AN EVALUATION OF FLEXIBLE PAVEMENT PERFORMANCE ON
THE BASIS OF DEFLECTION BASINS USING ILLIPAVE PROGRAM

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Master of Science

By
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1. INTRODUCTION

Pavements are extremely complex systems involving the interaction of a number of complex and interrelated variables. Their properties vary with time, traffic condition, repetition of loading and environmental condition. Therefore, a good approach must be used to analyze the pavement performance. Several computer programs have been developed in the past two decades to predict pavement system performance. Some of the computer programs use a linear approach and the other use a nonlinear approach.

The objective of this study is to calculate pavement response based on field deflections using the Falling Weight Deflectometer. Falling Weight Deflectometer (FWD) is one of many nondestructive testing (NDT) devices. Specific models include Dynaflect, Benkelmen Beam, and Roar Rater. An NDT is relatively simple to operate, it reduces testing time, and it is both economical and nondestructive. Using the FWD, deflections are measured at seven different sensors from the applied load.

Many Engineering problems have been successfully solved by the finite element method. Using finite element method, E.L Wilson [1] initially developed the pavement analysis model to back calculated material layer properties. In later years it has been modified by Barkdale, Duncan, Monismith and Wilson, who comprise the research staff of the Construction
Engineering Laboratory at Champaign, Illinois. This current version is called Illipave and according to Figuerua, Hoffman and Thompson, Illipave has provided reliable and realistic predictions of pavement system.

When all FWD deflections data are available. The pavement response can be analyzed with the use of a selected layered model to match up field deflections. The pavement material properties are back calculated and, in turn, represent the layer material rather than the overall pavement properties. The material properties can be evaluated for different seasons and for different loading.

One of the most important aspects of Illipave is an ability to deal with both linear and nonlinear stress-strain behavior of component pavement materials. In a flexible pavement system, subgrades frequently affect the pavement performance. This subgrade soil varies greatly and consists of vastly different materials. Because of that, Illipave treats subgrade as nonlinear stress-dependent materials. Mohr-Coulomb failure criteria have been used to define the strength of the subgrade material. Four different material models are available as an input of modulus function for each layer of the pavement. Further discussion is available from reference 1, Illipave User Manual.

Based on the interpretation of the deflection basin, other important features can be found such as stress and strain, predicting service life of the pavement and pavement maintenance.
In general, the structure of this thesis is as follows:

Chapter I: Introduction
The general concept of nondestructive testing, how to get field deflection and the Illipave program.

Chapter II: Literature review
Recent research regarding the development of structural system evaluation, definitions of pavement types and material characteristics are given. The basic differences between highway and airport pavement are described.

Chapter III: Data description
The method of measurement of field deflection is described and as well as an analysis of the data.

Chapter IV: Illipave program background
Illipave is discussed in detail along with an example of the input program.

Chapter V: Evaluation of the results
An comparison between field deflection and Illipave deflection is given along with a discussion of the results.

Chapter VI: Summary and conclusion
1. A HISTORY OF THE DEVELOPMENT OF THE PAVEMENT STRUCTURAL SYSTEM

Between the period of 1920 and 1940 engineers concentrated more in evaluating structural properties of soil to analyze or design pavement structural systems. A large amount of survey data of soil was available during this period of time and from all these available data, highway engineers were able to correlate the pavement performance with the actual subgrade type. However a pavement system does not depend solely on subgrade type but is an extremely complex system involving the interaction of many variables. Their performance is influenced by several factors: environment, traffic loading, years of service and material properties.

The first major road test was conducted in Illinois in 1920 [3] and it was called "Bates Experimental Road". Various types of materials including Asphaltic concrete, Portland cement concrete and brick were used in the road test. Many engineers used the Bates Experimental Road's results for many years.

Then in 1941 in Maryland a road test was constructed over a 1.1 mile section of rigid concrete pavement. Other tests for evaluating flexible
pavement were conducted in Idaho under the WASHO road test.

Beginning in the 1950's when both automobile and aircraft traffic began to increase rapidly, a lot of past statistical and road test data became absolute. Severe cracking was very common in some highways and airport pavements. Due to this condition, in 1951 another road test was constructed in Ottawa, Illinois and known as the AASHO road test. It used asphaltic and concrete materials. The purpose of AASHO road test was to find the effect of pavement thickness on pavement performance and serviceability. In addition to the AASHO road test, the Federal Highway Administration sponsored a considerable amount of pavement performance research throughout the United States. These pavements were evaluated under various conditions, such as traffic/loading conditions, seasonal variation in climate and soil variety. The results of these field tests have had a major influence on today's pavement evaluation and design.

Pavement design and analysis currently in use depends on empirical (i.e. observation) knowledge rather than theoretical. Theoretical is rather difficult because pavement performance does not always behave the same under certain conditions. Application of one condition to other sets of conditions is quite difficult.

Nobody can say exactly when engineers became interested in using and developing the mechanical approach (deformation, stress, strain) to evaluate pavement. The first publication was made by Burmister in 1943,
but the result was limited only to three layers of pavement. Material properties were very difficult to measure and design and evaluation criteria was not available.

In 1962, when the first International Conference on the Structural Design of Asphalt Pavement was held at the University of Michigan, researchers began to consider the mechanistic approach again. As the result, in 1964 Dorman and Metcalf published the first comprehensive and detailed design method using the mechanistic approach. Since that time, several publications and methods have been proposed for evaluating the pavement structural system.

Lately, as computers have became more widely used in this field, a variety of computer programs were made available for the solution of pavement structural system. Most of these programs are based on layered systems and finite elements. The finite element is considered more powerful because it can handle a greater number of variables. A finite number of elements makes it possible to assign different material properties depending on the pavement cross section. It can be applicable to any kind of pavement cross section or condition. This finite element approach will be discussed in greater detail later on in addition to using it to evaluate flexible pavement.
2. CRITERIA AND DEFINITIONS OF PAVEMENT TYPES

Originally, pavement has been divided into 2 categories: (See Figure 1)

1. Flexible pavement: A relatively thin surface with base course and subbase course underneath. All three of these rest upon the compacted subgrade.

2. Rigid pavement: This consist of portland cement concrete built over subgrade. It may or may not have a base course.
(a)

(b)

Figure 1
Component of (a) flexible and
(b) rigid pavement
The basic difference between flexible and rigid pavement is how the pavement distributes the load over the subgrade. Since flexible pavement consists of a series of layers with the highest quality of material in the top, it tends to distribute the load over the subgrade. From Figure 2, we can see how the load is distributed over the subgrade. For a well compacted subgrade layer, the angle is assumed to be over 45°, but this angle will vary according to material components of the surface and base course.

![Diagram of load distribution through flexible pavements](image)

Today, loads are applied over a larger area under pneumatic tires.
and this will affect load distribution over the subgrade as shown in Figure 3. The Load is distributed over a larger area of subgrade which under the affect of tire contact on the load distribution is comparable to the classical assumption.

![Figure 3](image)

**Figure 3**

Effect of vehicle tire on load distribution

Rigid pavement on the other hand, tends to distribute the load over the slab because of the slab's rigidity and high modulus of elasticity.
For this reason, the subgrade has little, if any, influence on pavement performance.

Nowadays, pavement is not just flexible or rigid. There are at least 2 other models of pavement types:

1. Full depth asphalt pavement: This type is composed of two layers asphalt concrete and subgrade. It is similar to flexible pavement but may behave like rigid pavement because of the asphalt concrete's high stiffness.

2. Stabilized pavement: This is composed of three layers: either a thin surface treatment or a thin asphalt concrete layer; and a stabilized layer over the subgrade. In addition, it may behave like flexible pavement.
3. MATERIAL CHARACTERIZATION OF PAVEMENT COMPONENTS

Understanding material characterization is very important, as it helps engineers predict how the pavement will behave under specific kinds of materials. Since flexible pavement is our main concern, typical flexible components will be discussed here.

1. BITUMINOUS MATERIALS FOR ASPHALT SURFACES AND BINDER COURSES

There are two types of bituminous materials: asphalts and tars. According to Asphalt Handbook [4], asphalt is defined as a powerful cement, readily adhesive, highly waterproof and durable. It is not affected by most acids, alkalies and salts, but it will liquefy under application of heat. Asphalts are commonly obtained by distillation of petroleum. This process will produce either asphalt cement (AC) or a liquid asphalt. The differences between these two are:

a) Asphalt cement is refined asphalt (in other words it has variety of types and grades, ranging from hard, brittle solids to almost liquids).

b) Liquid asphalt is soft and their consistency cannot be measured with a penetration test at normal temperature.

Road Tars are known as RT and ranging from RT1 to RT12 [5].

RT 1 is used for dust control

RT 2, 3 are light prime coats
RT 4 is a surface treatment
RT 5, 6, 7 is a surface treatment and road mixes
RT 8, 9 is a surface treatment, road mixes and seal coat
RT 10, 11, 12 is a seal coat and tar concrete.

