP/100.0
      PLDEF2=X*DELP
      RETURN
      END
```

```
      SUBROUTINE OUTPUT
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ5 / HXC,HPHI,HRF,HXK,HXN,RXC,RPHI,RRF,RXK,RXN
      WRITE(6,1717)
 1717 FORMAT(1H1)
      WRITE(6,101)
  101  FORMAT(10X,'INPUT INFORMATION',//,33X,'HIGHWAY PAVEMENT',//)
      WRITE(6,102) XH1,XH2,XE1,XE2,XE3
  102  FORMAT(13X,'THICKNESS (INCH)', T50,'MODULUS VALUES (PSI)',//,
     1 10X,'TOP LAYER',2X,'2ND LAYER',15X,'TOP _AYER',3X,'2ND LAYER*',
     22X,'SUBGRADE*',/, T12,F6.2,T23,F6.2,T45,F10.2,2X,F10.2,2X,F10.2,/)
      IF(HBASE.EQ.1.0) GO TO 201
      WRITE(6,103)
  103  FORMAT(10X,'UNSTABILIZED BASE')
      GO TO 202
  201  WRITE(6,104)
  104  FORMAT(10X,'STABILIZED BASE')
  202  CONTINUE
      WRITE(6,105) HXC,HPHI,HRF,HXK,HXN
  105  FORMAT(10X,'COHESION',4X,'PHI',8X,'RF',8X,'K',8X,'N',/,
     1 11X,'(PSI)',T23,'DEG.',/,10X,F6.2,5X,F6.2,5X,F5.3,2X,F9.2,3X,
     2 F4.2,/)
      WRITE(6,106)ACCLAY,SUCI70,TEMP
  106  FORMAT(10X,'SUBGRADE',/ ,10X,'CLAY CONT.=',F8.2,2X,'PERCENT',/,
     1 10X,'EQ.INT.SUC AT 70 CLAY CONT=', F8.2,2X,'PSI',/, 10X,
     2 'SUBGRADE TEMP=', F8.2,2X,'DEG.F',//)
      WRITE(6,107)
  107  FORMAT(T33,'RAILROAD TRACK',/)
      WRITE(6,102) RH1,RH2,RE1,RE2,RE3
      IF(RBASE.EQ.1.0) GO TO 203
      WRITE(6,103)
      GO TO 204
  203  WRITE(6,104)
  204  CONTINUE
      WRITE(6,105) RXC,RPHI,RRF,RXK,RXN
      WRITE(6,108)
  108  FORMAT(//,10X,' *  INITIAL GUESS IF MATERIAL IS NONLINEAR',//)
      WRITE(6,109)
  109  FORMAT(10X,'OUTPUT INFORMATION',//,T49,'DEFORMATIONS (INCH)',//,
     110X,'STRUCTURE',1X,'LOADIND',2X,'NO OF LOAD',2X,'TOP',5X,'2ND',5X,
     2'SUBG',3X,'TOTAL',3X,'DIFFERE',/,12X,'TYPE',6X,'TYPE',3X,
     3 'REPETITION',2X,'LAYER',3X,'LAYER',3X,'RADE',11X,'NTIAL',/)
      RETURN
      END
```

```
C
      SUBROUTINE XMOD3

      REAL MR
      DIMENSION EP(10),MR(10),DATA(15),Z(10),ZZ(10)
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      SM=(SD+3.0*S3)/3.0
      DO 5 K=1,3
      D1=SUCI(K)
      D2=SUCT(K)
      D3=SUCF(K)
      D7=SD
      D8=SM
      D9=S3
      D10=10000
      GO TO (10,20,30),K
10    CONTINUE
      XSATUR=94.00
      XPRSTY=0.50319
      D4=XSATUR
      D5=XPRSTY*XSATUR/100.0
      D6=1-XPRSTY
      MR(1)=-4791.99 -27272.4 *( (D3/D1)**.2 *D10**.084) * ( 1-45.0169
     1*D6**3.6*(1-3.733*D7**(-.6)) +1.706*10.**(-7.)*D4**3.6 *(1-5.0763
     2*D7**(-.6) -0.1288*D8**(-.27) ) +0.05999*D5**(-3.3)
     3*(1-5.8416*D7**(-.6)) )
      IF(MR(1).GT.25000.) MR(1)=25000.0
      IF(MR(1).LT.5000.0) MR(1)=5000.0
      GO TO 5
20    CONTINUE
      XSATUR=89.31
