CHAPTER 1

A LITERATURE SURVEY ON DEFLECTION BASIN MEASUREMENTS

CHAPTER 1: CONTENTS

		PAGE
1	INTRODUCTION	1.2
2	DEFLECTION BASIN PARAMETERS	1.5
2.1	Review of parameters	1.5
2.2	Evaluation of parameters	1.7
3	DEFLECTION MEASURING	1.19
3.1	Review of devices and methods	1.19
3.2	Evaluation of devices	1.21
3.3	Correlations between devices	1.31
4	CONCLUSIONS AND RECOMMENDATIONS	1.32

1. INTRODUCTION

In the development of fundamental methods of pavement evaluation by means of deflection measurements it is imperative that an accurate description of the deflection basin be given. In the majority of analysis procedures only the maximum surface deflection (δ_o) is measured (Epps and Hicks, 1982). Owing to the empirical nature of analysis techniques in the past, a more detailed description of the deflection basin has been neglected. In Figure 1.1 it can be seen that the same maximum deflection value, δ_o, can be measured on two pavements with totally different deflection basins and structural response characteristics. Whitcomb (1982) even concludes from various examples of this phenomenon that "....resilient moduli for layers in a pavement system cannot be back calculated using maximum surface deflection alone". In their analysis of Alascan highways during the spring thaw period Stubstad et al. (1983) show that the same maximum deflection value could lead to wrong assumptions of thaw depth and resulting damage potential. It is only by looking at the whole deflection basin that preventative predictions of thaw depth can be made. Although South Africa does not have pavement distress due to thaw as in North America, it is significant that the whole deflection basin can be used to indicate a change of state.

Paterson et al. (1974) states that when a pavement deflects under a load, the influence of the load extends over a certain area. In one dimension and for one depth this can be regarded as a deflection profile or influence line of, say, the surface. In two dimensions the deflection at any depth is given by an influence surface. The shape of the influence surface reflects the structure of the pavement. In Figure 1.2 the typical deflection profiles for a uniform circular load and the more complex shapes of dual wheel single axle loads and how front and back axles influence each other ,are shown. The depth profile of the deflection closely reflects the stiffness depth profile of the pavement in relation to the relevant stress levels.

This is best summarised by the Technical Committee Report of the XVII World Road Congress (Permanent International Association of

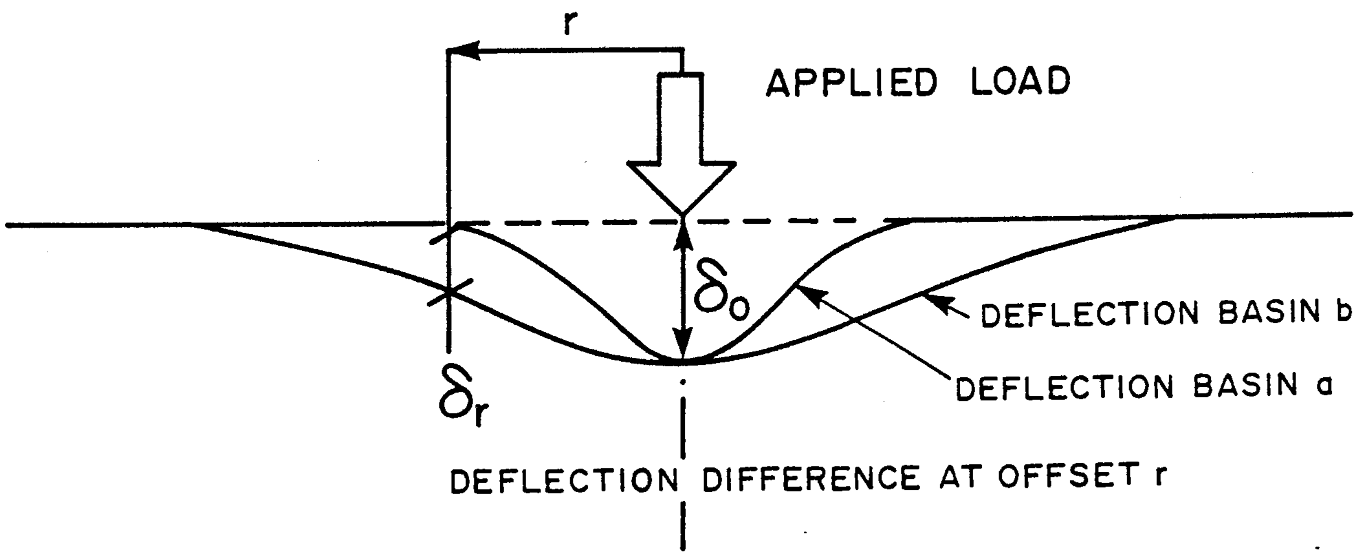
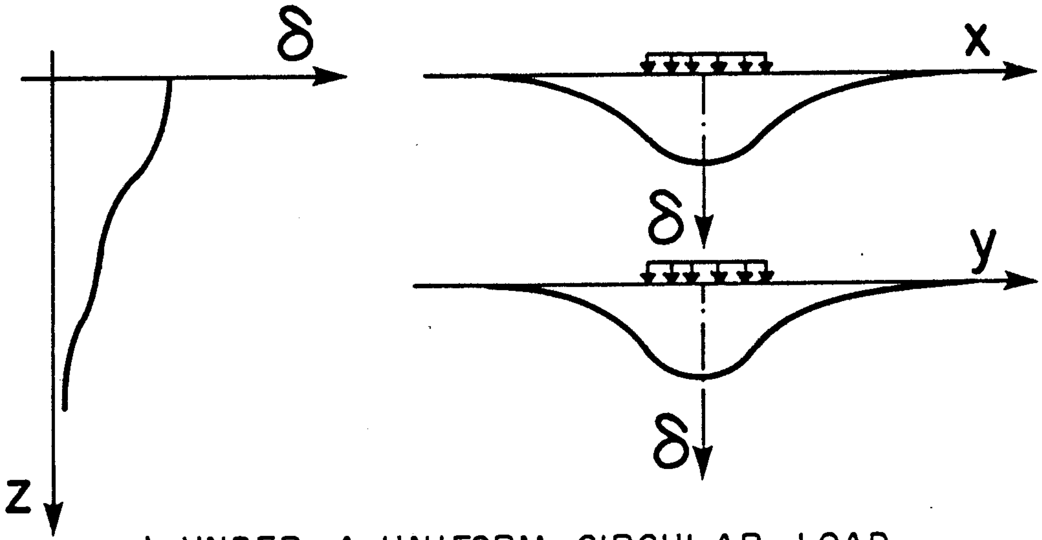
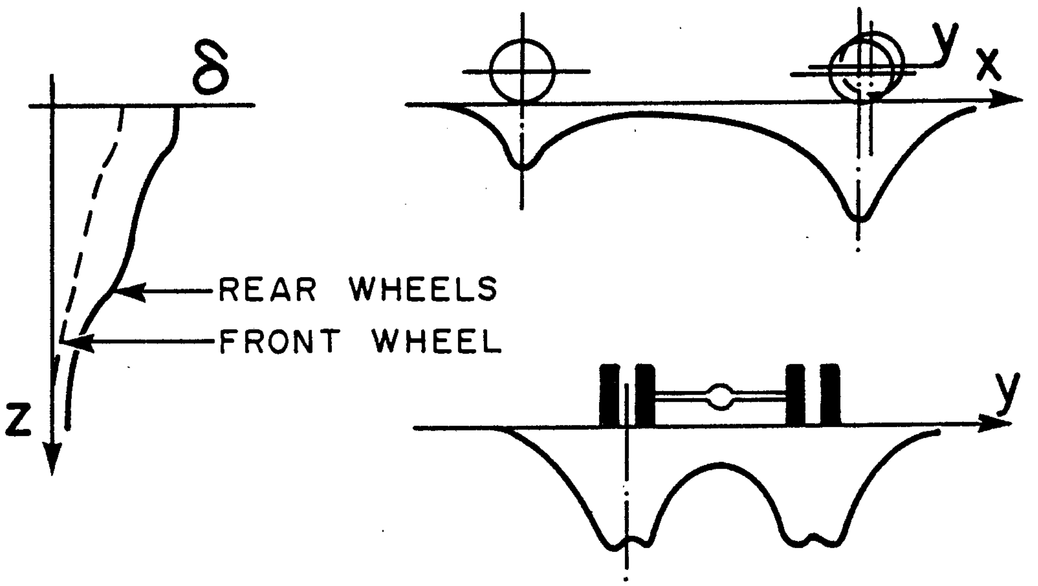


Illustration of the same maximum deflection for two different deflection basins.

FIGURE 1.1
DEFLECTION BASIN ILLUSTRATION.



a) UNDER A UNIFORM CIRCULAR LOAD



b) UNDER A SINGLE REAR AXLE DUAL WHEEL TRUCK

FIGURE 1.2
TYPICAL SHAPES OF DEFLECTION INFLUENCE LINES (Dehlen, 1962b)

Road Congresses,1983). "There are obvious attractions in making maximum use of the information that can, in principle, be obtained by detailed evaluation of the deflected shape as it obviates or reduces the need for assumptions about, or measurement of, properties of pavement materials."

This introductory discussion clearly indicate that there is a need for the measurement and interpretation of the full deflection basin. As will be discussed in later chapters, in South Africa the full deflection basin is measured, but the analysis historically focussed on the small area surrounding maximum deflection. In order to make better use of the full deflection basin , a literature survey was conducted. This literature survey covers the work done overseas with other measuring equipment , description methods of the full deflection basin , evaluation of various deflection basin parameters and devices. In order to bring it in line with the South African scene , all these discussions are related to the practice here . Recommendations are made by this author to enhance the current practice and use of South African devices and deflection basin description.

2 DEFLECTION BASIN PARAMETERS

2.1 Review of Parameters

Surface deflections are generally measured by sensor(s) of various kinds located in a fixed line and normally at a fixed distance relative to the centriod of the load area. Deflection basin parameters differ in their relation to the measuring device and degree to which they describe the essential features of the deflection basin. In Table 1.1 a summary is given of the parameters, formulae, test method or device normally associated with it and at least one reference.

It is suggested that the fold-in of Table 1.1 at the back be folded out to ensure ease of reading as exhaustive reference will be made to the various deflection basin parameters in the sections to follow, as well as the chapters to follow.

TABLE I.I SUMMARY OF DEFLECTION BASIN PARAMETERS

Parameter	Formula	Measuring device	Reference
1. Maximum deflection	δ_o	Benkelman beam Lacroix deflectograph	Kennedy, et al. Asphalt Institute(1978)
2. Radius of curvature	$R = \dfrac{r^2}{2\delta_o(\delta_o/\delta r - 1)}$ $r = 127$ mm	Curvaturemeter	Dehlen (1962a)
3. Spreadability	$S = \dfrac{[(\delta_o+\delta_1+\delta_2+\delta_3)/5]100}{\delta_o}$ $\delta_1 .... \delta_3$ spaced 305 mm	Dynaflect	Vaswani (1971)
4. Area	$A = 6[1+2(\delta_1/\delta_o) + 2(\delta_2/\delta_o) + \delta_3/\delta_o]$	Falling weight deflectometer(FWD)	Hoffman and Thompson (1981)
5. Shape factors	$F_1 = (\delta_o - \delta_2)/\delta_1$ $F_2 = (\delta_1 - \delta_3)/\delta_2$	FWD	Hoffman and Thompson (1981)
6. Surface curvature index	$SCI = \delta_o - \delta_r$; where $r = 305$ mm or $r = 500$ mm	Benkelman beam Road rater FWD	Anderson (1977) Kilareski,at al.(1982) Molenaar (1982)
7. Base curvature index	$BCI = \delta_{610} - \delta_{915}$	Road rater	Kilareski,et al.(1982)
8. Base damage index	$BDI = \delta_{305} - \delta_{610}$	Road rater	Kilareski,et al.(1982)
9. Deflection ratio	$Q_r = \delta_r/\delta_o$ where $\delta_r \simeq \delta_o/2$	FWD	Claessen and Ditmarsch (1977)
10. Bending index	$BI = \delta/a$ where a = Deflection basin	Benkelman beam	Hveem (1955)
11. Slope of deflection	$SD = \tan^{-1}(\delta_o - \delta_r)/r$ where $r = 610$ mm	Benkelman beam	Kung (1967)
12. Tangent slope	$ST = (\delta_o - \delta_r)/r$ where r is determined by a polynomial function	Benkelman beam FWD	University of Dundee (1980)
13. Radius of influence	$RI = R'/\delta_o$ where R' is the distance from δ_o to where basin is tangent to horizon.		Ford and Bissett (1962)

The first two parameters listed in Table 1.1 are the traditional maximum deflection and radius of curvature. Their formulas show that only the small area surrounding maximum deflection (positive curvature) is utilized. The spreadibilty (S) and area (A) parameters obviously cover the full deflection basin. Parameters such as the shape factors (F1 and F2), surface curvature index (SCI), base damage index (BDI), base curvature index (BCI), slope of deflection (SD) and tangent slope (ST) also cover more than only part of the area of positive curvature near the load. In fact some of the aforementioned parameters try to describe either the area of positive curvature or the area of reverse curvature or the important area surrounding the point of inflection where the positive curvature changes to that of the reverse curvature. The other parameters not mentioned, do describe the deflection basin better than maximum deflection and radius of curvature, but it is not clear in which area of the deflection basin they fall with their description.

2.2 Evaluation of Parameters

The Technical Committee Report on flexible Roads of the XVII World Road Congress (Permanent International Association of Road Congresses, 1983) states; "The transient displacement or deflection of the road surface represents the sum of all the vertical strains in the pavement and sub-grade and remains the most widely used measurement of structural condition. Its advantages are, the relative simplicity of the measurements, the large amount of experimental data that already exists and the strong correlation found between deflection and overall performance in well defined conditions. It is however not very responsive to changes in the stiffness of upper pavement layers and is not a unique measurement of performance on all types of pavement."

A sensitivity analysis done at the University of Dundee (1980) incorporated most of the parameters listed in Table 1.1. A three-layered pavement system was analysed. All the structural parameters were varied. The sensitivity analysis was done by converting to a standard dimensionless unit. The conclusions and summary of this study are briefly as follows:

Irrespective of the technique or measuring device, the parameters, bending index (BI) and radius of influence (RI) are difficult to determine. This is due to the fact that the length of the deflection basin is normally too long to measure accurately in situ. The value of maximum deflection (δ_o) was found to be unreliable when used alone owing to the difference in pavement conditions. This confirms the illustration in Figure 1.1 and conclusion by Whitcomb (1982) in Section 1.