Either asphalt concrete or tar can be used for roadway and runway material construction. These bituminous materials are usually placed in the top and in the upper part of pavement construction. Usually the uppermost layer is surface course and is usually thinner than the lower binder course. See Figure 1. The surface has smaller aggregates than the binder course, because the binder course is a transitional layer between the base course and the surface course. Ideally, flexible pavement should consist of a surface course and a binder course. However, many of them are built of only one layer. Many types of bituminous material are available in the market. These types are selected based on the climate condition and the nature of the construction area.

2. BASE AND SUBBASE MATERIALS

The base course is usually constructed of stone fragments, slag, or granular materials. The base course should have a higher quality of material than the subbase, because the base course is closer to the surface, and therefore will receive more pressure than the subbase. The Subbase can be constructed from local materials which helps reduce the construction cost. Under flexible pavement, these two courses are used to increase the support capacity of the pavement. So most of the shear
and deformation will not take place in the subgrade only, but will help
the subgrade from overstress. There are three types of base and subbase
materials use for pavement: 1. gravel
   2. crushed stone
   3. sand
Laboratory tests and field experience have shown that crushed stone is
more stable than gravel. Crushed as compared to gravel or sand stone is
believed to give a greater resistance to deformation.

3. SUBGRADES
Flexible pavement performance in most cases is affected by the
characteristics of the subgrades. It is necessary to study the condition
of the soils in place because subgrades greatly vary and because the
subgrade consists of highly variegated materials. Subgrades vary from
very soft to stiff material. Because of the complexity of the problem,
proper handling is necessary. For example, proper compaction of the
subgrade is essential to increase soil density, which in turn will help to
support the whole pavement loads.
4. THE DIFFERENCES BETWEEN HIGHWAY AND AIRPORT PAVEMENT

Since this research involves the evaluation of airfield pavement, it is necessary to understand the differences between highway and airfield pavements. The performance of highway and airport pavement in most cases are quite the same, but there are some basic differences between the two:

1. The differences in repetition of the load.
   The total weight of the aircraft is usually much greater than the weight of the vehicle. The number of load repetitions, however, are much higher on highways compared with airport pavement. For this reason, highway pavements are usually built thicker than airfield pavements.

2. Traffic distribution
   Almost all vehicles, which in most cases are trucks, travel within 3 to 4 feet from the edge of the pavement. Conversely traffic in the airport travels mostly in the center of the pavement. Pavement field studies report that a lot of highway pavements have serious cracking near the edge of the pavements, whereas airfield pavements do not. This phenomena is understandable, because on airfields, loads are distributed more equally on both sides.

3. Geometry of the pavement
   The geometry of the pavement is extremely important. From Figure 4, it can be seen clearly that the runway end is designed with greater
thickness because, this part of the pavement areas receives higher load concentration than other parts of the airfield. Meanwhile the pavement thickness in highways is almost the same.

4. Soil distribution

The shorter distance of an airfield makes the choice of its location easier than the choice of the location for highway. Because of this airfields usually have a more uniform subgrade.

Figure 4  Thickened pavements for heavy load concentration

(a) Transverse section of highway pavement
(b) Longitudinal section of runway
1. FIELD TESTING

Two highways and three county airports in Ohio were analyzed for this research. The data of surface deflections were collected by the Ohio Department of Transportation (ODOT) using the Falling Weight Deflectometer. This nondestructive testing method is a very recent one and it is widely accepted in the U.S. The Falling Weight Deflectometer device uses an impulse load applied by a weight package and dampening system (6). See Figure 5 below.

![Figure 5]

Schematic of FWD measurements
The applied loads are selected by the weight that drops onto a circular plate with a radius of 5.9 inches. The pavement deflections are measured at seven locations including the location under the applied load. Six additional sensors are placed at 7.9, 11.8, 23.6, 35.4, 49.0, and 70.9 inches from the center of circular plate.

The surface deflections for highways were taken at approximately every 100 feet along the selected part of the highway, for 40 different locations. Three different applied loads were used to get the surface deflection at each location. The field data deflections were taken layer by layer and the cross sections of the pavements are listed below from the top layer to the infinite subgrade layer.

1. Fairfield I-270 highway
   - 1.25" Asphalt Concrete 404
   - 1.75" Asphalt Concrete 402
   - 8" Bituminous aggregate
   - ∞ Semi infinite subgrade

2. Jackson US-35 highway
   - 1.75" Asphalt Concrete 404
   - 8" Bituminous aggregate
   - 8" Subbase
   - ∞ Semi infinite subgrade

The deflection data for the county airport were measured at five
feet from the center line of the pavement. This distance was chosen because cracking usually occurs around aircraft wheels which travel five feet from the center of pavement line. The criteria for Falling Weight Deflectometer was the same as for highways, only fewer data locations were chosen. The field data deflections were not taken layer by layer; therefore only deflection data from the top of the pavement are available. The cross sections of three county airports are listed below:

1. Pike County Airport

   This airport was built around 1967. Originally the runway started from station 3+50 to 38+50 and it was extended in 1970 from station 38+50 to 45+50. The pavement sections are:

   1.25" Asphalt concrete 404, overlay in 1980
   1.50" Asphalt concrete 402
   6" Prime aggregate base 304
   5" Subbase 310
   ∞ Semi-infinite subgrade

2. Bellefontaine Municipal Airport

   This runway was originally built around 1966 and the pavement cross sections are as follows:

   1.25" Asphalt concrete 404, overlay in 1984
   1" T35C (Asphalt concrete 404 or Asphalt concrete 403)
   4" Waterproof aggregate base
   2" I18 Crushed stone aggregate base
   ∞ Semi-infinite subgrade
3. Seneca County Airport

No airport history was given. The pavement sections are:

1.25" Asphalt concrete 404
1.25" Asphalt concrete 402
3" Bituminous aggregate base
5" Subbase
∞ Semi-infinite subgrade
2. DATA ANALYSIS

The deflection data received from ODOT were analyzed using Statistical Analysis System (SAS). The average of deflection from the same applied load is calculated for each run. This sample average serves as a tool to estimate the actual unknown population mean. The average of deflection data is shown in Table 1 through Table 5 for Fairfield I-270 and Table 6 through Table 9 for Jackson US-35 highway, while Table 10, 11, 12 represent the average of deflection data for Pike, Bellefontaine and Seneca county airport respectively. From the first 9 tables it was noticed that more data were taken for subgrade layer compared with other layers. Refer to Table 1 and Table 6. This is certainly due to the fact that soil consists of non-uniform material and varies greatly from place to place. So, by taking more data, the general type of soil in that area can be presented. Table 10, 11, 12 have a lot less data than data presented in other tables. A lot of data taken is not needed because of:

- The subgrade in airfields are usually more uniform than highway subgrades.
- Airfield pavements only cover short distances compared with highway pavements.

The next step is to find a better model to fit all the average deflection data. Using the Cricket Packet program, equation of the line, which represent the average data can be predicted. This Cricket Packet
program uses the principle of the least squares. Principle of the least squares involves fitting a line through the points so that the sum of the vertical differences between all the points and the lines is minimized. Three models are used to compare which equation fits the best to the data. These equations are:

1. \( Y = b_0 + b_1X \) Straight line equation
2. \( Y = b_0 + b_1X + b_2X^2 \) Quadratic equation
3. \( Y = b_0 + b_1X + b_2X^2 + b_3X^3 \) Cubic equation

\[ \text{note: } b_0, b_1, b_2 = \text{variable} \]
\[ Y = \text{deflection} \]
\[ X = \text{applied load} \]

For each model, Cricket Packet program gives the value of R-squared. R-squared is the fraction of the total variation caused by the variable in the model. R-squared always ranges from 0 to 1. The closer R-squared is to 1 represents the better model for certain variation in the data. By comparing R-squared from three different equations, the equation of the line with R-squared closest to 1 is chosen to represent that particular data set.

The equations of the lines corresponding to Table 1 through Table 12 are given after the presentation of each table. It is not necessary to find the equation for airport deflection data since there are only three different data sets given and the results are linear. With more data, however, there may be a variation. The results of the equations are
related to as follows:

1. Table 1 for the Fairfield subgrade and Table 6 for Jackson subgrade show that the subgrades have a non-linear response.

2. Table 7 for Jackson's second layer (subgrade + subbase) show that all of the deflections still have non-linear equations. Here, there is a bad data correlation ($R^2 = 0.478$, for D7) probably because of the geophone malfunction in sensor number 7.

3. Table 2 and 3 for the Fairfield case (subgrade + bituminous aggregate) show that the first three deflections give strong linear equation with $R^2$ ranging from 0.998 to 0.999, while deflection number 6 and 7 show non-linear relationship.

4. In Table 8 sensors 1 and 2 show a linear relationship of $R^2 = 0.999$ for the Jackson third layer (subgrade + subbase + bituminous).

5. Table 4 and 5 for the Fairfield pavement (subgrade + bituminous + asphalt) give deflection number 1 a quadratic equation of $R^2 = 0.999$ and $R^2 = 0.992$ respectively.