      XPRSTY=0.36050
      D4=XSATUR
      D5=XPRSTY*XSATUR/100.0
      D6=1-XPRSTY
      MR(2)=7980.89 + 2981.64*( (D3/D1)**.2 *D10**.145)*(1+64.397
     1*D6**3.3 *(1-4.2008*D7**(-.6)) -2.002*10.**(-3.)*D4**2.0*(1-3.7228
     2*D7**(-.6) -0.1639*D8**(-.23) ) -0.1974*D5**(-2.25) *(1-4.2766
     3*D7**(-.6) ) )
      IF(MR(2).GT.20000.0) MR(2)=20000.0
      IF(MR(2).LT.5000.0) MR(2)=5000.0
      GO TO 5
30    CONTINUE
      XSATUR=64.04
      XPRSTY=0.3482
      D4=XSATUR
      D5=XPRSTY*XSATUR/100.0
      D6=1-XPRSTY
      MR(3)=-1827.72 +171705.0*( (D3/D1)**.2* D10**.081) * (1+0.6566
     1*D6**1.4 *( 1-4.4849*D7**(-.16)) +64.6522*D4**(-.26) *(1-1.6108
     2*D7**(-.16) -.001155*D8**.063)- 14.8816*D5**(-.3)*(1-1.5899*D7**
     3(-.16) ) )
      IF(MR(3).GT.15000.0) MR(3)=15000.0
      IF(MR(3).LT.5000.0) MR(3)=5000.0
 5    CONTINUE
      A=.5032258*MR(1) -2.37691*MR(2) +2.8736842*MR(3)
      B=-3.8064516*MR(1) +15.280135*MR(2) -11.473684*MR(3)
      C=6.4516129*MR(1) -16.977928*MR(2) +10.526315*MR(3)
      XMR3=A+ B*CLAY +C*CLAY**2
      RETURN
      END
```

```
      SUBROUTINE  PRDEF3
      DIMENSION EP(10),MR(10),DATA(15),Z(10),ZZ(10)
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      SM=(SD+3.0*S3)/3.0
      DO 5 K=1,3
      D1=SUCI(K)
      D2=SUCT(K)
      D3=SUCF(K)
      D7=SD
      D8=SM
      D9=S3
      D10=NLANAL
      GO TO (10,20,30),K
10    CONTINUE
      EP(1)=-.000186 -.000443*D10**.45 *(1-63.0264*(D2/D3)**.61
     1*( (1/(D9*D3))**.25 -.09398*D7**.24*(1/D3)**.24) +123.8399*
     2( 1/(D8*D3))**.4 -5.9323*D7**.24*(1/D3)**.24 )
      IF(EP(1).LE.0.0) EP(1)=0.0
      GO TO 5
20    CONTINUE
      EP(2)=0.015196 -0.000254*D10**.63* (1-24.62205*(D2/D3 )**.5*
     1( (1/(D9*D3) )**.38 -.01297*D7**1.58*(1/D3)**.54) +61.1811*
     2(1/(D8*D3))**.6 -0.52205*D7**1.58*(1/D3)**.54 )
      IF(EP(2).LE.0.0) EP(2)=0.0
      GO TO 5
30    CONTINUE
      XSATUR=64.04
      XPRSTY=0.3482
      D4=XSATUR
      D5=XPRSTY*XSATUR/100.0
      D6=1-XPRSTY
      EP(3)=0.04076-1.2679*( (1/D2)**.65*D10**.395 ) *(1-1.2067*10.**
     1(-15.) *( D4**5.35/D5**5.1*(1/D6)**10.4 ) )
      IF(EP(3).LE.0.0) EP(3)=0.0
5     CONTINUE
      D=.5032258*EP(1) -2.37691*EP(2) +2.8736842*EP(3)
      E=-3.8064516*EP(1) +15.280135*EP(2) -11.473684*EP(3)
      F=6.4516129*EP(1) -16.977928*EP(2) +10.526315*EP(3)
      DELP=D+ E*CLAY +F*CLAY**2
      DELP=DELP/100.0
      CALL TEMPDF
      DELP=FDELP*DELP
2     PLDEF3=DELP*Y
      RETURN
      END
```

```
      SUBROUTINE XMOD2
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,PF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      XMR2=XK*S3**XN