Radius of curvature parameter (R) showed a high sensitivity to most changes in the pavement structural parameters, but was insensitive to subgrade elastic modulus. The deflection ratio (Q or Qr) and spreadability (S) showed even less sensitivity to changes in the other structural parameters. Contrary to this conclusion Koole (1979) reports that the parameter Q is a reliable parameter. The inconsistency seems to stem from the lack of adherence to Koole's (1979) precondition ($\delta_r \approx \delta_o/2$) in determining Q. It seems that using a fixed value of $\delta_r = \delta_{610}$, as in this analysis done by the University of Dundee (1980), can result in the recorded insensitivity to changes in the pavement structural parameters. If it is taken into consideration that this parameter was developed for a specific type of pavement structure, it is to be expected that variances in structure will definitely influence its sensitivity. Rohlf et al. (1985) indicate that spreadability (S) is an indication of the ratio of the surface layer to support layer strengths.

In this sensitivity analysis by the University of Dundee (1980) the slope of deflection (SD) and tangent slope (ST) have shown a high sensitivity to changes in the pavement structural parameters. It is concluded, though, that the slope of deflection (SD) may have the same unreliability as the maximum deflection (δ_o) when used alone. The reason for this is that, as illustrated in Table 1.1, a fixed value of $\delta_r = \delta_{610}$ is used. The tangent slope, on the other hand, makes use of a polynomial function to describe the deflection basin and form the basis for the selection of δ_r unambiguously. Dehlen (1962b) also observed that R is dependent mainly on the moduli of the upper layers of construction, and very little on those of the materials at depth. He states that; "The radius of curvature of a road surface under a given vehicle is dependent mainly on the Young's moduli of the materials in the base and subbase;... "He also observed that R is dependent to a considerable degree on tyre pressure and only to a lesser degree on the wheel load while δ_o is dependent on wheel load and little on tyre pressure.

Hoffman and Thompson (1981) developed the area (A) parameter from work done by Vaswani (1971). It is obviously related to the spreadability (S) parameter and an attempt to incorporate the full deflection basin.

The surface curvature index (SCI) indicates the strength of the upper portion of a pavement according to an analysis of standard pavement structures in the State of Victoria, Australia by Anderson (1977). Generally in these pavement structures, as in the dry regions of South Africa, rather thin ($\pm$ 50 mm) asphalt surfacing layers are used. In Anderson's (1977) study, maximum deflection (δ_o) was used simultaneously to describe the response of the lower portion of pavements successfully.

Kilareski et al. (1982) state that the base curvature index (BCI) indicates the strength of the lower portion of the pavement system. The basis for these statements concerning SCI and BCI can be seen in Figures 1.3 and 1.4 where the related structural parameters are varied. It can be seen in Figure 1.3 that a difference in deflection at 0 and 305 mm (SCI) will reflect the change in base elastic modulus. In Figure 1.4 the difference in deflection at 610 mm and 915 mm (BCI) will reflect the change in subgrade elastic modulus.

The Technical Committee Report on flexible roads of the XVII World Road Congress (Permanent International Association of Road Congresses, 1983) concludes on the deflected shape; "The

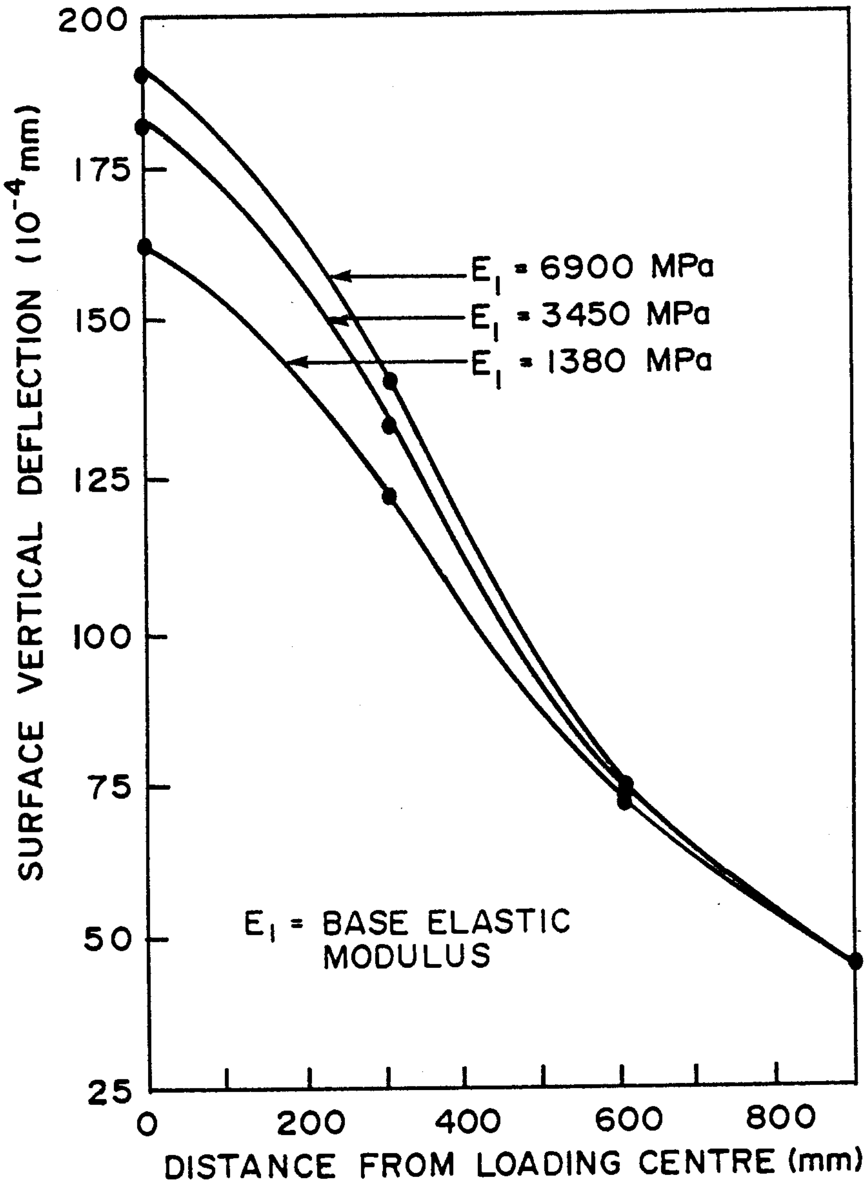


FIGURE 1.3
VARIATION OF SURFACE DEFLECTION BASIN WITH SURFACE MODULUS, E_1
(Kilareski, et al., 1982)

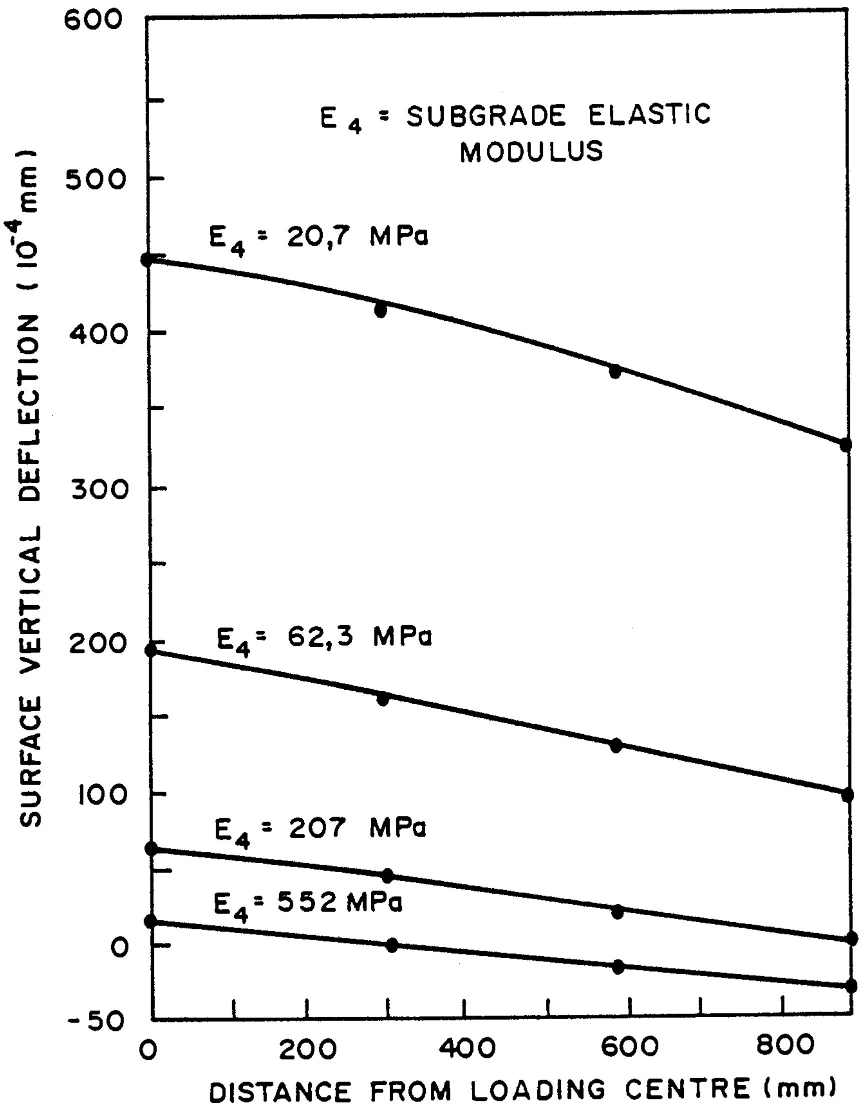


FIGURE 1.4
VARIATION OF SURFACE DEFLECTION BASIN WITH SUBGRADE MODULUS, E_4
(Kilareski, et al., 1982)

deflected shape near the point of maximum deflection is sensitive to changes in upper pavement layers and relatively unaffected by the subgrade. Deflection levels and their rate of change further from the maximum have been related to the stiffness of lower pavement layers and the subgrade."

Very interesting information is available on how the various deflection basin parameters relate to increase in number of standard axle repetitions. Kilareski et al. (1982) produced the typical results of deflection versus number of equivalent axle repetitions as shown in Figure 1.5. SCI (difference between Sensors 1 and 2) and the BCI (difference between Sensors 3 and 4) are virtually constant with the increase in number of standard axle repetitions. From this follows the need for the development of the base damage index (BDI) which is the difference between Sensors 2 and 3). The values of the BDI change with the increase in repetitions of standard axles. Figure 1.6 results from work done by Molenaar (1983); in spite of the difference in measuring technique and device (see Table 1.1) it shows considerable support for this approach of relating the surface curvature index (SCI) to the repetitions of standard axles.

Rohlf et al. (1985) did a multivariate analysis of pavement Dynaflect deflection data. A relationship between Dynaflect deflections and pavement temperature, subgrade moisture, and cumulative traffic loading for a number of different pavement sections was developed. A typical Dynaflect deflection basin measurement is illustrated in Figure 1.7. The major conclusion from this study is that the base thicknesses and base layer elastic moduli had significant effects on the sensor deflections. The first sensor deflection (maximum) was directly related to the base thickness.

Tam (1985) analysed three-, four- and five-layered pavement structures and determined how the variation of structural inputs affected the deflection basin. Figure 1.8 summarizes the relative importance of the effect of varying the stiffnesses of

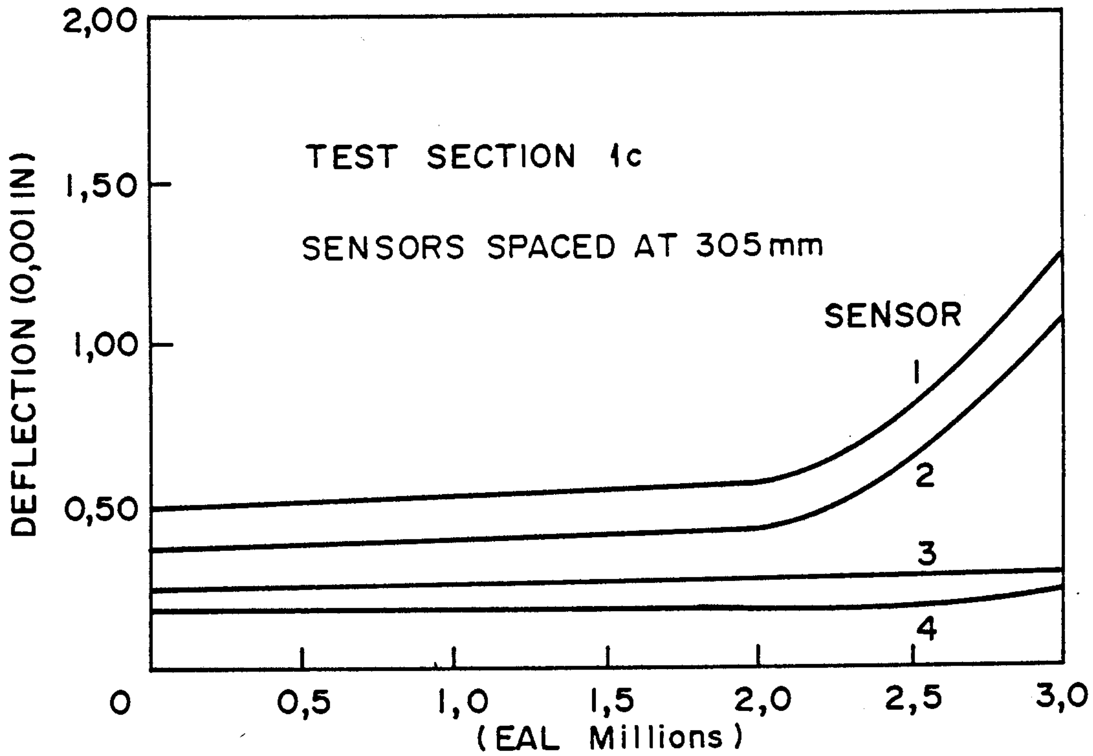


FIGURE 1.5
CORRECTED ROAD RATER SURFACE DEFLECTIONS VS. 18-KIP (80 kN) EQUIVALENT SINGLE-AXLE LOAD
(Kilareski, et al., 1982)