6. The Table 9 for the Jackson pavement (subgrade + subbase + bituminous + asphalt), the first 4 deflections have a linear equation of $R^2 = 1.000$. Deflection number 6 and 7 show cubic equation.
Fairfield I-270 subgrade

Data taken: June 8, 1988

Number of data: 640

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</table>

Table 1

Subgrade deflection data for Fairfield I-270

Equations of the best line for Table 1 are: (P in N/m² and D in 10⁻⁶ m)

D1 = -126.920 + 2.883P - 1.482E⁻³P² + 3.269E⁻⁷P³ ; R² = 0.985

D2 = -116.310 + 1.792P - 1.057E⁻³P² + 2.883E⁻⁷P³ ; R² = 0.989
D3 = -62.432 + 0.927P - 3.893E-04P^2 + 7.186E-08P^3 ; R^2 = 0.987
D4 = -14.750 + 0.247P - 1.378E-04P^2 + 6.774E-08P^3 ; R^2 = 0.986
D5 = 4.596 + 3.946E-02P + 3.876E-05P^2 - 2.148E-08P^3 ; R^2 = 0.991
D6 = 0.113 + 5.565E-02P - 5.422E-05P^2 + 3.089E-08P^3 ; R^2 = 0.985
D7 = 1.728 + 2.309E-02P - 6.260E-06P^2 + 2.529E-09P^3 ; R^2 = 0.970

Fairfield I-270 Subgrade + 4" Bituminous aggregate

Data taken : June 15, 1988
Air temperature : 63° F
Number of data : 480

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Table 2
Second layer deflection data for Fairfield I-270
Equations of the best line for Table 2 are: ( P in N/M² and D in 10E-06 m )

\[ D1 = -24.208 + 0.744P \quad ; \quad R^2 = 0.999 \]
\[ D2 = -16.861 + 0.540P \quad ; \quad R^2 = 0.999 \]
\[ D3 = -12.445 + 0.412P \quad ; \quad R^2 = 0.998 \]
\[ D4 = -7.667 + 0.193P - 1.616E-05P^2 \quad ; \quad R^2 = 0.994 \]
\[ D5 = -2.665 + 8.836E-02P - 1.281E-05P^2 \quad ; \quad R^2 = 0.991 \]
\[ D6 = -0.517 + 5.269E-02P - 3.443E-05P^2 + 1.101E-08P^3 \quad ; \quad R^2 = 0.975 \]
\[ D7 = 1.2785 + 2.205E-02P - 3.469E-05P^2 + 2.044E-08P^3 \quad ; \quad R^2 = 0.601 \]

**Fairfield I-270 Subgrade + 4" Bituminous aggregate +
4" Bituminous aggregate**

Data taken: June 23, 1988

Air temperature: 70° F

Number of data: 480
### Table 3
Third layer deflection data for Fairfield I-270

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<th>D3 x 10E-06 m</th>
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<th>D5 x 10E-06 m</th>
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Equations of the best line for Table 3 are: (P in N/m² and D in 10E-06 m)

- \( D_1 = -25.692 + 0.391P \); \( R^2 = 0.999 \)
- \( D_2 = -16.571 + 0.279P \); \( R^2 = 0.999 \)
- \( D_3 = -13.246 + 0.224P \); \( R^2 = 0.999 \)
- \( D_4 = -3.766 + 0.116P + 5.620E-06P^2 \); \( R^2 = 0.999 \)
- \( D_5 = -1.544 + 6.990E-02P + 7.453E-06P^2 \); \( R^2 = 0.997 \)
- \( D_6 = 0.828 + 3.823E-02P - 3.612E-06P^2 \); \( R^2 = 0.992 \)
- \( D_7 = 2.024 + 1.392E-02P - 5.466E-06P^2 + 3.324E-09P^3 \); \( R^2 = 0.979 \)
Fairfield I-270 Subgrade + 8" Bituminious aggregate + 1.75" Asphalt concrete

Data taken: August 10, 1988
Air temperature: 75° F
Number of data: 480

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<th>D3 x 10E-06 m</th>
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Table 4
Fourth layer deflection data for Fairfield I-270

Equations of the best line for Table 4 are: (P in N/M² and D in 10E-06 m)

D1 = -23.341 + 0.324P + 1.866E-05P² ; R² = 0.999
D2 = -6.405 + 0.171P + 1.188E-04P² - 4.940E-08P³ ; R² = 1.000
D3 = -4.522 + 0.139P + 1.028E-04P² - 4.169E-08P³ ; R² = 1.000
D4 = -1.770 + 8.392E-02P + 6.317E-05P² - 2.658E-08P³ ; R² = 0.999
D5 = -0.595 + 5.294E-02P + 3.647E-05P² - 1.710E-08P³ ; R² = 0.997
D6 = -0.632 + 3.790E-02P + 7.259E-07P² - 3.398E-09P³ ; R² = 0.986
D7 = 1.104 + 1.428E-02P + 2.845E-06P² -3.225E-09P³ ; R² = 0.949

Fairfield I-270 Subgrade + 8" Bituminous aggregate +
1.75" Asphalt concrete +
1.25" Asphalt concrete

Data taken: August 15, 1988
Air temperature: 77° F
Number of data: 480

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Table 5
Fifth layer deflection data for Fairfield I-270
Equations of the best line for Table 5 are: (P in N/M² and D in 10E-06 m)

D1 = -7.536 + 0.244P + 5.332E-05P² ; R² = 0.992
D2 = -4.945 + 0.183P + 2.782E-05P² ; R² = 0.998
D3 = 0.261 + 0.123P + 7.859E-05P² - 2.832E-08P³ ; R² = 1.000
D4 = 0.309 + 8.282E-02P + 4.716E-05P² - 2.154E-08P³ ; R² = 0.997
D5 = 2.797 + 4.430E-02P + 4.203E-05P² - 2.130E-08P³ ; R² = 0.986
D6 = 0.754 + 3.999E-02P - 5.456E-06P² - 1.348E-09P³ ; R² = 0.957
D7 = 2.307 + 1.179E-02P + 5.942E-06P² - 4.395E-09P³ ; R² = 0.939

Jackson US-35 Subgrade

Data taken: May 17, 1988

Number of data: 800
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Table 6
Subgrade deflection data for Jackson US-35

Equations of the best line for Table 6 are: (P in N/M² and D in 10E-06 m)

D1 = 0.761 + 2.093P - 5.786E-04P²; R² = 0.982
D2 = 10.307 + 0.663P + 6.244E-04P² - 4.152E-07P³; R² = 0.997
D3 = 7.356 + 0.314P + 4.692E-04P² - 2.401E-07P³; R² = 0.997
D4 = 0.373 + 0.150P + 7.067E-05P² - 3.702E-08P³; R² = 0.997
D5 = 0.717 + 9.225E-02P - 1.152E-05P²; R² = 0.996
D6 = 1.608 + 6.913E-02P - 5.837E-05P^2 + 2.610E-08P^3 \ ; \ R^2 = 0.972
D7 = 1.677 + 4.679E-02P - 4.674E-05P^2 + 2.051E-08P^3 \ ; \ R^2 = 0.913

Jackson US-35 Subgrade + 8" Subbase

Data taken : June 20, 1988
Air temperature : 72° F
Number of data : 480

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<th>Load Applied (Pa)</th>
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Table 7
Second layer deflection data for Jackson US-35

Equations of the best line for Table 7 are : ( P in N/M² and D in 10E-06 m )

D1 = 16.347 + 0.983P - 5.928E-05P^2 \ ; \ R^2 = 0.998
D2 = 15.759 + 0.415P + 2.459E-04P^2 - 1.420E-07P^3 \ ; \ R^2 = 0.998
D3 = 5.488 + 0.238P + 2.120E-04P^2 - 1.273E-07P^3 \ ; \ R^2 = 0.999
D4 = -2.965 + 0.140P - 1.777E-05P^2 \quad ; R^2 = 0.999

D5 = 2.222 + 5.588E-02P + 3.825E-05P^2 - 2.408E-08P^3 \quad ; R^2 = 0.993

D6 = 8.893 + 1.195E-02P + 4.882E-05P^2 - 2.425E-08P^3 \quad ; R^2 = 0.967

D7 = -4.817 + 7.243E-02P - 2.463E-05P^2 - 7.530E-09P^3 \quad ; R^2 = 0.478

**Jackson US-35 Subgrade + 8" Subbase + 8" Bituminious aggregate**

Data taken: July 5, 1988

Air temperature: 72.5° F

Number of data: 480

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<th>D3 x 10E-06 m</th>
<th>D4 x 10E-06 m</th>
<th>D5 x 10E-06 m</th>
<th>D6 x 10E-06 m</th>
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Table 8

Third layer deflection data for Jackson US-35
Equations of the best line for Table 8: (P in N/M² and D in 10E-06 m)

\[ D_1 = -13.527 + 0.371P \quad ; \quad R^2 = 0.999 \]
\[ D_2 = -10.685 + 0.289P \quad ; \quad R^2 = 0.999 \]
\[ D_3 = -6.351 + 0.217P + 3.881E-05P^2 - 1.603E-08P^3 \quad ; \quad R^2 = 1.000 \]
\[ D_4 = -5.816 + 0.139P \quad ; \quad R^2 = 0.999 \]
\[ D_5 = -1.663 + 7.717E-02P + 7.168E-06P^2 - 4.536E-09P^3 \quad ; \quad R^2 = 0.998 \]
\[ D_6 = 1.673E-02 + 3.711E-02P - 3.963E-06P^2 \quad ; \quad R^2 = 0.995 \]
\[ D_7 = -1.361 + 2.013E-02P - 1.462E-06P^2 \quad ; \quad R^2 = 0.996 \]