      IF(XMR2.LT.5000.0) XMR2=5000.0
      RETURN
      END
```

```
      SUBROUTINE HWSTRS
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ5 / HXC,HPHI,HRF,HXK,HXN,RXC,RPHI,RRF,RXK,RXN
      COMMON /AZ7/ CS2(5),DS2(5),TS2(5),CS3(5),DS3(5),TS3(5),D2(5),D3(5)
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      TOLIM2=1000.0
      TOLIM3=1000.0
      X=XH2/3.0
      Y= (80.0-(XH1+XH2) )/3.0
      XK=HXK
      XN=HXN
      H1=XH1
      H2=XH2
      E1=XE1
      E2=XE2
      E3=XE3
      RNE2=XE2
      RNE3=XE3
      ICOUNT=1
      TOTE2=0.0
      TOTE3=0.0
   44 XXE2=0.0
      XXE3=0.0
      DO 2 I =1,3
      D2(I)= XH1 + ( (I-1)+0.5)*X
      D3(I)=(XH1+XH2) +( (I-1) +0.5)*Y
C     ******* SIGN CONVENTION - COMPRESSIVE STRESS IS POSITIVE *********
C  ***  EQUATION FOR SIGMA1 **********************
      A0=1.67219 +H1**(-.706)*E1**(-.392)*E2**(.487) *(773.523*H2**
     1(-.271) -295.904*E3**(.0076)) +H1**(-.706)*E2**(.487) *(-3.88815*
     2 H2**(-.271) +2.11753)
      A1=-.25409 +H1**(-1.068)*H2**(-.728)*E2**(.642)*E3**(-.067)*
     1(-112.814*E1**(-.428) +.197376) +H1**(-1.068)*E2**(.642)*(5.68131
     2* E1**(-.428) -.0206118)
      A2=.00835535 +H1**(-1.45)*H2**(-1.22)*E2**(.685) *(17.7337
     1*E1**(-.446)*E3**(-.08) -.0117177) +H1**(-1.45)*E2**(.685)
     2*(-.243798*E1**(-.446) +.000723757)
      SIGMA1= A0+A1*D2(I)+A2*D2(I)**2
C  *** EQUATION FOR SDEV2 ***********
      A0= -2.40236 + H2**(-.377) *E2**(.529) * E3**(-.012) *( 1946.78*
     1 H1**(-.383)*E1**(-.327)-892.3982*E1**(-.327)-28.9221*H1**(-.383))
     2+E1**(-.327)*E2**(.529)*(-751.672*H1**(-.383)+396.368*E3**(-.012))
     3+E2**(.529)*(11.4648*H1**(-.383)+12.0964*H2**(-.377)-5.38283)
      A1=.148485+H1**(-.61)*H2**(-.869)*E2**.733*(-58.7292*E1**(-.369)*
     1E3**(-.1)+26.475*E1**(-.369)-.0125271*E3**(-.1))  +H2**(-.869)*
     2E2**(.733)*(-.278655*E1**(-.369)+.22897*E3**(-.1)-.0850578)
      A2=.00790123 +H2**(-1.521)*E1**(-.356)*E2**(.905)*(-1.29101*H1**
     1 (-.98)*E3**(-.239)+.39407*H1**(-.98)+.504433*E3**(-.239))
     2+E1**(-.356)*E2**(.905)*(-.0685944*H2**(-1.521)-.00379577*
     3H1**(-.98)+.000558257)
      SDEV2= A0 -A1*D2(I) +A2*D2(I)**2
      SCONF2=SIGMA1-SDEV2
      IF(SDEV2.LE.10.0) SDEV2=10.0
      IF(SCONF2.GT.1.0) SCONF2=1.0
      IF(SCONF2.LE..01) SCONF2=.01
      S3=SCONF2
      SD=SDEV2
      IF(HBASE.EQ.1.0) GO TO 15
      CALL XMOD2
```

```
      GO TO 14
 15   XMR2=XE2
 14   CONTINUE
      XXE2=XXE2+XMR2
      CS2(I)=S3
      DS2(I)=SDEV2
      TS2(I)=SDEV2+3.0*SCONF2
C  *** EQUATION FOR SCONF3 ********