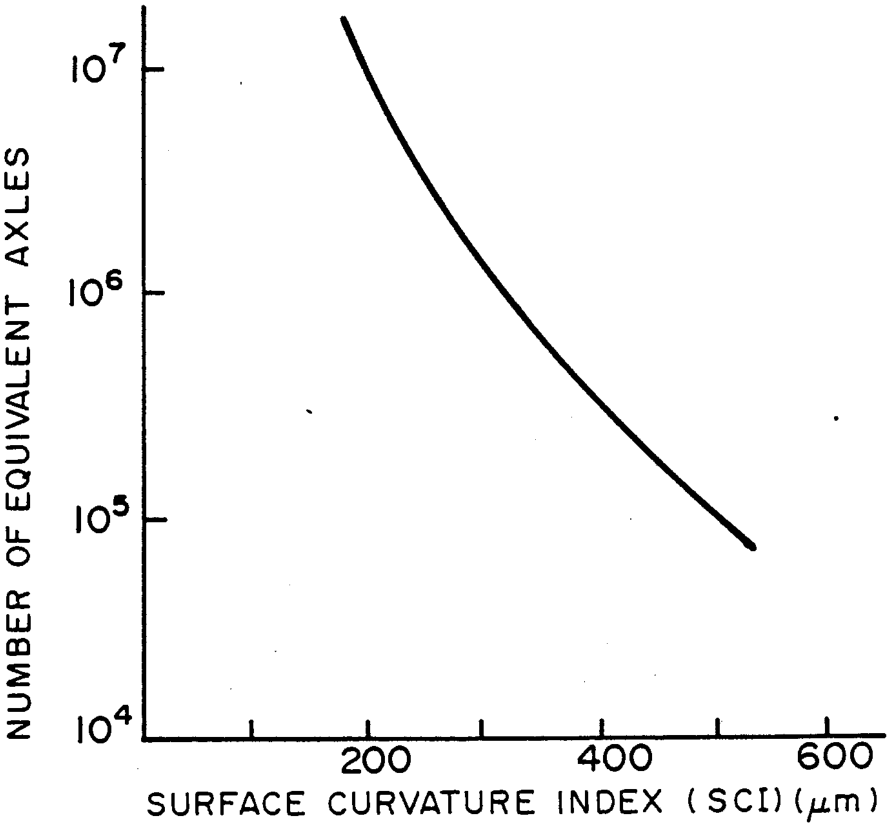


FIGURE 1.6
EXAMPLE OF A PAVEMENT DESIGN CURVE BASED ON THE SURFACE CURVATURE INDEX (SCI) (Molenaar, 1983)

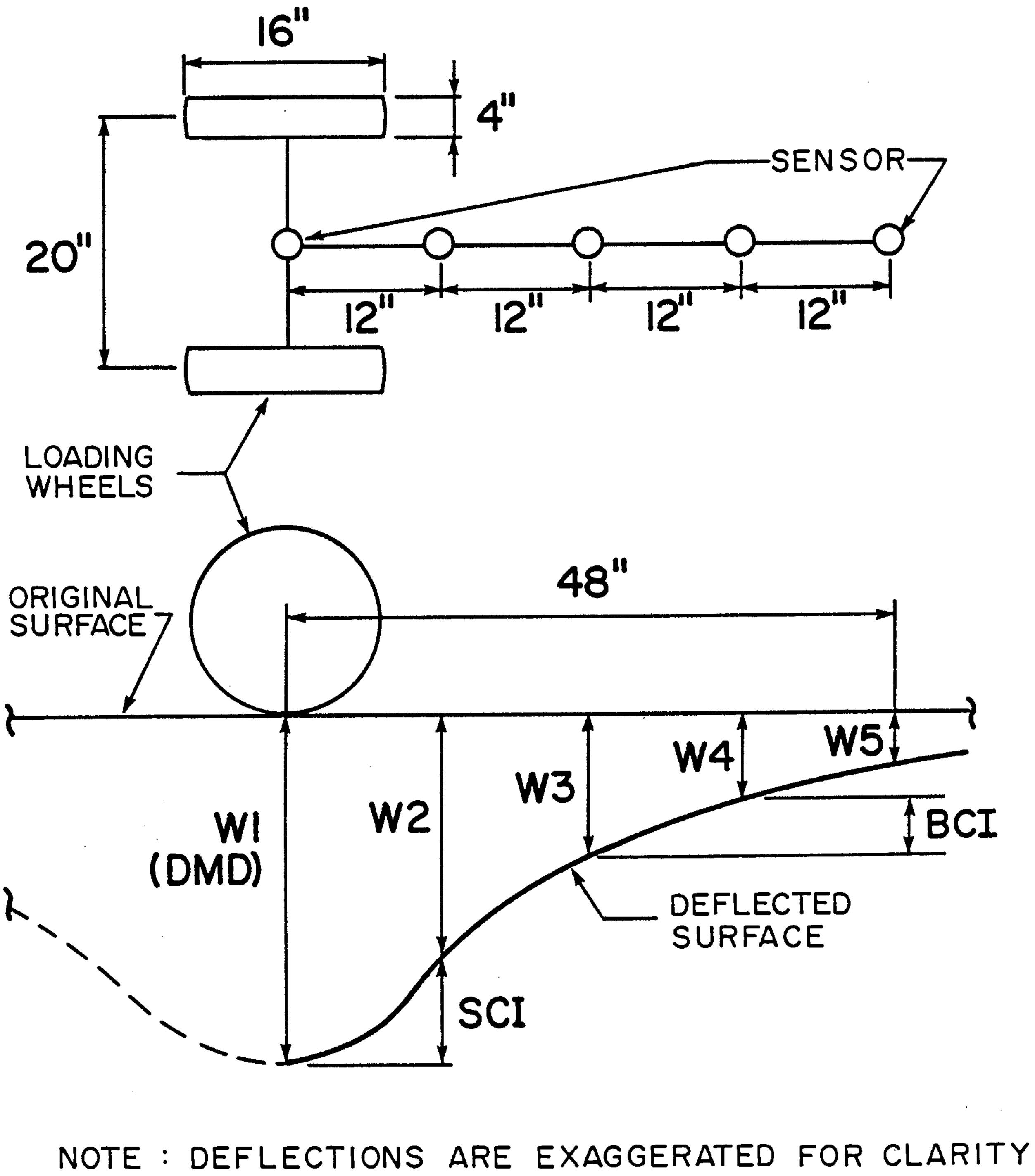


FIGURE 1.7

DYNAFLECT DEFLECTION BASIN (Rohlf, et al., 1985)

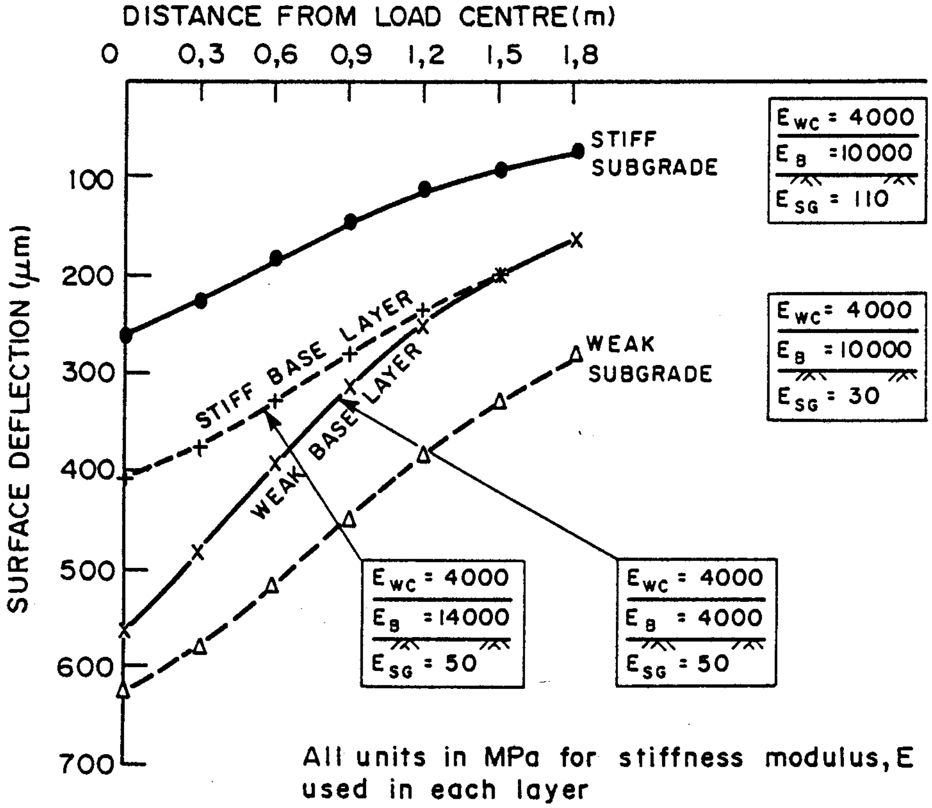


FIGURE 1.8

RELATIONSHIP BETWEEN VARIATION OF STIFFNESS MODULI IN BASE LAYER AND SUBGRADE (Tam, 1985) the base layer and the subgrade in a three-layered pavement structure. The main conclusions from the Tam study on three-layered pavement structures are as follows:

(a) The variation of base layer thickness had the greatest influence on maximum deflection (δ_o) and spreadability (S). This was followed by the subgrade stiffness and base stiffness. Spreadability (S) had seven deflections incorporated in the calculation instead of the normal five (see Table 1.1).

(b) As the subgrade stiffness increases (decreases), maximum deflection (δ_o) and spreadability (S) decrease (increase) (see Figure 1.8).

(c) The increase (decrease) in magnitudes of pavement structural parameters reduces (increases) maximum deflection (δ_o), but increases (reduces) spreadability (S).

In Figure 1.9 the typical four-layered pavement system, as analysed as reference system by Tam (1985), is shown. The spreadability (S) and maximum deflection (δ_o) was normalized by a ratio to the value of the reference structure. The main conclusions here were the same as for the three-layered pavement system regarding the influence of the base and subgrade. It also showed that maximum deflection (δ_o) and spreadability (S) were hardly affected by the change of sub-base parameters at all. The implications are that the actual stiffnesses of the subbase will be difficult to determine with accuracy in evaluating existing pavement conditions and high accuracy on subbase thicknesses will not be a prerequisite for analysing such pavements. In the South African context this is a typical granular subbase pavement.

In Figure 1.10 the typical reference five-layered pavement structure is shown. In this case a cemented subbase in the pre-crack phase was analysed. The sensitivity analysis on maximum deflection (δ_o) and spreadability (S) are also shown.

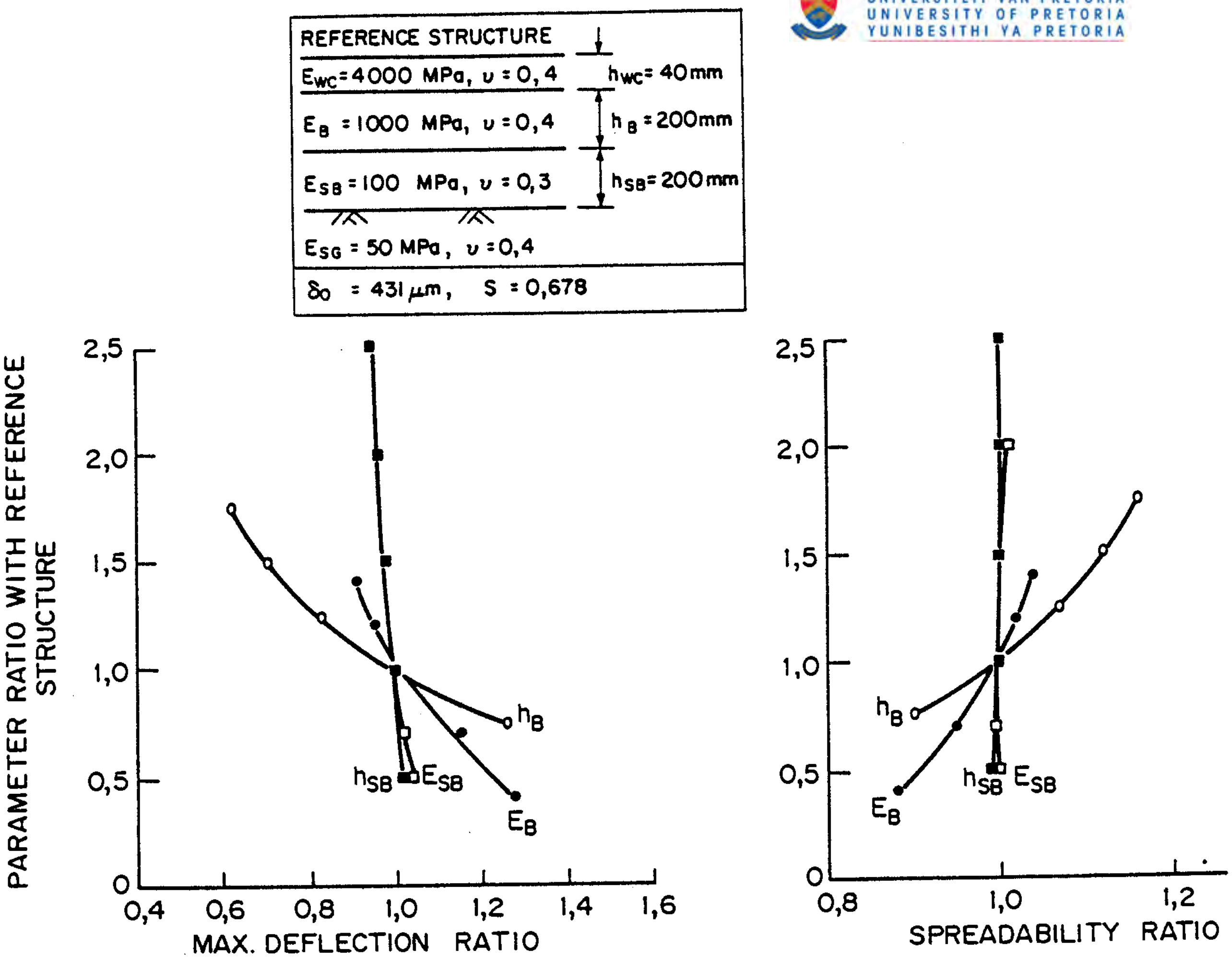


FIGURE 1.9

RELATIVE SENSITIVITY OF PARAMETERS OF SUBBASE LAYER (Tam, 1985)