**Jackson US-35 Subgrade + 8" Subbase + 8" Bituminous aggregate + 1.75" Asphalt concrete**

Data taken: September 19, 1988

Air temperature: 87.8° F

Number of data: 480
### Table 9

Fourth layer deflection data for Jackson US-35

Equations of the best line for Table 9 are: ( P in N/M² and D in 10E-06 m )

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<th>D 1 x 10E-06 m</th>
<th>D 2 x 10E-06 m</th>
<th>D 3 x 10E-06 m</th>
<th>D 4 x 10E-06 m</th>
<th>D 5 x 10E-06 m</th>
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\[
\begin{align*}
D_1 &= -17.353 + 0.369P ; \quad R^2 = 1.000 \\
D_2 &= -12.183 + 0.264P ; \quad R^2 = 1.000 \\
D_3 &= -10.808 + 0.223P ; \quad R^2 = 1.000 \\
D_4 &= -5.4950 + 0.132P ; \quad R^2 = 1.000 \\
D_5 &= -1.686 + 7.804E-02P ; \quad R^2 = 0.999 \\
D_6 &= 0.230 + 3.645E-02P - 3.433E-06P^2 ; \quad R^2 = 0.999 \\
D_7 &= 0.585 + 1.303E-02P + 7.966E-06P^2 - 3.736E-09P^3 ; \quad R^2 = 0.996 
\end{align*}
\]
Pike county airport, direction 7

Data taken: October 30, 1987
Air temperature: 75°F
Pavement temperature: 89°F
Number of data: 66

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<th>Load Applied (Psi)</th>
<th>D 1 x 10E-03 in</th>
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<th>D 3 x 10E-03 in</th>
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<th>D 6 x 10E-03 in</th>
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Table 10
Deflection data for Pike airport

Bellefontaine airport, direction 22

Data taken: September 17, 1987
Air temperature: 65°F
Pavement temperature: 85°F
Number of data: 66
Table 11
Deflection data for Bellefontaine airport

**Seneca airport, direction 6**

Data taken: June 24, 1987

Air temperature: 45°F

Pavement temperature: 63°F

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Table 12
Deflection data for Seneca airport
Illipave is the current version initially developed by E.L.Wilson. It has since been modified by Barlow, Duncan, Monismith and Wilson; who comprise the research staff of the Construction Engineering Laboratory at Champaign, Illinois. From the journal of "Illipave mechanistic analysis of AASHO road test flexible pavements", Figueroa, Hoffman and Thompson concluded that Illipave provided reliable and realistic prediction of the structural behavior of pavement. Therefore, from many mechanistic computer programs, Illipave was selected and used in this research to evaluate 2 highways and 3 county airports in Ohio.

Illipave is based on the finite element method, which is a powerful method used to solve many engineering problems. In the finite element method, the domain of the problem is divided into smaller regions without overlapping, allowing no gaps between the elements. This is called mesh generation. The smaller the region, the more precise the solution will be. After the mesh generation has been created, element equations can be written for each element of the same type. The element equation's derivation is a theoretical procedure performed by the program developer. These element equations are combined into a much larger equation called the system equation. This system equation is the answer to the entire system, which usually comprises a very large number of equations and can be solved using conventional numerical analysis techniques.
Three activities are involved in the finite element. See Figure 6 below

- Define a specific problem
  - Geometry
  - Physical properties
  - Loads

- Input data to program
  - Mesh generation
  - Physical properties
  - Load - interior & boundary
  - Type of output desire

- Finite element program

- Output data
  - Tables of values
  - Graphical x, y, z plots
  - Contour plots

Figure 6
Steps involved in F.E programming
The use of the finite element model to solve engineering problems offer certain advantages:

1. There are no ambiguities concerning the boundary condition.
2. Statistical checks on the results have physical meaning and can be made more adequately.
3. Variations in dimension and physical properties can be easily treated.

The finite element solution has been applied to solve many pavement problems in recent years. In the case of Illipave, the pavement is modeled as an axisymmetric three dimensional cylindrical configuration. Using a system of symmetry, this three dimensional pavement section (shown in Figure 7a) is transformed into a rectangular configuration (as shown in Figure 7b). See example in Figure 8.

![Figure 7 a, b](image)

F.E. idealization of cylinder
Mesh diagram of Bellefontaine cross sections pavement

Axis of symmetry

Z-axis

r = 5.9 in

300
297.75
293.75
291.75
285
275

Subgrade

All others move freely, vertically & radially

Waterproof aggregate

Crushed Stone Base aggregate

Move only in a vertical direction, not radially

All nodal points on the lower boundary are vertically and radially fixed

Figure 8
This rectangular configuration represents the mesh diagram for the Bellefontaine airport pavement subsystem. Each of the elements and nodes shown in Figure 8 are numbered according to elevation boundaries, from top to bottom. Radial boundaries are numbered from left to right. Elevation and radial boundaries can be adjusted and created to suit the pavement cross section and the location of Falling Weight Deflectometer's sensors respectively. All nodal points on the lower boundary are vertically and radially fixed. Meanwhile, all nodes along the symmetry axis and cylindrical boundary are free to move only vertically, but not radially. Besides these two restrictions, all nodes along the surface and inside the mesh diagram are free to move vertically and radially. A uniform pressure was applied over a 5.9 in a radius circular area to induce traffic loading conditions.

Due to the fact that Illipave computer program requires a lot of time and is too costly, Duncan, Monismith and Wilson have developed 4 criteria of "How to generate the mesh diagram to get better accuracy without requiring a lot of computer time". Their criteria are:

1. The element stresses will be sufficiently accurate so long as the length (vertical) to width (horizontal) ratio of the elements do not exceed 5 to 1.

2. Smaller elements near the load will increase accuracy where the influences of the applied load are more significant.

3. The rigid lower boundary should be placed at least at an approximate depth of 50 times the radius of the applied load. Since $r = 5.9$ in, so $Z$ at least 295 inches.
4. The outer side boundary should be specified at a minimum distance of 12 times of the radius of the applied load. In this case \( R \) should at least 70.8 inches.

So Figure 8 sufficiently meets these 4 criteria.

Besides mesh generation, physical properties are needed for input data to the program. Illipave has 4 different material models based on Mohr-Coulomb failure criteria, there are:

1. Materials with modulus as a function of minor principal stress, \( \sigma_3 \) indicated as 1.0.
2. Cohesive material with modulus as a function of stress difference, indicated as 2.0
3. Materials with constant modulus, indicated as 3.0
4. Materials with modulus as a function of the first stress invariant, \( \Theta \), indicated as 5.0.

These material models must be specified in each layer of the pavement subsystem. Further discussion about material models are available in Illipave Users Manual [7]. Besides these material models, density, poisson's ratio and the coefficient of earth pressure at rest are needed for each pavement layer. The summary of material properties is shown in Table 13.
<table>
<thead>
<tr>
<th></th>
<th>40°F</th>
<th>70°F</th>
<th>100°F</th>
<th>40°F</th>
<th>70°F</th>
<th>100°F</th>
<th>Stiff</th>
<th>Medium</th>
<th>Soft</th>
<th>V. Soft</th>
</tr>
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<tbody>
<tr>
<td><strong>Asphalt Concrete</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Unit Weight (psf)</td>
<td>145.00</td>
<td>145.00</td>
<td>145.00</td>
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<td>135.00</td>
<td>125.00</td>
<td>120.00</td>
<td>115.00</td>
<td>110.00</td>
<td></td>
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<td>Lateral Pressure</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coeff. at Rest.</td>
<td>0.37</td>
<td>0.67</td>
<td>0.85</td>
<td>0.60</td>
<td>0.60</td>
<td>0.82</td>
<td>0.82</td>
<td>0.82</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.27</td>
<td>0.40</td>
<td>0.46</td>
<td>0.38</td>
<td>0.38</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td>Unconf. Compress.</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>32.80</td>
<td>22.85</td>
<td>12.90</td>
<td>6.21</td>
<td></td>
</tr>
<tr>
<td>Deviator Stress</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Upper Limit (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>32.80</td>
<td>22.85</td>
<td>12.90</td>
<td>6.21</td>
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<tr>
<td>Deviator Stress</td>
<td></td>
<td></td>
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<tr>
<td>Lower Limit (psi)</td>
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<td></td>
<td></td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Codi (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.20</td>
<td>6.20</td>
<td>6.20</td>
<td>6.20</td>
<td></td>
</tr>
<tr>
<td>Epl (ksi)</td>
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<td></td>
<td></td>
<td></td>
<td>12.34</td>
<td>7.68</td>
<td>3.02</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>E-Failure (ksi)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>4.00</td>
<td>4.00</td>
<td>7.605</td>
<td>4.716</td>
<td></td>
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<td>500.00</td>
<td>100.00</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Er-Model (psi)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fric. Angle (deg)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Closion (psi)</td>
<td></td>
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</tr>
</tbody>
</table>
As was discussed in chapter 2, flexible pavement is composed of asphalt concrete, base / subbase and subgrade soil. Illipave treats these pavement materials as follows:

1. **Asphalt Concrete**

   Asphalt concrete is treated as linear elastic material in the Illipave model. According to Dehlen, G.L. [7] "The effect of Non-linear Material Response on the Behavior of Pavements Subjected to Traffic Load;" Ph.D. thesis, University of California, Berkeley, 1969, asphaltic concrete displays some degree of non-linear behavior. It is very common, however, to model the asphaltic concrete layer as a linear elastic material. From Table 13, the asphaltic constant modulus varies from 100 to 1400 ksi depending to the temperature. Figure 9 shows the comparison of Asphalt concrete modulus - temperature relationship according to Illipave model against Elliot, R.P and Thompson, M.R mechanistic design concept. These 2 curves do not show a lot of significant differences. Therefore, either one of these curves can be used as a reference for asphaltic concrete modulus.
Figure 9
Asphalt concrete modulus vs temperature relationship

2. Granular base / subbase

There are 2 models available in Table 13, crushed stone and gravel
materials. The basic equation to model the behavior of granular base material is as follows:

\[ Er = K \theta^n \]

- \( Er \) = resilient modulus of the material (psi)
- \( K, n \) = constant evaluated from repeated-load triaxial laboratories test result
- \( \theta \) = the sum of the principal test (psi)

\( \theta = \sigma_1 + 2\sigma_2 \)

For 2 models above, the equations are in these form

- \( Er = 9000 \theta^{0.33} \)  crushed stone material
- \( Er = 6500 \theta^{0.30} \)  gravel material

The friction angle used in Illipave is 40°. Any material other than crushed stone and gravel, the \( K \) and \( n \) values can be identified from Figure 10. This figure, by Rada and Witczak, shows the relationship between \( K \) and \( n \) values for various granular materials.

![Figure 10](image)
3. **Subgrade soil**

It can be seen from Table 13 that there are 4 models available for subgrade soil.

a) Stiff
b) Medium
c) Soft
d) Very soft

The behavior of subgrade soil is described according to Figure 11 below.

Mathematically, this figure can be expressed in these equations:

\[
E_r = E_{ri} + K_1 (\sigma_d - \sigma_{di}) \quad \text{for } \sigma_d < \sigma_{di}
\]

\[
E_r = E_{ri} + K_2 (\sigma_d - \sigma_{di}) \quad \text{for } \sigma_d > \sigma_{di}
\]

where \(E_r\) = resilient modulus
Eri = resilient modulus at the turning point
Ef = modulus of failure
EGi = deviator stress at the turning point
    this deviator stress is fixed for all subgrade soil models
    for the Illipave model = 6.2 psi
    other research suggest the value of 6.0 psi
K1, K2 = slopes

**Example solution**

Now, consider the Bellefontaine airport as an example of back calculated material properties of the pavement.

1. Study the pavement cross section and temperature. The pavement cross section and temperature are given in chapter III.

2. We have to make a layer model based on the summary of material properties in Table 13
   - Layer I : 2.25" Asphalt concrete
   - Layer II : 4" Asphalt concrete as aggregate base
   - Layer III : 2" crushed stone as granular base
   - Layer IV : ∞ Semi-infinite subgrade

3. Make a mesh diagram
   - The mesh diagram can be seen earlier in Figure 8

4. Make an input program
   - The input program can be seen as follows:
TITLE 'BELLE 22'
INCR 1 HPAV 6.25 HSUB 291.75
RAD 5.95 PSI 84.21
PROP 1 3.0 DEN 1 145. KO 1 0.72 H1 2.25 E 1 4.000E05 U1 0.41
PROP 2 3.0 DEN 2 145. KO 2 0.76 H2 4.00 E 2 3.000E05 U2 0.43
PROP 3 5. DEN 3 135. KO 3 .6 H3 002. U3 .38 KONE3 9000
X3 0.33 MAXSR 3 4.8 MINSIG 3 0.01 EFAIL 3 4000
PROP 4 2. DEN4 125. KO4 0.82 H4 291.75 U4 0.45 KONE4 6.20
TAUSUB 4 18.00 EFAIL4 4865. KTWO4 9600. KTHREE4 1000. KFOUR4 -178
DSLL4 2. DSUL4 32.80
NCOL19 NROW18
R1 0.0 R6 5.90 R7 7.90 R9 11.80 R13 23.60 R15 35.40 R17 49.0 R19 70.9
R20 100.0
PRESS DISTALL 6 1 84.21
Z1 300.0 ZMAT1 1
Z3 297.75 ZMAT3 2
Z6 293.75 ZMAT6 3
Z7 291.75 ZMAT7 4
Z9 285.00 ZMAT9 4
Z11 275.0 ZMAT11 4
Z13 255.0 ZMAT13 4
Z15 215.0 ZMAT15 4
Z16 185.0 ZMAT16 4
Z17 145.0 ZMAT17 4
Z18 95.0 ZMAT18 4
Z19 0.0 ZMAT19 4
Prop1 through prop 4 represent pavement layer number 1 through layer number 4 respectively. Density, coefficient at pressure, layer thickness and poisson's ratio have to be assigned for each layer. Prop1 and prop 2 have a modulus function indicator of 3.0, which means that the material for layer number 1 and number 2 have a constant modulus. Asphaltic concrete is one of these materials. Prop 3 uses modulus function indicator of 5. to represent base material. KONE and X are normally determined from repeated triaxial testing. MAXSR is the maximum allowable stress ratio. Use MINSIG = 0.01 if limit not known. EFAIL is modulus after failure. Prop 4 uses modulus function indicator of 2.0 to represent subgrade (see Figure 11). KTHREE = K1, KFOUR = K2, KTWø = Eri, EFAIL = Ef and TAUSUB are assigned equal to cohesion.

R1 - R20 represent the nodal column number in R-axis with respect to the axis symmetry.

Z1 - Z19 represent nodal row number starting from the pavement surface. Each Z is followed by ZMAT. This ZMAT indicates the kind of material below the boundary number.

The above example is the final solution for Bellefontaine airport. Trial and error are needed to backcalculate material properties. There are three
iterations of each Illipave output. Iteration number 2 is usually accurate enough to match field deflection for backcalculated material purposes. However great care must be taken if there is any unpredicted result.
V. EVALUATION OF THE PAVEMENT RESULTS

In recent years, mechanistic models have been used to predict pavement performance. Unfortunately, strain and stress from each of pavement layers are not easily measured in the field. The easiest way to predict pavement performance and backcalculate material properties is to use surface deflections. The method used in this evaluation is based upon Finite element Illipave program, by comparing Illipave deflections with field deflections.

Six applied loads are used for evaluation, ranging from 17 psi to 195 psi. The FWD (field) deflections correspond to these loads are obtained from the equations line on chapter III. The average temperature of 5 days prior to the data taken, has been calculated as a control for modulus of asphaltic concrete. Using Table 13 as an input in the Illipave program, pavement material properties can be back-calculated by matching the FWD deflection with Illipave deflection. The pavements' solutions are presented in the following:

Figures 12a - 12d show the vertical deflection of FWD vs Illipave for the Fairfield I-270 subgrade.

Figures 13a - 13c through Figures 16a - 16c show the vertical deflection of FWD vs Illipave for the Fairfield I-270 second through fifth layer pavement respectively.

Figures 17a - 17e show the vertical deflection of FWD vs Illipave for the Jackson US-35 subgrade layer.

Figures 18a - 18d through figures 20a - 20c show the vertical
deflection of FWD vs Illipave for the Jackson US-35 second through fourth layer pavement respectively.

Figures 21a - 21c, 23a - 23c, 25a - 25c show the vertical deflection of FWD vs Illipave for the Pike, Bellefontaine and Seneca airports respectively.

After Figure 16, 20, 21, 23, 25, a summary of backcalculated materials and parameter tables are listed. Figure 22, 24, 26 shows the effect of various applied loads on the vertical deflection. Illipave give a straight line results for all loads shown.

The FWD data available for layer by layer pavement easily has an advantage in back-calculating the value of layer modulus. The layer by layer results for highway are:

1. Fairfield I-270
   - The subgrade layer presented in Figures 12a - 12d are very sensitive to further sensors, especially sensor numbers 5, 6, 7. Sensor number 1 is more sensitive for a lower load up to 50 psi, see Figures 12a and 12b.
   - The second layer which consists of subgrade and 4" bituminous aggregate, gives very good matching deflections for all sensors. Especially for loads between 50 psi to 123.23 psi. See Figures 13a - 13d for clarification.
   - Figures 14a -14c are the results of the third layer (subgrade + 8" bituminous) pavement evaluation. FWD for sensor number 1 and 7 give the best matching deflections as shown in Figures 14b and 14c. For lower applied loads (see Figure 14a), Illipave deflection number 1 does
not match FWD deflection.