      A0=.233285+ H1**(-.61)*H2**(-.706)*E3**(.25)* (41.011*E1**(-.254)
     1 +108.767*E2**(-.184) -14.6131)+H1**(-.61)*H2**(-.706)*E1**(-.254)
     2 *(-24986.4*E2**(-.184)+3295.44)
      A1=.013372 +H1**(-.799)*H2**(-.905)* (-4421.43*E1**(-.289)
     1*E2**(-.24) +5.32534*E2**(-.24)*E3**(.34) -.380767*E3**(.34)
     2+366.521*E1**(-.289) ) +H1**(-.799)*(.0053385*E3**(.34)-.200216)
      A2=.00011311 + H1**(-.918)*H2**(-1.03)*E3**(.392)* (-2.36541
     1*E1**(-.309)*E2**(-.272) +.13664*E1**(-.309)+.107941*E2**(-.272)
     2-.00546569) +H1**(-.918)*H2**(-1.03)*(-2.14616*E2**(-.272)
     3 +.110538) -.000540568*H1**(-.918)
      SCONF3= A0-A1*D3(I)+A2*D3(I)**2
      IF(SCONF3.LE.0.1)  SCONF3=.1
C  *** EQUATION FO SDEV3 ***********
      A0=.339135 + E1**(-.222)*E3**(.645) * (52.3428*H1**(-.545)
     1 *E2**(-.233) -21.6485*H1**(-.545)*H2**(-.465)+57.9236*H2**(-.465)
     2*E2**(-.233) )+ E3**(.645)*(.711581*H1**(-.545)*H2**(-.465)
     3 -2.29282*E2**(-.233) +.0807088)
      A1=-.0013966 + H1**(-.666)*H2**(-.6)*E3**(.759) * (7.37209*
     1E1**(-.255)*E2**(-.249) -.379944*E2**(-.249) -.436305*E1**(-.255))
     2+H1**(-.656)*E3**(.759)* (.0178911*H2**(-.6)+.0252902*E2**(-.249))
     3+E3**(.759)* (.00192404*H2**(-.6) -.000487606)
      A2=-.0000109923+H1**(-.738)*H2**(-.692)*E3**(.809)*(.0635893*
     1E1**(-.272)*E2**(-.248) -.00383969*E1**(-.272)-.00256485*
     2E2**(-.248)) +H1**(-.738)*E3**(.809)*(.000127151*H2**(-.692) +
     3.000122254*E2**(-.248))+E3**(.809)*(.00001019*H2**(-.692)
     4 -.0000020902)
      SDEV3=A0-A1*D3(I)+A2*D3(I)**2
      IF(I.EQ.1.AND.SDEV3.LE.2.0) SDEV3=2.0
      IF(I.EQ.2.AND.SDEV3.LE.1.5) SDEV3=1.5
      IF(I.EQ.3.AND.SDEV3.LE.1.0) SDEV3=1.0
      CS3(I)=SCONF3
      DS3(I)=SDEV3
      TS3(I)=SDEV3+3.0*SCONF3
      S3=CS3(I)
      SD=DS3(I)
      SM=TS3(I)/3.0
      CALL XMOD3
      XXE3=XXE3+XMR3
  2   CONTINUE
      E2=XXE2/3.0
      E3=XXE3/3.0
      TOTE2=TOTE2+E2
      TOTE3=TOTE3+E3
      IF(ICOUNT.GT.10) E2=TOTE2/ICOUNT
      IF(ICOUNT.GT.10) E3=TOTE3/ICOUNT
      IF(ABS(E2-PNE2).GT.TOLIM2) GO TO 41
      IF(ABS(E3-PNE3).GT.TOLIM3) GO TO 41
      GO TO 42
 41   PNE2=E2
      PNE3=E3
      ICOUNT=ICOUNT+1
      GO TO 44
 42   CONTINUE
      RETURN
      END
```

```
      SUBROUTINE RWSTRS
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ2/ BASE,XC,PHI,RF,XK,XN,HBASE,RBASE
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ5 / HXC,HPHI,HRF,HXK,HXN,RXC,RPHI,RRF,RXK,RXN
      COMMON /AZ6/  RCS2(5),RDS2(5),RTS2(5),RCS3(5),RDS3(5),RTS3(5)
     1,RD2(5),RD3(5),RX,RY
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,EI
      TOLIM2=1000.0