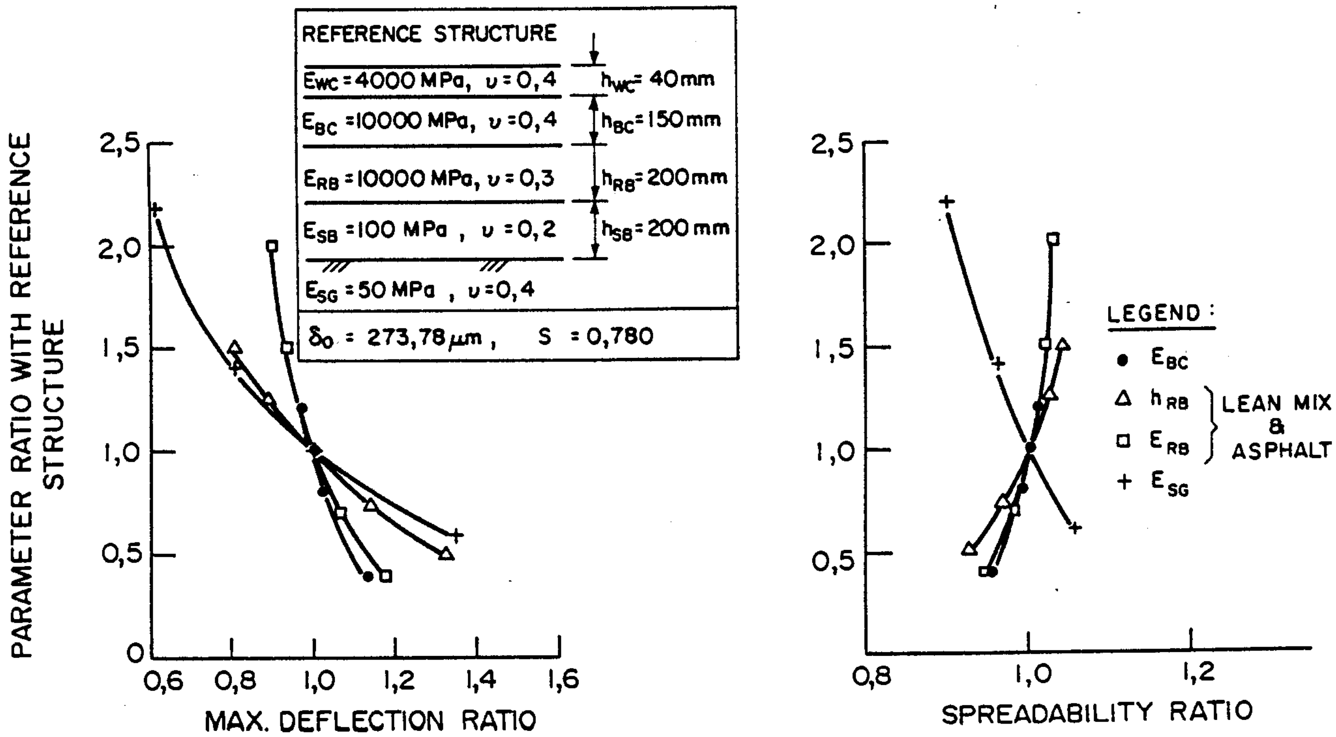


FIGURE 1.10

RELATIVE SENSITIVITY OF PARAMETERS OF LEAN MIX AND ASPHALT BASE LAYER (Tam, 1985)

The main conclusion here was that deflections are sensitive to the variation of thickness of the lean concrete subbase and the asphalt base. Hence, it is important to know what their actual thicknesses are to enable one to evaluate the condition of the existing pavements with confidence.

3 DEFLECTION MEASURING DEVICES AND METHODS

3.1 Review of devices and methods

Pavement deflection devices made their appearance in 1938 in California. The California Division of Highways installed the General Electric Travel Gauge state-wide in that year. This type of device developed evolutionarily. This process was enhanced by the use of these devices at test tracks such as those at Brighton and Stockton, and at WASHO and AASHO road tests. (Whitcomb, 1982). Since then the use of non-destructive deflection measuring devices has become standard practice world wide.

The development of new equipment has been prompted by the need to:

a) increase the testing production rate
b) increase the accuracy of measurements
c) simulate moving traffic loads in terms of magnitude of load, shape and equivalent time of loading
d) provide simplicity of operation and interpretation of results
e) reduce the cost of testing.

Most deflection testing equipment can be classified by loading mode. Generally these devices fall into three main groups: static or slow-moving wheel devices, steadystate vibratory devices and impulse or falling weight devices. Table 1.2 lists a brief summary of the devices as classified by Monismith (1979).

Whitcomb (1982) mentions several other sophisticated devices. The moving vehicle device and accelerometers which need instrument installation in the pavement, cannot strictly be regarded as non-destructive deflection basin testing. Others mentioned are a deflection beam measuring perpendicular to a passing vehicle, laser technology and new techniques in photogrammetry. The road surface deflectometer (RSD) and multi-depth deflectometer (MDD) developed in South Africa for use with the Heavy Vehicle Simulator (HVS) described by Freeme et al. (1981) can also be seen as recent developments. Strictly speaking the MDD is a destructive deflection measuring device in terms of its installation procedure.

TABLE 1.2 - Deflection measuring devices

Method by which load is applied	Device	Organization by which used (Examples)
Slow-moving Wheel	Benkelman beam	Asphalt Institute, College Park, Maryland
	Road surface deflectometer (RSD)	NITRR, CSIR, South Africa
	Travelling deflectograph	California Department of Transportation
	Deflectograph	Transport and Road Research Laboratory, Great Britain. National Institute for Transport and Road Research, S.A. Main Roads Department, New South Wales, Australia, LCPC, France
	Curviameter	LCPC, France.
Vibratory load	Light vibrators, e.g. Road raters and Dynaflect	States: California, Kentucky, Louisiana, Utah. Federal Highway Administration
	Heavy vibrators	U.S. Army Corps of Engineers, Waterways Experiment Station
Falling weight	Falling weight deflectometer (FWD)	Shell Research, Amsterdam

3.2 Evaluation of devices

Whitcomb (1982) states that criteria for determining the "best" deflection testing device from a technical standpoint are difficult to describe. Various comparative studies have been made. In his comprehensive study Bush (1980) compared the operational characteristics (for example ease, speed and manpower requirements), costs (initial and operating), accuracy, reproducibility of measurements and depth of influence of several devices. The results are summarized in Table 1.3.

Relative values of 1 to 10 were used with 1 being the most desirable and 10 the least. The results of this table clearly indicate a preference for the Dynaflect Model 2008, Road Raters and FWD.

The most comprehensive recent study comparing non-destructive testing (NDT) devices and methods for use in the overlay design of flexible pavements is that by the Federal Highway Administration (FHWA, 1984). User comments on the various devices were collected from all over the world. The following factors were considered:

(a) time in service
(b) crew size
(c) professional qualifications of crew
(d) number of test points per day
(e) cost per test point
(f) maintenance costs
(g) traffic control costs
(h) data recording methods
(i) data storage
(j) towing vehicle.

Typical results are summarized in Table 1.4. Most of the variation is apparently due to a difference in user rather than equipment. It does, however, give a good guide. It is obvious that the device that comes out best will depend on which factor is selected.

Another comparative study, also from the FHWA (1984) study, is shown in Table 1.5. It covers a wide range of criteria and the final result indicates that the FWD is favoured. The Road Raters and even the Dynaflect are not far behind in this rating. This emphasizes the fact that each device has its strong and weak points.

A few relevant conclusions from this study by the FHWA (1984) are as follows:

(a) The Dynaflect, Road Raters and FWD are equipped to measure deflection basin parameters more quickly and efficiently than the static and automated beam devices;

TABLE 1.3 - Ranking of candidate devices by evaluation parameters (from Bush, 1980)

	Benkelman beam	Dynaflect	FWD	Model 400 road rater	Model 510 road rater	Model 2008 road rater
Optional characteristics	6	1	5	3	4	2
Ease	6	1	5	3	4	2
Speed	6	1	5	2	2	2
Manpower	6	1	3	3	3	1
SUBTOTAL	18	3	13	8	9	5
Costs	6	1	5	2	2	4
Accuracy	3	2	1	3	3	2
Deflection	6	2	1	2	2	5
Force	1	4	3	5	5	1
SUBTOTAL	7	6	4	7	7	6
Transportability by cargo aircraft	1	2	2	3	2	2
Depth of influence	1	5	2	6	4	3
Suitability	6	3	1	5	3	1
TOTALS	23	14	16	22	18	14

TABLE 1.4 : SUMMARY OF SELECTED DATA REPORTED ON VARIOUS NON-DESTRUCTIVE TESTING DEVICES

	No. of persons in crew	No. of test points per day	Man-hours per test day	Cost to analyze one day's data	Analysis cost per point	Average annual maintenance cost	Average daily traffic control costs
Deflection beam							
Mean	3	83	23	$ 127	$ 1,52	$ 42	$ 262
Standard deviation	0	24	1,4	$ 106	$ 0,99	–	$ 12,50
Dynaflect							
Mean	1,8	234	14	$ 88	$ 0,44	$ 2242	$ 408
Standard deviation	0,5	130	5	$ 67	$ 0,21	$ 1950	$ 229
Falling weight deflectometer							
Mean	1,9	169	15,5	$ 262,5	$ 1,41	$ 3250	$ 363
Standard deviation	0,2	65	0,9	$ 237,5	$ 1,09	$ 1750	$ 274
Road rater (all models)							
Mean	1,3	292	10	$ 362	$ 1,69	$ 2075	$ 176
Standard deviation	0,4	86	2,5	$ 178	$ 1,17	$ 2104	$ 43
Travelling deflectometer /deflectograph							
Mean	2	2667	18,7	$ 278	$ 0,10	$ 3312	$ 600
Standard deviation	0	656	3,8	$ 111	$ 0,02	$ 312	–

TABLE 1.5: COMBINED RATINGS REPORTED IN LITERATURE (LARGER NUMBERS INDICATE BETTER RATING)

Rating Criteria	Dynaflect	RR 400B	RR2000	FWD
Cost per lane mile	9.5	10.0	9.2	7.5
Operator training	10.0	10.0	10.0	10.0
Speed of operation (1)	7.5	10.0	10.0	10.0
Traffic interference (1)	7.5	10.0	10.0	10.0
Ease of data collection (2)	10.0	10.0	10.0	10.0
Ease of calibration (3)	2.0	5.0	5.0	10.0
Equipment versatility (4)	0	10.0	10.0	10.0
Actual load capability (5)	1.0	2.9	5.5	10.0
Design compatibility	10.0	10.0	10.0	10.0
Combined rating	57.5	77.9	79.7	87.5

(1) Based on the theoretical test program.
(2) Assumes the recommended options.
(3) Based on the time required for calibration.
(4) Based on ability to vary the applied load.
(5) Based on ability to produce a 10,000 lb. dual wheel load.

(b) The automated beam device, FWD and Road Rater model 2008 can develop loads at, or near, normal design loads;

(c) Load as well as deflection can easily be measured. The Road Raters and FWD are equipped to measure load as well as deflection;

(d) Devices capable of producing several load levels up to, or near, design loads can be used to determine the stress sensitivity of pavement systems, and

(e) Steady state dynamic devices, which use a relatively heavy static pre-load, change the stress state in the pavement before the testing.

The slow-moving wheel devices like the Benkelman beam and deflectographs may be seen as representing the first generation of measuring devices; the vibratory and falling weight devices are more recent developments. Of prime concern is the fact that the slow-moving devices are normally associated with a single measuring point, whereas the later generation measuring devices are increasingly able to describe the deflection basin. This is normally achieved with the equally spaced measuring points away from the point of loading. The development of the RSD (Freeme et al. 1981) and modification of the standard Benkelman beam and Lacroix deflectograph (Anderson, 1977), which can produce a "continuous" plot of the deflection basin, overcome this disadvantage. In general it can be stated that the more accurate the measuring device can measure the deflection basin, the better it is.

The curviameter can strictly be called a fast rolling wheel technique. The Technical Committee Report on flexible roads of the XVII World Road Congress (Permanent International Association of Road Congresses, 1983) described it as follows; "The Curviameter carries a velocity-sensitive transducer on an endless moving chain that places and replaces the transducer in advance of the loaded dual wheels of a lorry moving at 20 km/h. Although the machine gives an increase in route-capacity of rolling wheel techniques the derived deflections and curvatures are of limited accuracy."

The slow-moving wheel devices generally measure deflection with a standard axle load. Depending on the measuring technique of the wheel moving to or from the measuring point, deflections can vary. Although a lot can be said for these techniques and devices simulating the actual moving wheel loads (more accurately), it must be remembered that the simulation is normally much slower than the real situation and effects like plastic deformation come into play. In this regard Whitcomb (1982) states that the input from the static and vibratory devices commonly used bears little resemblance to the input from an actual vehicle. Molenaar and Koole (1982) mention that the low force levels of the light vibrators on predicted pavement behaviour are a cause for concern. This concern is not only with respect to possible non-linear behaviour of the pavement, but also with respect to errors in measurement, particularly measurements taken at the extremes of the deflection basin where deflections measured and normal variations of instruments are of the same magnitude. The heavy vibrators described by the Federal Aviation Administration (1979) have adequate loading force, but are mainly used to evaluate airfield pavements.

The FWD meets all requirements for reliability, reproduction, accuracy, simulation of moving wheel loads and measuring the whole deflection basin and it does not alter the conditions of the pavement before loading (Whitcomb, 1982). The FWD can vary the force from 40 to 125 kN and will represent any loading condition on a pavement or airfield realistically (Claessen and Ditmarsch, 1977 and Koole, 1979). Ullidtz (1982) concludes that the FWD simulated the influence of a heavy fast-moving wheel load on the maximum values of the deflections, stresses and strains in the pavement structure.

In the study by the University of Dundee (1980) it is shown that when equipment measuring deflection with dual wheel loads is used, (for example the Benkelman beam), the maximum deflection is not located at the centroid of the loading area. This is illustrated by the results of a typical analysis in Figure 1.11. Anderson (1977) also recognizes the importance of this phenomenon and noted that it might lead to wrong conclusions, particularly for relatively weak (thin) pavements. He suggested that field performance should be calibrated with analysed pavements before being adopted in the design phase. The former study recommended the use of a single wheel device and proved that deflections measured at the extremes of the loaded area did not significantly vary from deflections measured inside the loaded area. Dehlen (1962b) also recognized this phenomenon of δ_o (maximum) and R minimum being situated under the loaded area. The transverse position also lead to the more severe curvature parameters measured of the deflection basin. This is illustrated in Figure 1.12. Dehlen (1962b) states; "The longitudinal elongation observed in many chicken net crack patterns is, as has been pointed out by others, another indication that the factors giving rise to cracking are most severe in the transverse direction."