- As the layer increases to the fourth layer, the second and the seventh sensors give the best matching deflection. Refer to Figures 15a and 15b. Again at lower loads (as shown in Figure 15a), only sensor number 7 has become sensitive.

- The result of pavement performance from the top layer can be seen in Figures 16a - 16c. The second sensor gives the best matching deflection for loads bigger than 50.0 psi.


- The subgrade layer for Jackson shown in Figures 17a - 17e give the same results as the Fairfield subgrade. The deflections match for further sensors.

- The performance of the second layer (subgrade + subbase) are given in Figures 18a - 18d. Illipave gives the best fit deflections for almost all sensors at higher applied loads (see Figures 18c and 18d).

- The results of the third layers (subgrade + subbase + bituminous) are given in Figures 19a - 19c. The applied loads between 50 psi to 123.23 psi give the best matching deflection. Sensor number 2 begins to be sensitive at higher loads.

- The last layer (Figures 20a - 20c), gives the best deflection matching for sensor number 2 in the range of P = 50 psi to P = 82.3 psi, while sensor number 4 is sensitive to loads greater than 123.23 psi.

The airport pavements performance are given below:

1. Pike airport

The results of pavements performance are shown in Figures 21a - 21c.
Figure 21a and 21b show the best matching deflections for all sensors, whereas the deflection shown in Figure 21c is sensitive to sensors number 4 and 7.

2. Bellefontaine airport

Figures 23b and 23c give better matching deflections for P = 84.21 psi and P = 116.14 psi. Applied loads at 50 psi shown in Figure 23a give nice deflection matching for sensor number 7.

3. Seneca airport

See Figures 25a-25c for the results of pavement performance. All sensors give good matching deflections, especially for sensor number 1 and 5. This case applied to all 3 loads.

To get better information about the pavement evaluation, the following parameters are needed:

1. Area = 6 \( \left( 1 + 2D3/D1 + 2D4/D1 + D5/D1 \right) \) in 'inches'
2. \( F1 = \frac{D1 - D4}{D3} \) shape factor 1
3. \( F2 = \frac{D3 - D5}{D4} \) shape factor 2
4. Spread(%) Vaswani = \( \left( \frac{D1 + D3 + D4 + D5}{4D1} \right) \times 100 \)
5. Spread(%) Kuo = \( \frac{D1 + D2 + \ldots + Dn}{n \times D1} \times 100 \)

Note: D1 = deflection at the center of the load

\( D2 = \text{deflection at 7.9" from the center of the load} \)

\( D3, \ldots, D7 = \text{deflection at 1', 2', 5' from the center load.} \)

For most pavements, the FWD 'area' will be between the value of 11.1 to 36 inches [7]. The shape factors 1 and 2 usually define the profile of the deflection basin. Spreadability by Vaswani is almost the same as 'area'.
listed on number 1 in the parameter. Shiou-San, Kuo spreadability on the other hand, uses all 7 sensors deflection. He claimed that the spreadability value represents the strength of the pavement. Lower spreadability indicates that the pavement is weak. For satisfactory pavement the spreadability value should range between 65% to 80%

From Table 14, the differences between FWD and Illipave spreadability for Fairfield pavement performance are around 9.29%, according to Vaswani, and 9.67%, according to Kuo. The shape factors F1, F2 and area do not give a very precise answer. For the Jackson case (see Table 15), the 2 spreadabilities (6.9% and 6.82%) and F2 give better matching values. Results given in Tables 15, 16, 17 give best matching values in spreadability, shapes factor and area. All percentage differences in spreadability are below 5%.
Vertical deflection vs Distance for the Fairfield I-270 subgrade (E = 16.6 ksi)

---

**Figure 12a**

---

**Figure 12b**
Figure 12c

Figure 12d
Vertical Deflection vs Distance for
the Fairfield second layer ( $E = 700$ ksi )

Figure 13a

Figure 13b
\[ P = 195.0 \text{ psi} \]
\[ P = 123.23 \text{ psi} \]

**figure 13c**
Vertical Deflection vs Distance for the Fairfield I-270 third layer (E = 900 ksi)

Figure 14a

Figure 14b
Figure 14c
Vertical Deflection vs Distance for
the Fairfield I-270 fourth layer (E = 1000 ksi)

Figure 15a

Figure 15b
Figure 15c
Vertical Deflection vs Distance for
the Fairfield I-270 fifth layer (E = 1000 ksi)

Figure 16a

Figure 16b
Summary of back-calculated material properties and the parameter table for the Fairfield I-270 are listed below:

1.25" AC 404  \( E = 1000 \text{ ksi} \)
1.75" AC 402  \( E = 1000 \text{ ksi} \)
4" Bituminous  \( E = 900 \text{ ksi} \)
4" Bituminous  \( E = 700 \text{ ksi} \)
\( \infty \) Subgrade  \( E_{ri} = 16.6 \text{ ksi} \)
<table>
<thead>
<tr>
<th>Applied Load (psi)</th>
<th>Area ( in )</th>
<th>F 1</th>
<th>F 2</th>
<th>Vaswani spr (%)</th>
<th>Kuo spr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FWD Illipave</td>
<td>50.0</td>
<td>19.85</td>
<td>0.97</td>
<td>0.86</td>
<td>57.21</td>
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<td></td>
<td></td>
<td>23.78</td>
<td>0.61</td>
<td>0.71</td>
<td>66.69</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>82.30</td>
<td>19.57</td>
<td>0.99</td>
<td>0.90</td>
<td>56.44</td>
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<td></td>
<td></td>
<td>23.87</td>
<td>0.60</td>
<td>0.71</td>
<td>66.90</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>123.23</td>
<td>19.34</td>
<td>1.02</td>
<td>0.95</td>
<td>55.82</td>
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<td></td>
<td></td>
<td>23.81</td>
<td>0.61</td>
<td>0.71</td>
<td>66.75</td>
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<td>FWD Illipave</td>
<td>Average</td>
<td>19.58</td>
<td>0.99</td>
<td>0.90</td>
<td>57.49</td>
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<td></td>
<td></td>
<td>23.82</td>
<td>0.61</td>
<td>0.71</td>
<td>66.78</td>
</tr>
</tbody>
</table>

Table 14

Pavement parameter table for the Fairfield I-270
Vertical deflection vs Distance for the Jackson US-35 subgrade (E = 14.0 ksi)

Figure 17a

Figure 17b
Figure 17c

Figure 17d
Figure 17e

- Field deflection
- Illipave deflection

$P = 186.0 \text{ psi}$
Vertical deflection vs Distance for
the Jackson US-35 second layer ( $E = 34$ ksi, $E_i = 22$ ksi )

![Graph](image)

**Figure 18a**

![Graph](image)

**Figure 18b**
Figure 18c

Figure 18d
Vertical deflection vs Distance for the Jackson US-35 third layer (E = 750 ksi)

Figure 19a

Figure 19b
Figure 19c
Vertical deflection vs Distance for the Jackson US-35 fourth layer (E = 750 ksi)

Figure 20a

Figure 20b
Summary of back-calculated material and the parameter table for the Jackson US-35 are listed below:

<table>
<thead>
<tr>
<th>Material</th>
<th>E (ksi)</th>
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<tbody>
<tr>
<td>1.75&quot; AC 402</td>
<td>750</td>
</tr>
<tr>
<td>8&quot; Bituminous</td>
<td>650</td>
</tr>
<tr>
<td>8&quot; Subbase</td>
<td>34</td>
</tr>
<tr>
<td>Subgrade</td>
<td>26</td>
</tr>
</tbody>
</table>
### Table 15
Pavement parameter for the Jackson US-35

<table>
<thead>
<tr>
<th>FWD Load (psi)</th>
<th>Area (in)</th>
<th>F1</th>
<th>F2</th>
<th>Vaswani spr (%)</th>
<th>Kuo spr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Illipave</td>
<td>50.0</td>
<td>18.92</td>
<td>1.06</td>
<td>1.01</td>
<td>54.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.77</td>
<td>0.76</td>
<td>0.93</td>
<td>61.39</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>82.23</td>
<td>18.90</td>
<td>1.06</td>
<td>1.05</td>
<td>54.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.07</td>
<td>0.73</td>
<td>0.93</td>
<td>62.05</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>123.23</td>
<td>18.87</td>
<td>1.06</td>
<td>1.06</td>
<td>54.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.72</td>
<td>0.76</td>
<td>0.92</td>
<td>61.27</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>Average</td>
<td>18.90</td>
<td>1.06</td>
<td>1.04</td>
<td>54.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.85</td>
<td>0.75</td>
<td>0.93</td>
<td>61.57</td>
</tr>
</tbody>
</table>
Vertical deflection vs Distance for the Pike County Airport

Figure 21a

Figure 21b
Summary of back-calculated material and the parameter table for the Pike County Airport are listed below:

- **8.75" AC 404**: $E = 1000$ ksi
- **5" Subbase**: $E = 34$ ksi
- **∞ Subgrade**: $E_i = 7.68$ ksi