      TOLIM3=1000.0
      RX=RH2/3.0
      RY= (90.0-(RH1+RH2))/3.0
      XK=RXK
      XN=RXN
      H1=RH1
      H2=RH2
      E1=RE1
      E2=RE2
      E3=RE3
      RNE2=RE2
      RNE3=RE3
      ICOUNT=1
      TOTE2=0.0
      TOTE3=0.0
 44   RRE2=0.0
      RRE3=0.0
      DO 1  I=1,3
      RD2(I)= RH1+((I-1)+0.5)*RX
      RD3(I)=(RH1+RH2) +((I-1)+0.5)*RY
C     ***  SIGN CONVENTION *** COMPRESSIVE STRESS IS POSITIVE ************
C     *** EQUATION FOR  RCONF2  ******************
C
      A0=1.78920 +H1**(.522) *H2**(-1.385) *E2**(1.155) *(-.00536975 *
     1 E1**(.250) *E3**(-.186) +.000830849 *E1**(.250) +.128764*E3**
     2 (-.186) -.0200543) + H2**(-1.385) *E2**(1.155) *( +.0243088 *
     3 E1**(.250) *E3**(-.186) -.00372796 * E1**(.250) -.580973 * E3**
     4  (-.186) +.090371)
      A1=-.111305 +H2**(-1.363) *E2**(1.299) *E3**(-.325) *(.0774579 *
     1 H1**(-.434) *E1**(-.156) -.000703219* H1**(-.434) -.0214913 *
     2 E1**(-.156) ) +H2**(-1.363) *E2**(1.299) *( -.000551022 *H1**
     3 (-.434) +.000150856)
      A2=.00714227 +H1**(-1.59) *E2**(1.456) *E3**(-.484) *( .163560 *
     1 H2**(-1.368) *E1**(-.345) -.00176703 *H2**(-1.368) -.00626615 *
     2 E1**(-.345) ) +H1**(-1.59) *E2**(1.456) *( .0000921984 * E3**
     3(-.484) + .0000053247 1 * E1**(-.345))-.0000000107315*E2**(1.456)
      RCONF2=-(A0+A1*RD2(I)+A2*RD2(I)**2)
C     *** EQUATION FOR  RDEV2   ******************
C
      A0=5.23725 +H1**(-.859) *E2**(.65) *E3**(-.08) *(516.472*H2**
     1 (-.504) *E1**(-.285) -137.697*E1**(-.285) -13.0602*H2**(-.504)
     2  +4.41353) -.00706606*E2**(.65)
      A1=-.389912 +H1**(-1.482) *E2**(.855) *E3**(-.168) *(.0592021 *
     1 H2**(-.847) *E1**(.325) -3.75471*H2**(-.847) -.0067074*E1**
     2 (.325) +.343391) +.000497097*E2**(.855)
      A2=-.00379223 +H1**(-2.152) *E2**(1.025) *E3**(-.277) *( 17.8758*
     1 H2**(-1.153) *E1**(-.357) -.14552*H2**(-1.153) -.88369* E1**
     2 (-.357) +.0108234) +5.27254*H1**(-2.152)
      RDEV2=A0+A1*RD2(I)+A2*RD2(I)**2
```

```
      IF(RCONF2.GT.1.0) RCONF2=1.0
      IF(RCONF2.LE..01) RCONF2=.01
      IF(RDEV2.LE.5.0) RDEV2=5.0
      S3=RCONF2
      SD=RDEV2
      SM=(SD+3.0*S3)/3.0
      IF(RBASE.EQ.1.0) GO TO 15
      CALL XMOD2
      GO TO 14
   15 XMR2=RE2
   14 CONTINUE
      RRE2=RRE2+XMR2
      RCS2(I)=RCONF2
      RDS2(I)=RDEV2
      RTS2(I)=(SD+3.0*RCONF2)
C     *** EQUATION FOR  RCONF3  ******************
C
      A0=.229435 + H1**(-.98) *E1**(-.344) *E2**(-.028) *(-331576.0 *
     1 H2**(-.433) +29371.8 *E3**(.151) ) +H1**(-.98) *E1**(-.344) *
     2 (245758.0* H2**(-.433) -21436.1* E3**(.151) ) -7.97399*H1**(-.98)