The Technical Committee Report of the XVII World Road Congress (Permanent International Association of Road Congresses, 1983) states that interest in developing analytical design methods is linked more to stationary test techniques than to the rolling wheel techniques. The light vibrators are obviously ideal for standard field work as Tables 1.3, 1.4 and 1.5 indicate. In spite of the advantages of using the FWD mentioned earlier, Molenaar and Koole (1982) comment that, according to conclusions of a study group of the Dutch Study Centre for Road Construction, the

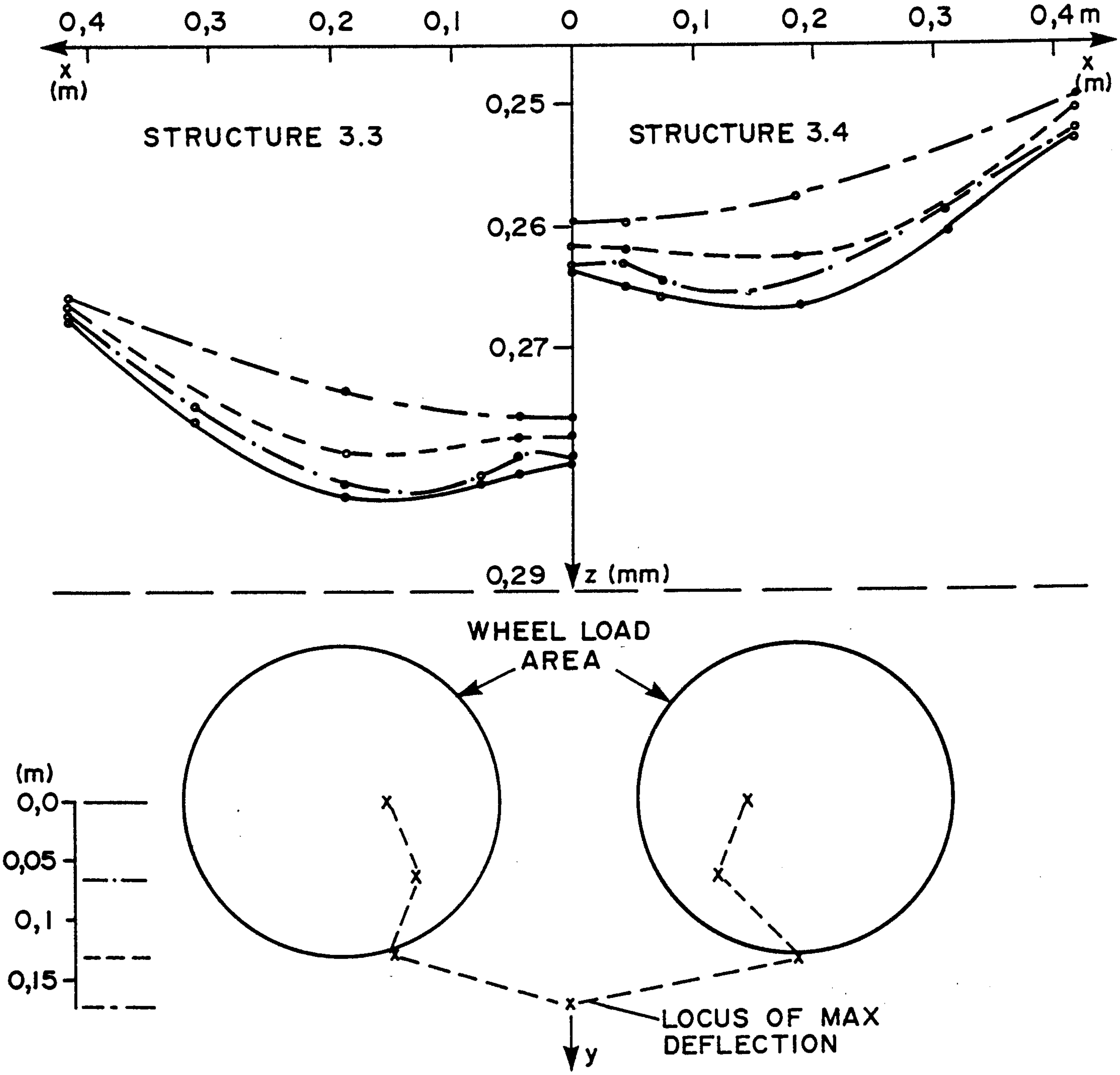


FIGURE I.11
TRANSVERSE DEFLECTION PROFILES
(University of Dundee, 1980)

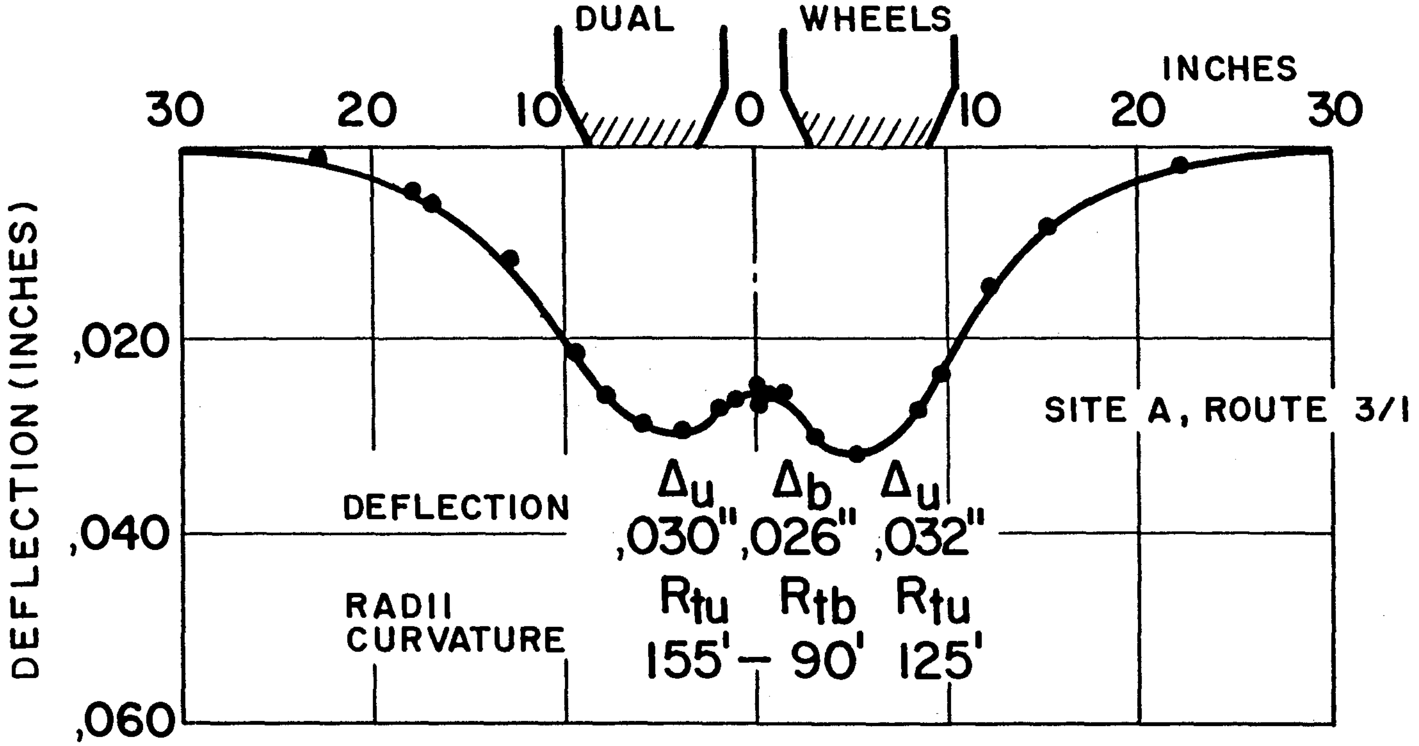


FIGURE 1.12
TYPICAL EXAMPLE OF TRANSVERSE DEFLECTION PATTERN BENEATH DUAL WHEELS (Dehlen, 1962 b)

Lacroix deflectograph is more suitable for routine evaluations than the FWD. The RSD developed in South Africa overcomes the normal problems experienced with a standard Benkelman beam concerning poor sampling frequency and testing in adverse weather conditions. Owing to the limited length of any such beam device the problem of the beam supports being inside the deflection basin will not be eliminated completely. This is particularly true of stiffer pavements where the deflection basin is broad. The Technical Committee Report on flexible Roads of the XVII World Road Congress (Permanent International Association of Road Congresses, 1983) also concludes that; "Precision of measurement can be difficult when testing stiff pavements containing cemented layers and cracking in those layers can reduce the significance of measurements of the deflected shape."

3.3 Correlations between Devices

There are numerous references in the literature to the correlation of deflections obtained with the different devices. Generally the newer generation measuring devices are correlated with the older generation devices like the Benkelman beam. Whitcomb (1982) records relationships between the Road rater, Dynaflect and Benkelman beam or travelling deflectometer. Hoyinck, et al. (1982) even correlate Lacroix deflectograph and Benkelman beam deflections measured with different techniques.

The reason for the correlations is that one type of deflection measurement may be translated into another, so that empirical or established interpretation graphs can be used without the measurement having to be repeated in the prescribed manner. In addition to the reason noted above, Whitcomb (1982) notes that the newer generation vibrating measuring devices offer significant advantages of ease and speed of operation over the Benkelman beam and California deflectometer.

Moore et al. (1978) make the following statement on the quality of such correlations: "All of the steady state dynamic deflection devices can be expected to correlate reasonably well with static deflection measurements. Many evaluation procedures employ these dynamic deflection devices for estimating the anticipated useful life (or load-carrying capacity) of pavements based upon correlation of the measurements with static deflection measurements".

It is obvious though, from the discussion on deflection parameters (see 1.3.2) that the correlation of a single deflection point, like a Benkelman beam deflection, with a highly complicated deflection basin description will not necessarily be a good one. For this reason Whitcomb (1982) cautions about the use of applying published correlations without some knowledge of the degree to which the two variables have been correlated. It is more desirable to avoid the need to make the correlations at all.

The following is quoted from the FHWA study (1984): On the correlations between NDT deflection devices "In general, a different correlation should be developed for each major pavement type and for different pavement thicknesses within particular types of pavement because the correlation is not unique, ...". Regarding the interchangeability of data the following is also quoted: "The source, testing procedure and equipment configuration used in developing the data must be fully understood before data collected by another agency can be used."

4 CONCLUSIONS AND RECOMMENDATIONS

Ideally deflection basin measuring equipment should:

(a) realistically simulate moving traffic loads in terms of magnitude of load, shape and equivalent time of loading

(b) accurately measure the whole deflection basin with high levels of reproducibility

(c) be simple to operate, so that it is possible to use it with competence in the field; but it should also be applicable to research

(d) be capable of attaining high levels of productivity, which must reduce the cost of testing.

Considering these prerequisites, it seems that the FWD is the most appropriate of the devices discussed. As far as cost and ease of operation in particular are concerned, the vibratory devices such as Dynaflect and Road Raters are also considered appropriate. The main advantage of these newer generation deflection measuring devices lies in the fact that they normally measure at least four points on the deflection basin. The modified RSD or Benkelman beam and a modernized Lacroix deflectograph are able to monitor the deflection basin at frequent intervals in spite of there being only one measuring point. The Lacroix deflectograph is used by the road authorities in South Africa on a network basis with a proven level of efficiency. A vast amount of information is available on various pavement structures from HVS testing and on full structural lives measured with the RSD and MDD on the same location. The information gained from these devices may be used to determine suitable deflection basin parameters accurately or to perform realistic correlation studies between these parameters. It is possible to use deflection basin measurements obtained from a modernized Lacroix deflectograph for more detailed analysis. To date only the maximum deflection (δ_o) has been used to distinguish between various uniform sections of road statistically. By the use of deflection basin parameters related to specific pavement structure type and state a higher level of engineering interpretation and effective service can be provided for the purpose of pavement management, rehabilitation and overlay design. It is suggested though, that tests be done with the FWD and vibratory deflection basin measuring devices in South Africa in view of their advantages mentioned above. These devices will have to be correlated with the above-mentioned data bank on deflection measurements from HVS sites in order to ensure uniform standards of deflection interpretation.

The parameters of the deflection basin must -

(a) represent the full characteristics of the whole deflection basin (not only maximum deflection (δ_o), but rather a combination of parameters like SCI and BCI covering the whole deflection basin);

(b) be simple to calculate and interpret, and

(c) be able to relate to the structural characteristics of the full depth of pavement structures.

From the survey it is obvious that in the past at least two deflection points on the deflection basin are normally needed in an analytical procedure. One such point is the point of maximum deflection (δ_o). The other point should preferably be varied in distance relative to the centroid of the loaded area (δ_r) in accordance with prescribed requirements. Examples of such parameters are Q and ST, which still require other points measured on the deflection basin in order to select a value of δ_r. Another alternative is to use fixed values of radius for deflection points, for example δ_o, δ_{305}, δ_{610}, etc. It is suggested that the parameters that use these deflection values, for example SCI, ST, BDI, BCI, etc., be investigated in view of the guidelines described above and the equipment available in South Africa. A further suggestion, which will be discussed in more detail in a later section, is that these selected parameters be related to the pavement structure classification used in TRH4 (NITRR, 1985a) and to pavement performance models. A parameter like SCI will then be determined differently for a granular base than, for example, from for a cemented base pavement, reflecting its different deflection basin characteristics and structural performance.

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CHAPTER 2

MEASUREMENT AND DATA PROSESSING OF DEFLECTION BASINS IN SOUTH AFRICA

CHAPTER 2: CONTENTS

	PAGE
1. INTRODUCTION	2.2
2. THE MEASURING EQUIPMENT AND PROCEDURE	2.2
2.1 Measuring equipment	2.2
2.2 The procedure	2.3
2.2.1 Site selection and preparation	2.3
2.2.2 Measuring procedure	2.6
3. EXISTING PROCEDURES TO EVALUATE DEFLECTION MEASUREMENTS	2.9
3.1 Pre-automation data evaluation (Rebound procedure)	2.9
3.2 Automated data evaluation (WASHO procedure)	2.14
4. THE NEED FOR CURVE FITTING PROCEDURES	2.17
5. PROPOSED DEFLECTION BASIN MEASURING EVALUATION PROCEDURE	2.19
5.1 Data manipulation and preparation	2.19
5.2 Higher level deflection basin data manipulation	2.20
6. CONCLUSIONS AND RECOMMENDATIONS	2.25

1. INTRODUCTION

In South Africa the use of the full deflection basin was generally neglected in the analysis of deflection basin parameters. In the past the analyses were mostly concentrated on the small area of the deflection basin in the vicinity of maximum deflection. In the program of accelerated testing with the Heavy Vehicle Simulator (HVS) fleet in South Africa, the whole deflection basin is measured on the surface and in depth of the pavements. The same limitation of data analysis was concentrated on the small area of the deflection basin near the vicinity of the maximum deflection for these depth deflection basins measured. In this chapter a brief description is given of how the surface deflection basin is measured in South Africa by means of the automated Benkelman beam or also called the road surface deflectometer (RSD). The procedure and equipment to measure the deflection basin at various levels in the depth of a pavement by means of a multi-depth deflectometer (MDD) are also briefly discussed.