![Figure 21c](image-url)
<table>
<thead>
<tr>
<th>Applied Load (psi)</th>
<th>Area (in)</th>
<th>F 1</th>
<th>F 2</th>
<th>Vaswani spr (%)</th>
<th>Kuo spr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FWD Illipave</td>
<td>50.5</td>
<td>16.24</td>
<td>1.35</td>
<td>1.99</td>
<td>47.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.71</td>
<td>1.24</td>
<td>1.69</td>
<td>49.86</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>85.04</td>
<td>16.35</td>
<td>1.29</td>
<td>2.03</td>
<td>48.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.71</td>
<td>1.29</td>
<td>1.80</td>
<td>48.82</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>116.14</td>
<td>16.22</td>
<td>1.34</td>
<td>2.04</td>
<td>47.51</td>
</tr>
<tr>
<td></td>
<td></td>
<td>18.58</td>
<td>1.16</td>
<td>1.61</td>
<td>51.21</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>Average</td>
<td>16.27</td>
<td>1.33</td>
<td>2.02</td>
<td>47.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.33</td>
<td>1.23</td>
<td>1.70</td>
<td>49.96</td>
</tr>
</tbody>
</table>

Table 16

Pavement parameter for the Pike County Airport pavement
$y = -1.3976 + 0.36908x \quad R^2 = 0.991$  
Field deflection equation

$y = 0.62575 + 0.30704x \quad R^2 = 0.981$  
Illipave deflection equation

Figure 22
Various applied loads vs vertical deflection between Field and Illipave deflection for Pike County airport
Vertical deflection vs Distance for
the Bellefontaine County Airport

Field deflection
Illipave deflection

$P = 53.56$ psi

Figure 23a

Field deflection
Illipave deflection

$P = 84.21$ psi

Figure 23b
Summary of back-calculated material and the parameter table for the Bellefontaine County Airport are listed below:

2.25" AC \quad E = 400 \text{ ksi}

6" Bituminous \quad E = 300 \text{ ksi}

2" Crushed aggregate \quad E = 30 \text{ ksi}

∞ Subgrade \quad E_{ri} = 9.6 \text{ ksi}
Table 17
Pavement parameter for the Bellefontaine County Airport

<table>
<thead>
<tr>
<th>Applied Load (psi)</th>
<th>Area (in)</th>
<th>F 1</th>
<th>F 2</th>
<th>Vaswani spr (%)</th>
<th>Kuo spr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FWD Illipave</td>
<td>53.56</td>
<td>19.82</td>
<td>0.92</td>
<td>1.29</td>
<td>56.26</td>
</tr>
<tr>
<td></td>
<td>19.07</td>
<td>1.00</td>
<td>1.31</td>
<td>54.59</td>
<td>44.35</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>84.21</td>
<td>19.71</td>
<td>0.93</td>
<td>1.30</td>
<td>55.95</td>
</tr>
<tr>
<td></td>
<td>19.02</td>
<td>1.01</td>
<td>1.34</td>
<td>54.43</td>
<td>44.23</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>115.95</td>
<td>19.71</td>
<td>0.93</td>
<td>1.30</td>
<td>55.94</td>
</tr>
<tr>
<td></td>
<td>19.04</td>
<td>1.00</td>
<td>1.36</td>
<td>54.43</td>
<td>44.21</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>Average</td>
<td>19.75</td>
<td>0.93</td>
<td>1.30</td>
<td>56.05</td>
</tr>
<tr>
<td></td>
<td>19.04</td>
<td>1.00</td>
<td>1.34</td>
<td>54.48</td>
<td>44.26</td>
</tr>
</tbody>
</table>

\[
y = -1.0462 + 0.22144x \quad R^2 = 0.985
\]
Field deflection equation

\[
y = -0.19656 + 0.21652x \quad R^2 = 0.999
\]
Illipave deflection equation

Figure 24
Various applied loads vs vertical deflection between Field and Illipave deflection for Bellefontaine County Airport
Vertical deflection vs Distance for the Seneca county Airport

Figure 25a

Figure 25b
Summary of back-calculated material and the parameter table for the Seneca County Airport are listed below:

2.5" AC  \hspace{1cm} E = 1500 ksi
3" Bituminous  \hspace{1cm} E = 1400 ksi
5" Subbase  \hspace{1cm} E = 34 ksi
∞ Subgrade  \hspace{1cm} E_{\text{ri}} = 5.8 ksi
<table>
<thead>
<tr>
<th>Applied Load (psi)</th>
<th>Area (in)</th>
<th>F1</th>
<th>F2</th>
<th>Vaswani spr (%)</th>
<th>Kuo spr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FWD Illipave</td>
<td>54.01</td>
<td>21.16</td>
<td>0.84</td>
<td>0.77</td>
<td>60.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.37</td>
<td>0.63</td>
<td>0.89</td>
<td>65.52</td>
</tr>
<tr>
<td></td>
<td>89.77</td>
<td>21.16</td>
<td>0.84</td>
<td>0.78</td>
<td>60.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.34</td>
<td>0.63</td>
<td>0.90</td>
<td>65.09</td>
</tr>
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<td>122.08</td>
<td>21.13</td>
<td>0.84</td>
<td>0.78</td>
<td>60.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.38</td>
<td>0.63</td>
<td>0.90</td>
<td>65.20</td>
</tr>
<tr>
<td>FWD Illipave</td>
<td>Average</td>
<td>21.15</td>
<td>0.84</td>
<td>0.78</td>
<td>60.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.36</td>
<td>0.63</td>
<td>0.90</td>
<td>65.27</td>
</tr>
</tbody>
</table>

Table 18
Pavement parameter for the Seneca County Airport
Field deflection equation

\[ y = -0.25138 + 0.16571x \quad R^2 = 0.999 \]

Illipave deflection equation

\[ y = 2.9472 \times 10^{-2} + 0.15957x \quad R^2 = 1.000 \]

Figure 26
Various applied loads vs vertical deflection between Field and Illipave deflection for Seneca County Airport
Since asphalt concrete modulus is highly dependent upon the effect of temperature, consider Table 19 in the following. $T'$ is the average 5 days temperature prior to the deflection data taken. $E$ base on viscosity are taken from Figure 28 correspond to the average of three temperatures given in column 3.

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Temperature (°F)</th>
<th>$E$ (ksi) back-calculated</th>
<th>$E$ (ksi) base on viscosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fairfield I-270</td>
<td>Aug 15, 1988</td>
<td>$T_{air} = 77$</td>
<td>$E_{ave} = 900$</td>
<td>$L$ viscosity = 100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{pav} = N A$</td>
<td></td>
<td>$H$ viscosity = 1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T' = 80.6$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jackson US-35</td>
<td>Sept 19, 1988</td>
<td>$T_{air} = 87.8$</td>
<td>$E_{ave} = 700$</td>
<td>$L$ viscosity = 320</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{pav} = N A$</td>
<td></td>
<td>$H$ viscosity = 1250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T' = 67.1$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pike Airport</td>
<td>Oct 30, 1987</td>
<td>$T_{air} = 75$</td>
<td>$E_{ave} = 1000$</td>
<td>$L$ viscosity = 270</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{pav} = 89$</td>
<td></td>
<td>$H$ viscosity = 1240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T' = 48.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bellefontaine Airport</td>
<td>Sept 17, 1987</td>
<td>$T_{air} = 65$</td>
<td>$E_{ave} = 350$</td>
<td>$L$ viscosity = 250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{pav} = 85$</td>
<td></td>
<td>$H$ viscosity = 1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T' = 70$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seneca Airport</td>
<td>June 24, 1987</td>
<td>$T_{air} = 45$</td>
<td>$E_{ave} = 1450$</td>
<td>$L$ viscosity = 660</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{pav} = 63$</td>
<td></td>
<td>$H$ viscosity = 1970</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T' = 72.3$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 19

The results of asphalt concrete modulus for 5 pavements cases based on back-calculated and temperature

In the case of the following, refer to Figure 27:

1) In the Pike County airport pavement, the back-calculated AC modulus is equal to 1000 ksi. This $E$ corresponds to 50°F in Table 13. The most likely is the AC in the deepest layer was still cold due to cold temperature ($T' = 48.4°F$) before the deflections data were taken. See
the Seneca case also in Figure 27.

2) Jackson US-35 and Bellefontaine pavements give good estimation of $E$ values corresponding to temperature since there was no sudden change in temperature.

3) Modulus of AC for Fairfield is higher than $E$ corresponding to the temperature. This is because of pavement behaved as rigid pavement with high modulus of elasticity.

![Figure 27](image)

**Figure 27**

Comparison between $E$ back-calculated and $E$ actual for 5 pavements cases

As mentioned earlier in chapter 2, flexible pavement consists of several layers including the binder course. This binder course is affected by temperature and it is believed to be the main cause of pavement cracking [31]. The majority of pavement cracks are due to a hardening of the asphalt
binder (see Figure 28 below and the result in Table 19).