      A1=-.00315258 +H1**(-1.367) *E1**(-.432) *E2**(-.048) *(37214.9 *
     1 H2**(-.571) -1118.31*E3**(.232) ) +H1**(-1.367) *E1**(-.432) *
     2 (-22232.2*H2**(-.571) +651.612*E3**(.232) ) +.342981*H1**(-1.367)
      RCONF3=A0+A1*RD3(I)
      IF(RCONF3.LE.0.1 ) RCONF3=0.1
C     *** EQUATION FOR  RDEV3  ******************
C
      A0=-.135338 +H1**(-.6) *H2**(-.145) *E3**(.684) *(134.041
     1 *E1**(-.318) *E2**(-.106) - 45.5381*E1**(-.318)-2.71228*E2**
     2 (-.106)+1.00933) +E3**(.684)*(.85696*E1**(-.318)-.0195183)
      A1=.00375682 +H1**(-.667) *H2**(-.198) *E3**(.817) *(-1.46644*
     1 E1**(-.319) *E2**(-.115) +.437029*E1**(-.319) +.0301024 *E2**
     2 (-.115) -.00963337) +E3**(.817) *(-.00412002*E1**(-.319)
     3 +.000097604)
      A2=-.000019561+H1**(-.711) *H2**(-.26) *E3**(.855) *(.00566798 *
     1E1**(-.288) *E2**(-.11) -.00160626*E1**(-.288) -.000170594 *
     2E2**(-.11) +.000018243) -.000000048915 7*E3**(.855)
      RDEV3=A0+A1*RD3(I)+A2*RD3(I)**2
      IF(I.EQ.1.AND.RDEV3.LE.2.0) RDEV3=2.0
      IF(I.EQ.2.AND.RDEV3.LE.1.5) RDEV3=1.5
      IF(I.EQ.3.AND.RDEV3.LE.1.0) RDEV3=1.0
      RCS3(I)=RCONF3
      RDS3(I)=RDEV3
      RTS3(I)=RDEV3+3.0*RCONF3
      S3=RCS3(I)
      SD=RDS3(I)
      SM=RTS3(I)/3.0
      CALL XMOD3
      RRE3=RRE3+XMR3
      E2=RRE2/3.0
      E3=RRE3/3.0
      TOTE2=TOTE2+E2
      TOTE3=TOTE3+E3
      IF(ICOUNT.GT.10) E2=TOTE2/ICOUNT
      IF(ICOUNT.GT.10) E3=TOTE3/ICOUNT
      IF(ABS(E2-RNE2).GT.TOLIM2) GO TO 41
      IF(ABS(E3-RNE3).GT.TOLIM3) GO TO 41
      GO TO 42
   41 RNE2=E2
      RNE3=E3
      ICOUNT=ICOUNT+1
      GO TO 44
   42 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE BALAST
      COMMON /AZ1/ XE1,XE2,XE3,XH1,XH2,RE1,RE2,RE3,RH1,RH2
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ10/DVSTR1,CONF1,DVSTFL,DELP1,XDELP1,PLDEF1,XPLDF1,TOTDF1
      D=RH1/2.0
C     DEVIATOR STRESS IN THE BALLAST LAYER
C
      H1=RH1
      H2=RH2
      E1=RE1
      E2=RE2
      E3=RE3
      A0=6.90067 + H1**(-.221) *E1**(-.131) *E2**(.046) *E3**(.094)
     1*(53278.1 *H2**(-.057) -46431.5) + H1**(-.221) *E1**(-.131) *E2
     2**(.046) *(-130145.0*H2**(-.057) + 113842.) + 116.334*H1**(-.221)
      A1=-.303037+H1**(-.829) *E1**(-.129) *E2**(.039) *E3**(.065)
     1*(-131815.0 *H2**(-.049) +117753.0) +H1**(-.829) *E1**(-.129) *E2
     2**(.039) *(246473.0*H2**(-.049) -220326.0) -133.453*H1**(-.829)
      A2=.0443057 + H1**(-1.77) *H2**(-.075) *E2**(-.013) *(-348002. *E1
     1**(-.025) +202822.0*E3**(.025)) + H1**(-1.77) *H2**(-.075)*(301938
     2.0*E1**(-.025) -175891.0 *E3**(.025)  )
      DVSTR1=A0+A1*D+A2*D**2.0