The present data analysis procedure is discussed. Based on this and a discussion of the whole deflection basin and related parameters, the author suggests changes to this data analysis procedure. Specific reference is made of the need for a better description of the full deflection basin . Curve fitting models are discussed briefly and recommendations are made for practical use with the road surface deflectometer (RSD) and the deflectograph..

2. THE MEASURING EQUIPMENT AND PROCEDURE

2.1 Measuring Equipment

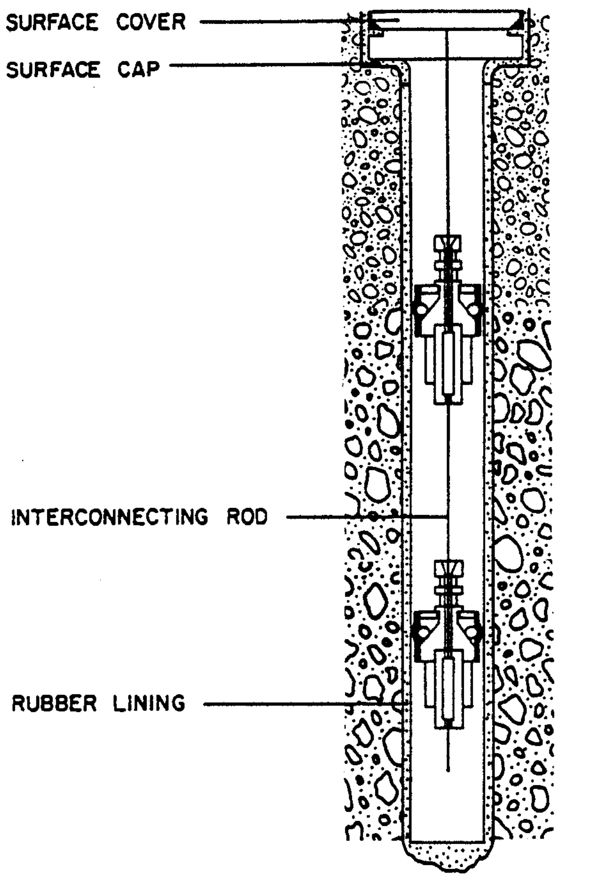
HVS testing is done by repeated application of a chosen wheel load to the road structure. Several sophisticated instruments are used to monitor the response of the road. In the field, a micro-processor system is used to record the measured data on a magnetic disc. The data on the magnetic disc is then transported to the central laboratory in Pretoria for processing by the main computers (Freeme, et al., 1981). During an HVS test the measurements of the surface deflection and curvature of the pavement with an automated Benkelman Beam, also called the Road Surface Deflectometer (RSD) (Basson, 1985), are collected as data. Elastic deflection and permanent deformation measurements are taken at different depths within the pavement using the multi-depth deflectometer (MDD). (Basson, et al. 1980.)

The MDD is a device that can simultaneously measure the vertical deflections and permanent deformations of up to six points on a vertical line in any pavement. The methods of site preparation and illustration of the MDD is shown in Figure 2.1. The RSD is an electronically instrumented deflection beam which can measure dynamic resilient deformations to an accuracy of ± 0,01 mm. (Shackel, 1980.) This modified Benkelman Beam (the RSD) and the use of the RSD and MDD plus other related instruments are illustrated in Figure 2.2. The modules of the MDD and the measuring point of the RSD make use of a linear variable differential transformer (LVDT) to measure pavement deflections. This discussion serves only as a brief description of the equipment used to measure deflections in depth and on the surface of a pavement structure being tested with the HVS.

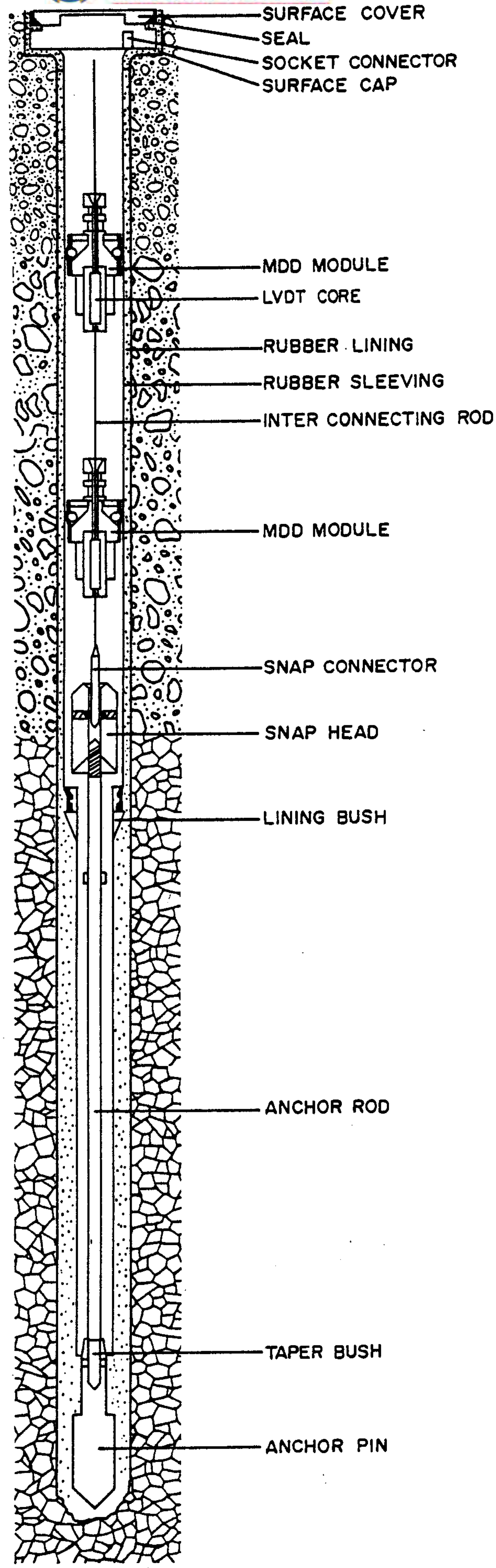
2.2 The procedure

2.2.1 Site selection and preparation.

A typical HVS test section is selected after a deflectograph survey of the road length was done. Uniform sections of the road are then identified which differ significantly statistically. A decision is made whether to select a section that represents for example the 85th percentile, 15th percentile or average of the road in terms of the deflectograph survey. A detail survey of each meter of normally a 100 meter section is



SURFACE REFERENCE



BOTTOM REFERENCE

FIGURE 2.1

METHODS OF SITE PREPARATION OF THE MULTI-DEPTH-DEFLECTOMETER

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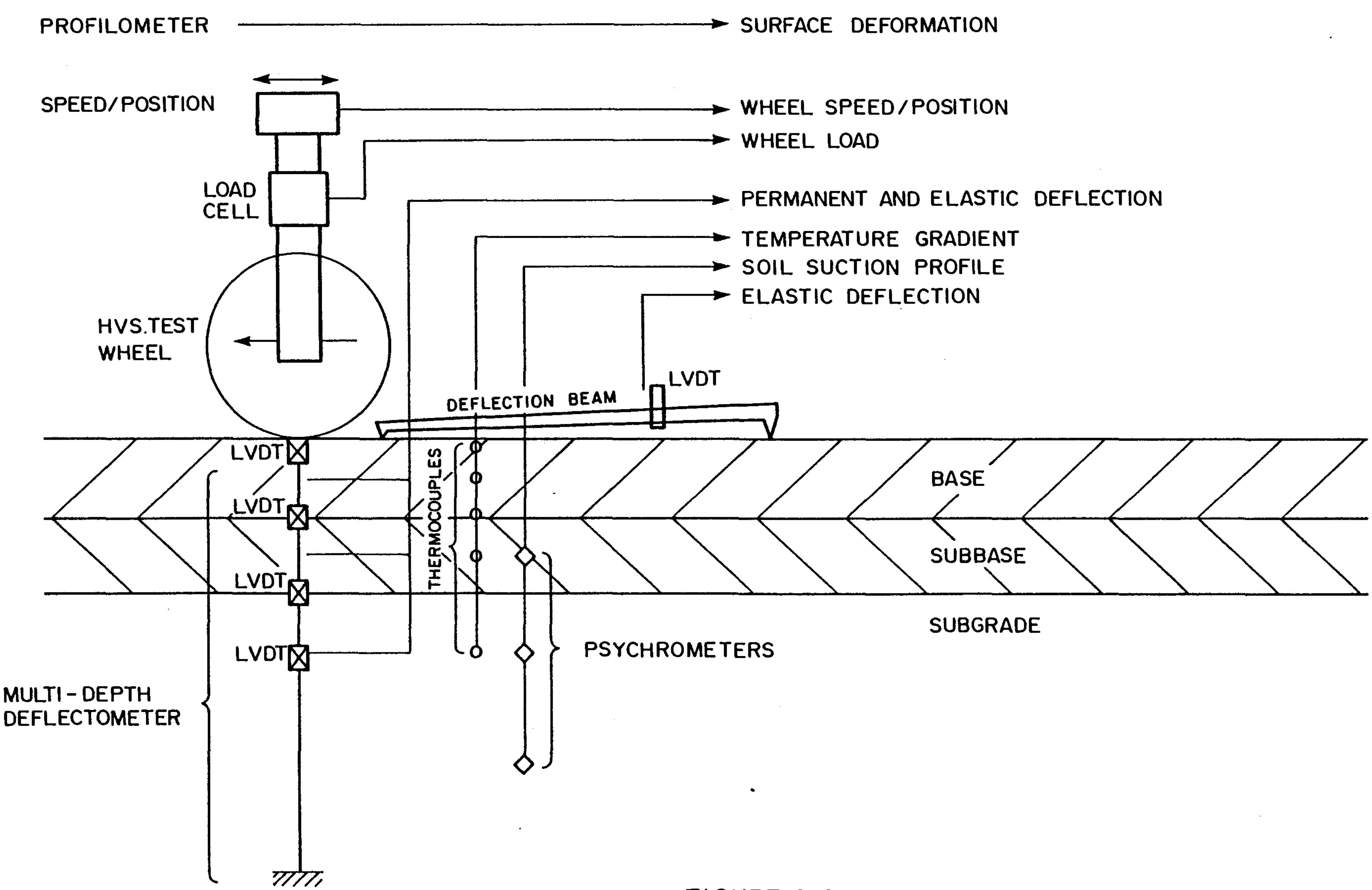


FIGURE 2.2

THE HVS SECTION INSTRUMENTATION

then carried out with the standard Benkelman Beam. Based on these results a 8 meter section of road is selected which is uniform in terms of its maximum deflection measurements.

Other factors such as road safety, gradient etc. are also considered in the selection of such an HVS test section. (Shackel, 1980.) In some cases, particularly short experimental sections, the selection of a site may follow a different procedure, but the aim is normally to have at least an HVS test section that is uniform over its 8 meter length in terms of Benkelman Beam deflections.

The instrumentation and marking of such a selected test section is as shown in Figure 2.3. As can be seen, normally at least two MDD holes are installed while the RSD measuring points are marked on the surface in order to represent the whole section and enhance repeatability.

2.2.2 Measuring procedure

The dual wheel with the specified tyre pressure and load is moved ± 3 meters away from the measuring point. The deflection produced by the load approaching the measuring point of either the RSD or MDD is recorded on a chart as a continuous trace, while switches, at measured distances along the road, record the passage of the load passing them. A typical trace is shown in Figure 2.4. The square waves of the switches reflect the variance in speed at which the wheel load approaches the measuring point in order to facilitate the correction to relate distance to deflection accurately. The loaded wheel moves past the measuring point for a distance of about 1,2 meter whereafter pulses simulating measurements are generated automatically to complete the standard set of 256 measurements of a deflection basin. Each measured point on this set is a standard distance apart, usually 22,07 mm, depending on the switch characteristics. Usually measurement sets with the MDD are taken only on the centre line and with the RSD in line with the measurement point. Off-centre measurements can be taken with the MDD. At