For 5 pavement cases, 4 of them correspond to high viscosity, with the exception of Bellefontaine pavement. For an effective design it is better to

---

**Figure 28**
Modulus of asphalt concrete related to viscosity and temperature [31]
use lower viscosity than a higher binder course. On the other hand, a high viscosity binder course will provide more resistant to the traffic load. Under severe temperatures, however, high viscosity will produce cracking.
SUMMARY AND CONCLUSION

An increase in the development of sophisticated computer programs for use in evaluating the pavement structural performance initiated this research study. One of these computer programs, Illipave, was developed through using 112 conventional pavement cases in Illinois.

The evaluation presented in this thesis is based on the measurements on the layer by layer deflection and overall deflection. The field deflections were measured by falling weight deflectometer for the Fairfield I-270 and Jackson US-35 highways as well as three county airports: Pike, Bellefontaine and seneca.

By using the statistical computer program SAS, field data (FWD data) were analyzed in chapter III. The best curves for load versus FWD deflection are obtained from the Cricket program.

The parameter used in the analysis are circular load with 12" diameter, material of the pavement components and environmental effects. An example of the input program is given in Chapter IV. Trial and error material components are used as an input to the program to predict modulus in every layer to best fit in the field deflection basin. The pavement performance results and discussion are presented in chapter V.
The results from data analysis have shown that

1. For subgrade deflections data only, almost all sensors have cubic equations with $R^2$ ranging from 0.913 to 0.997. This result indicates that subgrade layer is nonlinear material, which is in agreement with laboratory results where subgrade soils display non-linear for stress-strain relationship.

2. For layer pavement consisting of subgrade and subbase (case of second layer of Jackson US-35), all sensors still have a strong correlation of nonlinear equations.

3. Where the pavement consists of subgrade (+ may or may not subbase) and linear asphalt layers, the first sensor tends to show linear equation. Since the asphalt layer is treated as a linear model, it is reasonable that the first sensor gives a linear result. Meanwhile the seventh sensor remains a cubic equation.

4. It seems that from all 7 sensors, the seventh sensor usually has lower $R^2$ value compared to the $R^2$ value from other sensors. It is most likely this 7th sensor represents the subgrade condition.

The FWD data available for layer by layer pavement has an advantage in back-calculating the value of layer modulus more accurately. The layer by layer results for highways are:

1. The subgrade layers are very sensitive to further sensors, especially sensor number 5, 6, 7.

2. The results show that applied loads in the range of 50 psi to 125 psi give better results than lower or very high loads.

3. For layers consisting of asphalt concrete, the first 4 sensors are very
sensitive.

4. The result of these 2 highways are not very satisfy for overall layers.

One possible explanation is that Illipave was developed for conventional flexible pavement, AC layer: 0" - 8" and granular base: 4" - 24". So, consider these:

1) **Fairfield I-270 case**

Subgrade + 4" AC give very good results in the range of sensitive loads. With subgrade + 8" AC the result is still good. When the AC layer is added over an 8" thickness, the first sensor and seventh sensor no longer match while the second sensor becomes more sensitive. There is no changing in resilient modulus of subgrade when the layers are added (see Figure 29).

![Figure 29](image_url)

**Figure 29**

Subgrade resilient modulus vs pavement sublayer for Fairfield I-270
2) **Jackson US-35 case**

The layer where AC = 8" gives the most satisfactory result. There are, however, unpredictable results in the solution (see Figure 30).

a) Subgrade layer only, gives $E_{ri} = 14$ ksi

b) When the subbase layer is added, $E_{ri}$ becomes 22 ksi. Probably this is due to compaction, weather conditions and more stress when the second layer data was taken. In the Fairfield case, the problem did not arise for modulus of subgrade (refer to Figure 29) because this pavement can be considered as full depth asphalt pavement. Therefore, it behaves like rigid pavement. Most likely the subgrade layer has only a slight effect on the pavement performance.

c) The final $E_{ri}$ value = 26 ksi and $E$ for bituminous aggregate changes from 750 ksi to 650 ksi in the last layer. This is mostly due to the stress and the effect of temperature and also because deflection data for the last layer was taken approximately 2.5 months after the bituminous deflection data was taken.
5. Compared to highway pavement solutions, airport back-calculated properties give a better result, because:

1) Layer by layer material in the pavement cross section is given more specific. So, there is no ambiguity in assuming value of E.

2) All three airports have AC thicknesses near or less than 8" which make them sensitive to the Illipave solution.

3) The pavement temperature was measured along with air temperature.

4) The soil selection for the airport location is probably better than highway soil conditions.

5) The location of the airport have a more uniform subgrade over a shorter distance compared with highway distance.

6. Pavements with base/subbase layers less than 4" gave excellent results (in the case of Bellefontaine airport pavement).
7. From the Pavement Parameter tables, we can conclude that:

The spreadability percentage presented by Vaswani gives higher value than the spreadability percentage by Kuo. The reason involves the accuracy of the deflection data. I think Kuo gives a better prediction for pavement condition because of 7 sensors deflection involved. The spreadability percentage will also indicate the life of the pavement service. The Bellefontaine pavement has lower spreadability percentage than the Pike pavement because the Pike airport was built after the Bellefontaine airport.

The results of this study provided the following major conclusions:

1. In general FWD is a good non-destruction method to obtain field deflection data.

2. FWD data taken layer by layer is a good method which gives a better prediction of the resilient modulus of the subgrade, especially for rigid and full depth flexible pavements.

3. From data analysis, it shows that all sensors for subgrade have nonlinear relationships. This agrees with laboratory results. Subgrade soils display nonlinear relationships for stress-strain.

4. Illipave gave the best response for the applied load between 50 psi to 125 psi.

5. For pavements consisting of AC with less than an 8" thickness, the first sensor is very sensitive to the pavement performance. For AC greater than an 8" thickness, pavement performance is probably represented by sensor number 2. Sensor number 7 represents the subgrade performance.

6. In general, airport pavement performance give better results than
highway performance. Reasons are given above.

7. An average of five days of air temperature prior to deflection measurements play an important role in back-calculating pavement properties. Because, if there is a sudden change in the temperature, the average of five days temperature can be considered when back-calculating the AC modulus.

8. It seems that a lot of pavements used asphalt binder with high viscosity rather than with low binder viscosity. Resistance to traffic load is probably more important than worrying about pavement cracking.

9. Pavement parameters can give better information about the pavement performance, such as the accuracy of the results and conditions of the pavement.

10. In general the Illipave solution for flexible pavement is sensitive to asphalt thickness, modulus of asphaltic concrete, resilient modulus of subgrade and subbase thickness.

The results of this study indicate that, generally, Illipave is suitable for back-calculating material of the pavement components. Probably the weakest part is that Illipave is a time consuming and an expensive computer program to run.
Recommendation

**Recommendation for evaluation new pavement**

Evaluation of new pavement is a very important step for not only determining the pavement service life, but also for deciding the properties of the materials to be evaluated later. If evaluating the new pavement by using the layer by layer method, consider the following:

1. Take many FWD deflections data for the subgrade layer after compaction. Deflections data must cover most parts of the unfinished pavement.

2. Before each layer is added, take more FWD deflection data. Especially when adding one layer to another take some time.

3. Record asphalt temperature along with air temperature. These asphalt temperature readings are very handy for back-calculating modulus of bituminous aggregate and asphalt concrete. Take several asphalt temperature readings during the FWD test.

4. Identify the types of base / subbase materials. This will ease the prediction of pavement performance for most materials. K and n values of base / subbase material have been evaluated in the lab.

5. To reduce operational time and cost, take deflection data only for sensitive applied loads (50 psi to 125 psi).

If overall method will be used, FWD deflection data for the subgrade layer must be taken. This is very important because sometimes it is very hard to determine the type of subgrade by only relying on FWD deflection data from the top of the pavement. This subgrade deflections data is very useful when evaluating Full depth flexible and rigid pavements.
Evaluation should be carried on each season in the first year of pavement life. Also a study on volume of traffic passing through the new pavement should be made. After that, evaluations can be done at specific time intervals. If an extreme weather condition exists, get FWD deflections data if possible.

Keep track of new traffic records, because, often pavement which is designed for a certain life span shows a lot of cracks and bumps before the end of its intended life of service. This is mostly due to traffic increase and hardening of the binder asphalt layer. By constantly recording traffic flow, the remaining pavement life can be better predicted. If an overlay is needed, it is better to get deflection data before the pavement overlay is made. In this way a comparison can be made regarding the strength of the pavement after pavement overlay has been completed.

Recommendation for future research

For future research, evaluation in following areas can be considered:

1. The remaining life of the existing pavement

This can be developed from the concept of pavement failure, based on the traffic study, back-calculated tensile strain at the bottom of asphalt concrete layer and vertical compression strain at the top of the subgrade.

2. Flexible boundary can be used for the Illipave mesh diagram.

To accommodate the nonlinear behavior of material used in the pavement, traditional finite element meshes need to provide infinitely deep subgrade layer. This is not convenient because of the time consuming
task of running the program. The flexible boundary reported by Ronald Harichandran and Ming-Shan Yeh in the report of "Flexible Boundary in F.E Analysis of Pavement" can be used as reference [33]. Here it was stated that the flexible boundary gave significantly better results.

3. Results of this study can be compared with laboratory study and other back-calculated materials from different seasons as the deflections data become available.
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