      IF(DVSTR1.LT.20.0) DVSTR1=20.0
C
C     CONFINING STRESS AT MID-POINT OF BALLAST
      C1=.664423+H1**(-5.135) *H2**(-1.83) *E1**(1.01) *E2**(.655)*
     1 (-12.8779*E3**(.23) +77.5752) +H1**(-5.135) *E1**(1.01) *E3**
     2 (.23) *(7724.05 *H2**(-1.83) +.0988775*E2**(.655) )
      C2=H1**(-5.135) *H2**(-1.83) *(-31632.5* E1**(1.01) +3824180.0* E2
     1**(.655) -1974900000.0) +H1**(-5.135) *E1**(1.01) *(-0.544612 *E2
     2**(.655) -256.957) -46242800.0 *H1**(-5.135)
      CONF1=C1+C2
      IF(CONF1.LT.0.5) CONF1=0.5
      DVSTFL=36.7501*CONF1**.5352
      RATIO=DVSTR1/DVSTFL
      IF(RATIO.GT..75) RATIO=.75
      NLANAL=NLRWAY
      DELP1=10.**(-1.8688+0.1666* ALOG10(FLOAT(NLANAL))+2.4048*RATIO)
      DELP1=DELP1/100.0
      PLDEF1=DELP1*RH1
      DVSTR1=DVSTR1*0.4
      CONF1=CONF1*0.4
      DVSTFL=36.7501*CONF1**.5352
      RATIO=DVSTR1/DVSTFL
      IF(RATIO.GT..75) RATIO=.75
      NLANAL=NLHWAY
      XDELP1=10.**(-1.8688+0.1666* ALOG10(FLOAT(NLANAL))+2.4048*RATIO)
      XDELP1=XDELP1/100.0
      XPLDF1=XDELP1*RH1
      TOTDF1=PLDEF1 +XPLDF1
      RETURN
      END
```

```
      SUBROUTINE   TEMPDF
      COMMON/AZ3/ACCLAY,CLAY,SUCI70,SUCI(3),SUCT(3),SUCF(3)
      COMMON /AZ4/ TEMP,NLDATA,NLANAL,NLHWAY,NLRWAY,TOLIM2,TOLIM3
      COMMON /AZ8/ SIGMA3,DEVSTR,S3,SD,SM,X,Y,XD,XD2,XD3
      COMMON /AZ9/ XMR2,XMR3,FXMR3,PLDEF2,PLDEF3,DELP,FDELP,FI
      DIMENSION FD(3),XFD(3),DIVFCT(3)
      DO 1 K=1,3
      RD=SD/13.7
      RT=TEMP/72.0
      RN=NLANAL/10000.
      GO TO(10,20,30),K
10    CONTINUE
      RS=SUCI(K)/110.0
      XCLAY=.70
      XSL=.14
      P200=.91
      GO TO 15
20    CONTINUE
      RS=SUCI(K)/50.0
      XCLAY=.39
      XSL=.14
      P200=.71
      GO TO 15
30    CONTINUE
      RS=SUCI(K)/28.0
      XCLAY=.2
      XSL=.23
      P200=.72
15    CONTINUE
      B=0.6761-0.2384*(1/XCLAY)
      C=-1.7043+1.9130*P200
      D=2.3620-0.4128*(1/XCLAY)
      E=0.3716+.1700*XCLAY
      A0=-114.111+159.213*P200
      A1=119.823-166.053*P200
      A2=-81.345-41.866  *(1/ALOG10(XCLAY))
      A3=0.7882+1.4700*XSL
      A4=-0.0663+1.5214*P200
      A5=-0.2791+1.7426*P200
      XFD(K)=A0+A1*RS**C+A2*RT**D*RN**E*(1-A3*RS**C+A4*RS**C*RD**B
     1  -A5*RD**B)
      DIVFCT(K)=A0+A1*RS**C+A2*1.**D*RN**E*(1-A3*RS**C+A4*RS**C*RD**B
     1  -A5*RD**B)
      FD(K)=XFD(K)/DIVFCT(K)
      FD(K)=ABS(FD(K))
1     CONTINUE
      XCA=.5032258*FD(1)-2.37691*FD(2)+2.8736842*FD(3)
      XCB=-3.8064516*FD(1)+15.280135*FD(2)-11.473684*FD(3)
      XCC=6.4516129*FD(1)-16.977928*FD(2)+10.525315*FD(3)
      FDELP=XCA+XCB*CLAY+XCC*CLAY**2
      RETURN
      END
```