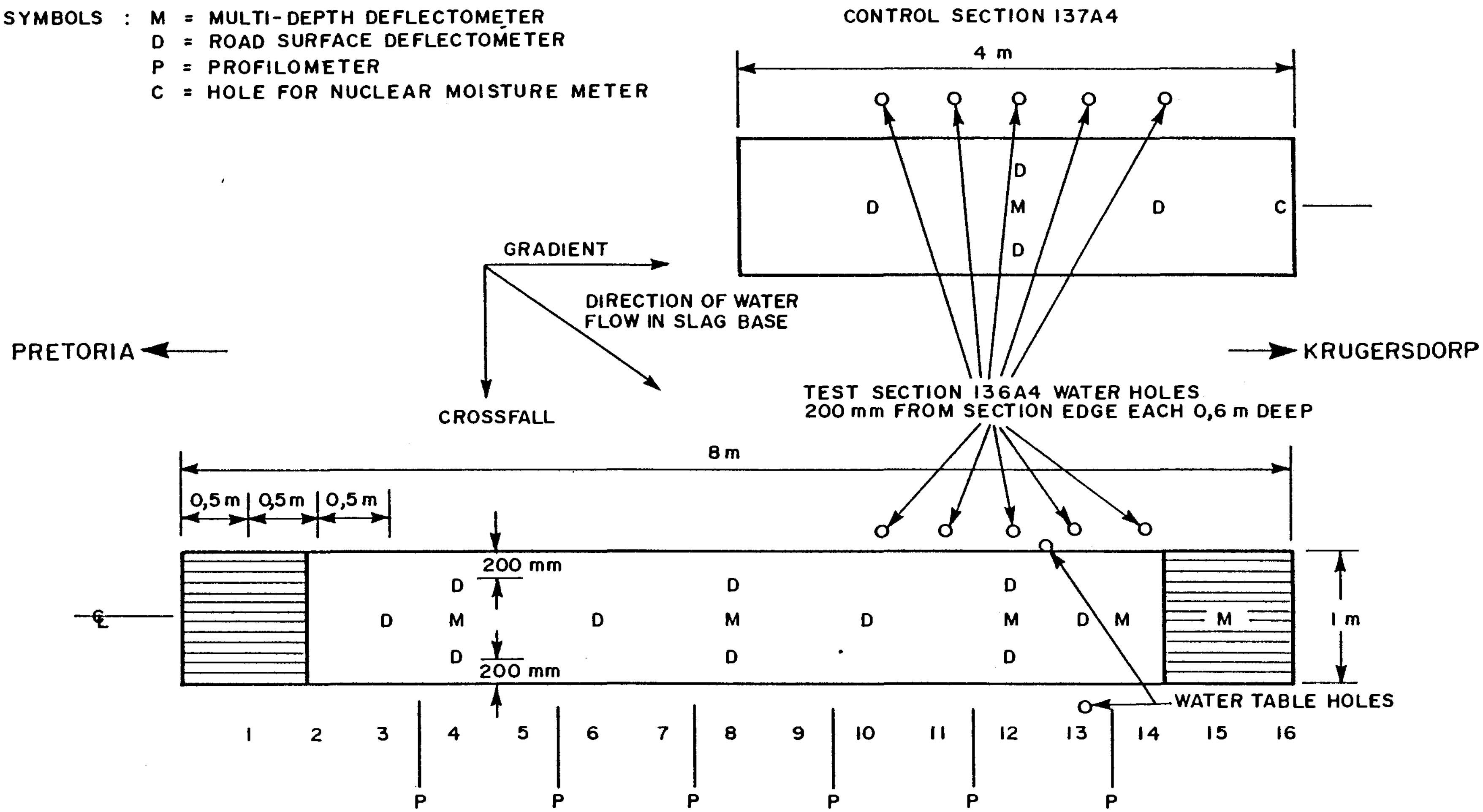


FIGURE 2.3

TYPICAL LAYOUT OF TEST AND CONTROL SECTION

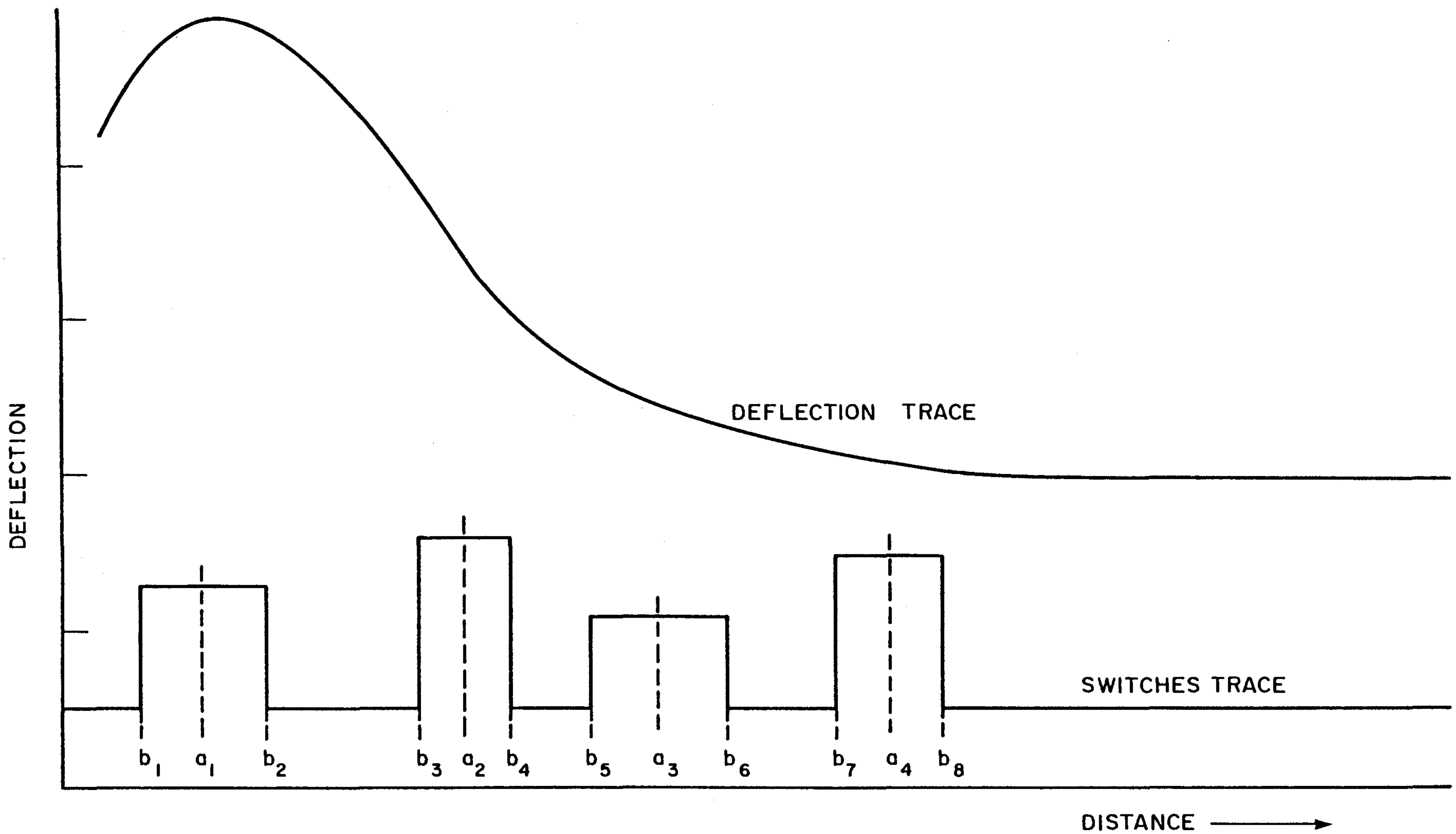


FIGURE 2.4

TYPICAL RECORDED CHART FROM RSD MEASUREMENT SHOWING DEFLECTION AND SQUARE WAVE PRODUCED least 2 measurement sets are taken as control at each measuring point of the RSD and MDD at a speciffic time during the testing. Various wheel load and tyre pressure combinations are used during measurements.

It is important to note that the method of measurement of the standard Benkelman Beam and that of the RSD and MDD under HVS conditions differ. The RSD and MDD measurements use the WASHO procedure (Monismith, 1979) while with the standard Benkelman Beam the rebound procedure is used. In the WASHO procedure the load approaches the end of the probe or measuring point and the deflection is observed. In the rebound procedure the wheel moves away from the measuring point and rebound of the pavement is measured. Rebound deflection is about two times the deflection measured with the WASHO procedure. This may vary with the type of pavement measured. This is probably due to plastic deformation.

3. EXISTING PROCEDURES TO EVALUATE DEFLECTION MEASUREMENTS

3.1 Pre-automation data evaluation (rebound procedure)

The Benkelman Beam was used very effectively by Dehlen (1961) to measure deflection-distance curves manually. This was done by measuring the deflection basin manually at 3 inch (75 mm) intervals as the wheel load approached the measuring point. Smooth curves were drawn through the plotted points. Particular care was taken over the central 2 feet (610 mm), and any irregular curve rejected. This is illustrated in Figure 2.5. The maximum deflection was obtained directly from the observations. The radius of curvature at the point of maximum deflection was obtained by determining the circle which is the best fit to the curve over the central 6 or 9 inches (150 to 225 mm). The radius of curvature of the road surface was then computed from the radius of the circle of best fit from the formula;

$$R = r.\frac{v}{h^2}$$

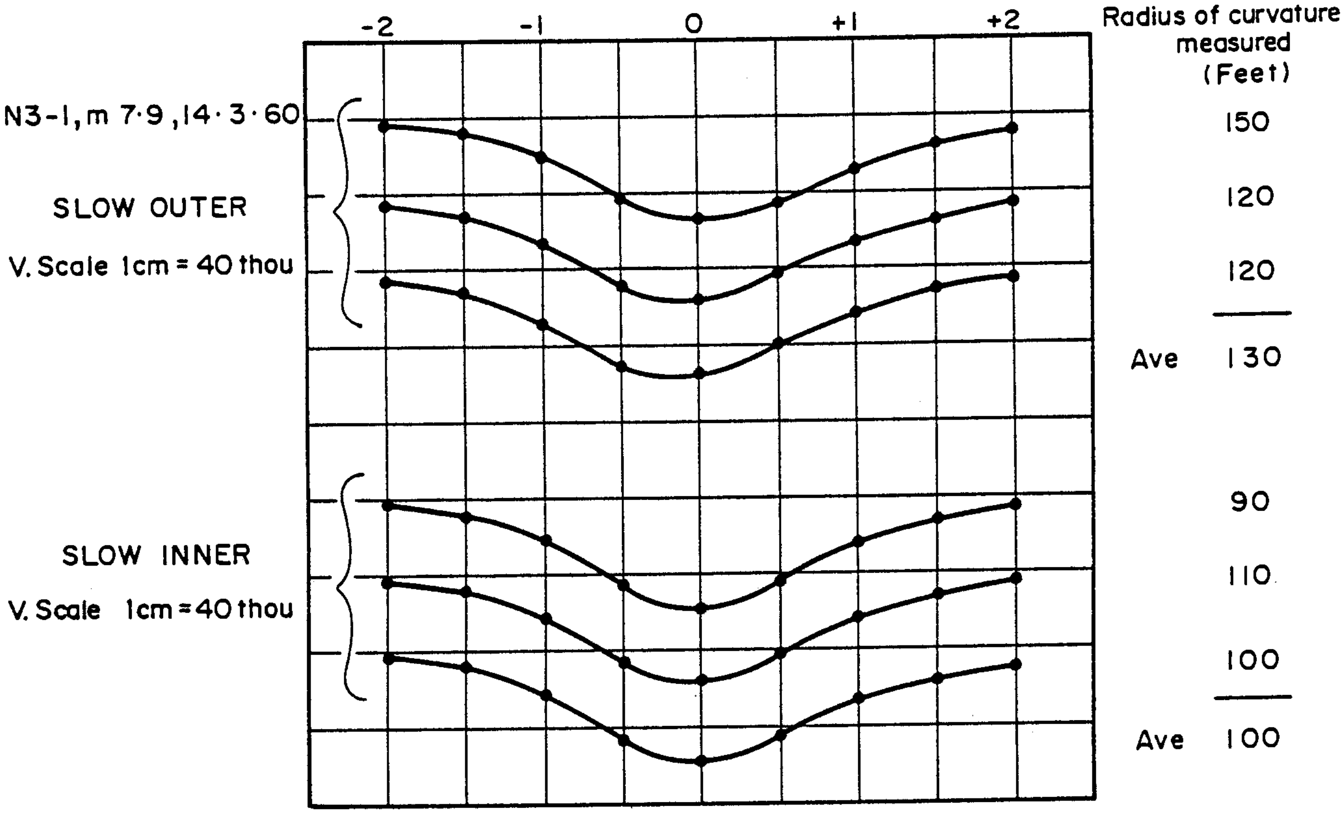


FIGURE 2.5

EXAMPLE OF THE PLOTTING OF DEFLECTION OBSERVATIONS AND MEASUREMENT OF RADIUS OF CURVATURE where R = radius of curvature of road surface,

r = radius of plotted circle, in the same units as R,

v = vertical scale of plot expressed as a dimensionless ratio,

h = horizontal scale of plot expressed as a dimensionless ratio

Dehlen (1961) noted that a circle fitting the deflected surface in the field became an ellipse when plotted to different horizontal and vertical scales. Thus ideally, determination of radius of curvature from the deflection plot should be made by fitting ellipses. Dehlen (1961) estimated that the error introduced by fitting circles however, was not likely to exceed 5 per cent. It is also observed by Dehlen (1961) that care and experience is necessary for the accurate determination of the radius of curvature from the observations.

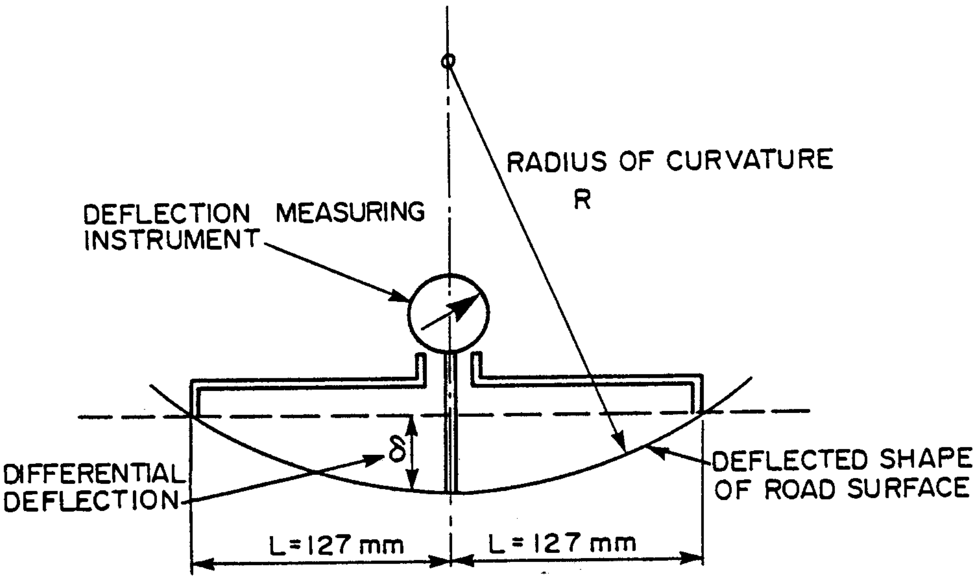
The Dehlen curvature meter developed by Dehlen (1962a), enabled measurement of the curvature directly, as illustrated in Figure 2.6. The relation between curvature and differential deflection may be deduced by simple geometry by fitting an appropriate curve to the three points on the road surface defined by the instrument. Dehlen (1962a) indicated that using a parabola, the radius of curvature is given by

$$R = \frac{L^2}{2\delta}$$

where δ is the differential deflection

L is the distance between the deflection gauge and each support

Dehlen (1962b) observed that a study of numerous deflection patterns obtained over a period of time indicated that in the vicinity of the points of maximum deflection the curves are typically of a sine form. It has also been noted that points of inflection (points P in Figure 2.7, where the curve changes from concave to convex) occur fairly consistently at distances (S) of 6 inches (150 mm) on either side of the point of maximum deflection. The



NOT TO SCALE

FIGURE 2.6

DIAGRAMMATIC SKETCH SHOWING THE PRINCIPLE OF THE CURVATURE METER

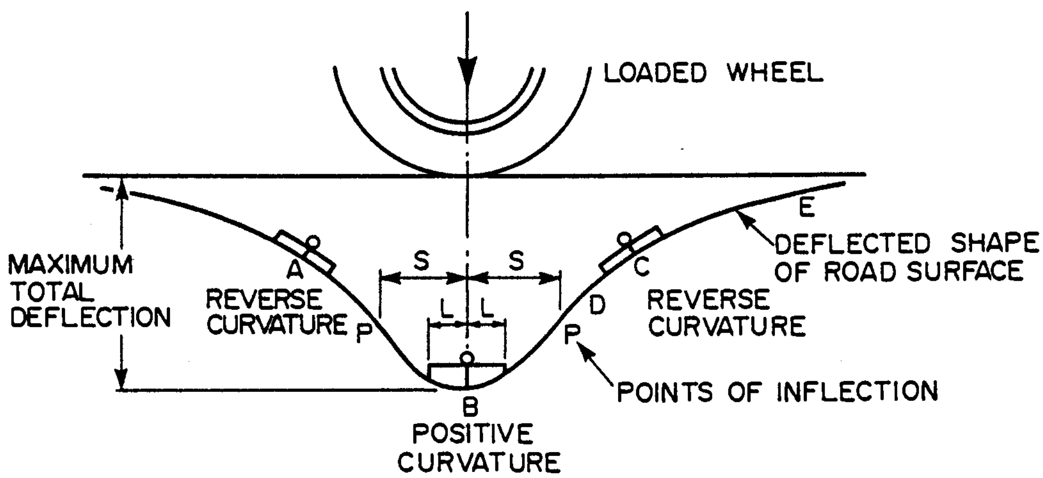


FIGURE 2.7

TYPICAL PATTERN OF DEFLECTION OF A ROAD SURFACE BENEATH A LOADED WHEEL relation between radius of curvature (R) and differential deflection (δ) in the case of a sine curve is

$R = \frac{L^2}{F\delta}$ where F is a factor which varies between 2 and 2,47.

With F = 2,3 and L = 5 inches (127 mm) the formula is reduced to:
$R = \frac{7}{\delta}$

δ is measured in mm; R in meter.

3.2 Automated data evaluation (WASHO procedure)

A program BENKI was written by Szendrei (1974) and later updated as BENK5 (Szendrei, 1975(a)) to evaluate the automated Benkelman Beam (RSD) field data on computer. These programs were later updated again by (Szendrei, 1975(b)) to accommodate the processing of the MDD deflection data too.

In these programmes the deflection tracer, as supplied from the field measurements, is defined by 11 points of the original 256 points measured, along the length of the trace. The points on the chart are converted to true road distances and road deflections. The result of these operations is to produce 11 points on a plot of road deflections versus road distances. The 11 points lie on a plot as shown in Figure 2.8 which still follow the basic shape, although somewhat distorted.

Szendrei (1975(a)) noted that the top part of the curve down to some level PQ (Figure 2.8), which is about 30 per cent of the maximum, approaches a probability curve closely. This is represented by the equation;

$y = A \exp(-kx^2)$ in which A is the maximum amplitude occurring at $x = 0$ and k is an attenuation constant which determines the width of the curve. Below PQ, (see Figure 2.8) a linear interpolation method was followed. The values of k and A are determined by

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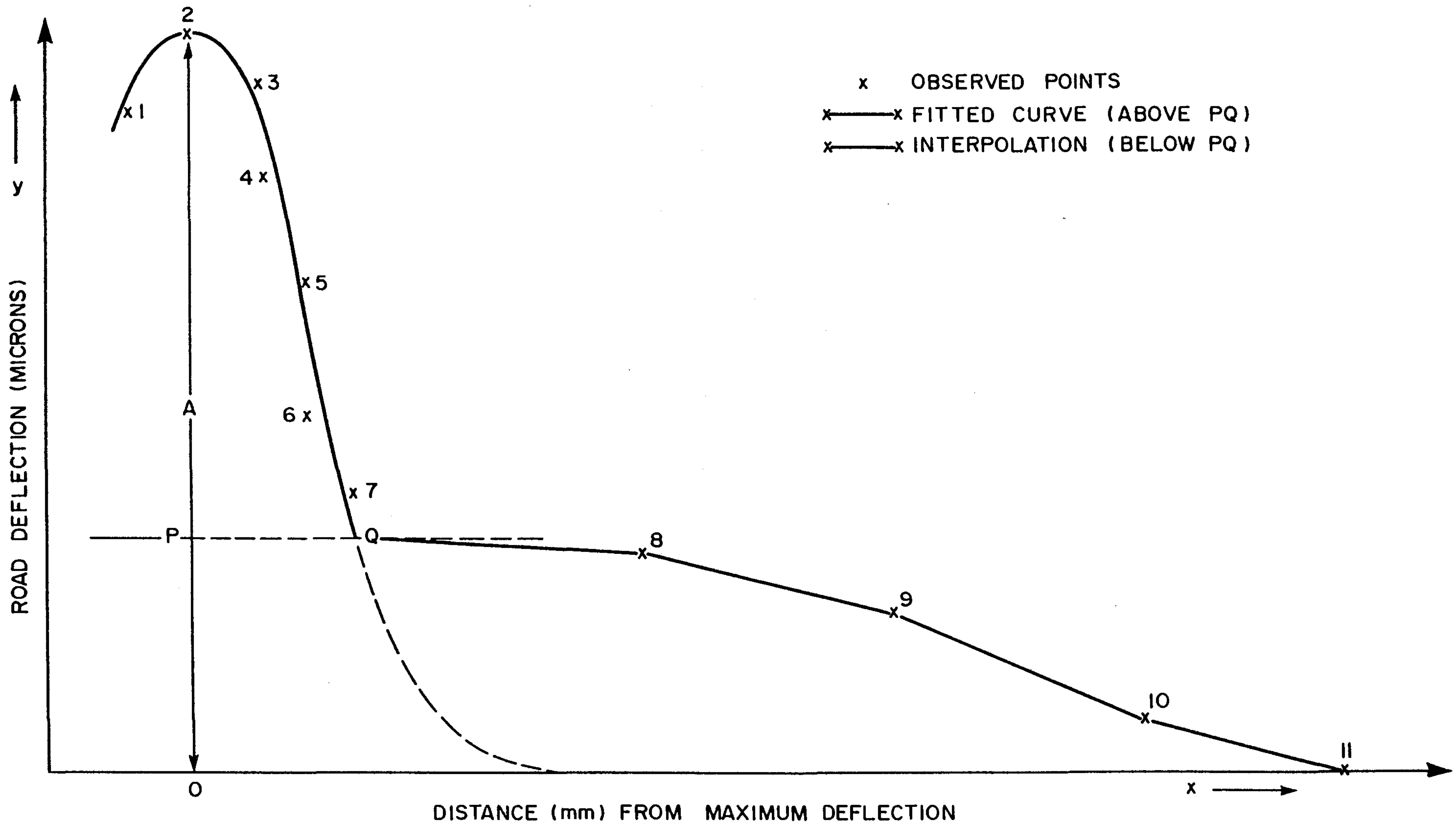


FIGURE 2.8

FITTING OF EXPONENTAL CURVE TO MEASURED POINTS OF THE POSITIVE using the y co-ordinates of the measuring points from point 2 down to level PQ.

The standard error e of the fitted curve can then be estimated from;

$$e = \sqrt{\Sigma d_i^2/n}$$

in which d_i = deviation of observed point from value calculated from the fitted curve (y_i - A exp $(-kx_i^2)$) and Σ indicates the summation for all n points observed above level PQ.

The main aim of these programs are to determine the radius of curvature at the point of maximum deflection. By using the general formula of radius of curvature (Szendrei, 1975(a)) at any point along a curve defined by y = f(x), radius of curvature is given by;

$$R = \left| \frac{1 + \left(\frac{dy}{dx}\right)^2}{\frac{d^2y}{dx^2}} \right|^{3/2}$$

For the curve fitted, Szendrei (1975(a)) derives a radius of curvature at any point along the curve:

$$R = \frac{(1 + 4k^2x^2y^2)^{3/2}}{2ky\ (2kx^2-1)}$$

The minimum radius of curvature, occurs at the point of maximum deflection of the curve. Substituting the values of y = A and x = 0 into this derived equation, Szendrei (1975(a)) derives;

$$R = \frac{0,5}{kA}$$

Using Dehlen's radius of curvature formula, R = $\frac{7}{\delta}$ when δ is measured in mm and L = 127 mm, Szendrei (1975(a)) made a more direct comparison with the Dehlen radius of curvature. This is done by calculating deflection δ_{127} at 127 mm from the deflection basin. Assuming that the curve is symmetrical about the vertical and passing through the maximum, the values of δ_{127} will be the same on both sides. Differential deflection δ, as described by Dehlen and corresponding to the measurement of the Dehlen curvature meter instrument is then expressed as follows:

$$\delta = A \left| 1 - \exp(-(127)^2 k \right|$$

To find the mean radius of curvature between x = 127 mm and x = -127 mm, where x = 0 at the maximum of the curve, a solution for the circle with origin at (o,y_o) and intersecting the exponential curve (Zendrei, 1975(a) at $x = \pm 127$ mm is closely approximated by;

$$R = \frac{8,065}{\delta} \quad \text{where } \delta \text{ is in mm and R in meter.}$$

Szendrei (1975(a)) noted that this formula gave a discrepancy of 14 per cent if compared to the Dehlen radius of curvature. This does not necessary reflect the difference in the model used to fit this part of the deflection basin. It must be remembered that the difference in measuring procedure (WASHO versus rebound), for the particular pavement type compared, will also contribute to this difference.

4. THE NEED FOR CURVE FITTING PROCEDURES

In the preceding sections it was illustrated that even with the automated Benkelman Beam or RSD, the emphasis was to measure mainly two deflection basin parameters, namely: maximum deflection (δ_o) and radius of curvature (R). Both these parameters are measured in the area of positive curvature of the deflection basin (see Figure 2.7). This is only a very small part of the whole deflection basin. The fact that these two parameters alone are reflecting only limited information available from any possible deflection basin of any particular pavement is described in full elsewhere. (Horak, 1984.)

In the development of fundamental methods of pavement evaluation by means of deflection basin measurements it is imperative that an accurate description of the deflection basin is obtained. In Table 1.1 a summary of deflection basin parameters and their respective formula are given (Horak, 1984). The level of description of the deflection basin varies from a singular point to a highly sophisticated polynomial function. It is clear though that there is a move towards incorporating parameters of the deflection basin that attempt to describe the reverse curvature (see Figure 2.7) of the deflection basin too. Horak (1984) clearly indicates that there is reason to believe that deflections measured (and their related parameters) on the outer edges of the reverse curvature of the deflection basin give a clearer indication of the structural value of the subgrade. Deflections measured nearer to the point of maximum deflection (on the positive curvature, Figure 2.7) give a better indication of the structural value of the upper layers. The structural effect of the lower layers are also reflected by these parameters and the need arises therefore to separate these effects by giving more attention to the proper description and measurement of the reverse curvature of the deflection basin.

The preceding sections indicated that the standard procedure at any HVS test section is in fact to measure the whole deflection basin very accurately with the RSD on the pavement surface and the MDD in depth of the pavement. As stated earlier the data evaluation of these curves were traditionally only focused on the related parameters of the positive curvature part. In fact the model for curve fitting used, only attempts to describe the 30 per cent part of maximum deflection while the reverse curvature part is linearly interpolated. Only 7 selected points on the positive curvature are used to do curve fitting with (see Figure 2.8) in the Szendrei model (1975(a)). This can lead to a misrepresentation although a statistically acceptable fit is achieved.

There exists a discrepancy in the interpretation of or calculation of radius of curvature (R) if the Dehlen radius of curvature and the Szendrei radius of curvature is calculated. The Dehlen radius of curvature measurement uses, the rebound method while the Szendrei radius of curvature, from RSD data, is measured with the WASHO procedure. The maximum deflection differs by a factor of two when these two procedures are compared. It is obvious that the radii of curvature will also be influenced. While the circle being fit through points at ± 127 mm and maximum deflection (through δ_o) in the Dehlen procedure of radius of curvature calculation, the Szendrei procedure only require it to intersect at ± 127 mm on the deflection basin. Both radii of curvature further do not really comply to the strict definition of radius of curvature as two or three points are intersected. Curvature at a point is the rate at which the curve is turning away from the tangent line at that point and radius of curvature is the reciprocal of curvature. (Bedford et al., 1970) For such calculations, a proper definition of the curve is needed mathematically. Factors such as plastic deformation and reverse curves within the positive curvature centre (Horak, 1984) can remove radius of curvature calculations even further away from a soundly based scientific definition and interpretation.

The standard evaluation of maximum deflection and Dehlen radius of curvature of the RSD and MDD results lacks the freedom or ability to do more sophisticated analysis of the pavement structurally. The need arises therefore to relate deflection basin parameters of the whole deflection basin to specific structural layers or zones or behaviour states.

The vast amount of RSD and MDD measurements available from all the HVS tests and quality of measurements make it imperative that better use of the whole deflection basin measurements must be made. New models and procedures should be investigated to evaluate such deflection basin data.

5. PROPOSED DEFLECTION BASIN MEASURING EVALUATION PROCEDURE

5.1 Data manipulation and preparation

In Section 3.2 it was described how RSD and MDD deflection basin data are prepared. At the stage before the modelling of the positive curvature (see Figure 2.7) is done, the data is in a standard form where the 256 measurements of deflection are correlated correctly to horizontal distance. In Figure 2.9 a graphical illustration is given of such a typical set of measurements. As can be seen the deflection has, as measured, a negative sign and the origin is not at the point of maximum deflection (δ_o). The possibility of the reference points of the RSD being inside the deflection basin and leading to incorrect deflection measurements increase significantly as the loaded wheel passed the measuring point (point of maximum deflection). Additionally there exists the danger of plastic deformation at particularly higher wheel loads, interfering with the quality of deflection measurements, as the wheel passes the measuring point and slowing down to a halt. For that reason the author decided to work only with the first part of the curve up to the point of maximum deflection. In order to make calculations easier too, the negative sign of the deflection was reversed and the origin was chosen at the point of maximum deflection. The description of the programmes and procedures to do these manipulations on computer are given in detail by Horak and Otte (1985). In some cases "spikes" occur due to interferences like wind, touching wheels, etc., on the original data set (as in Figure 2.9). This is first smoothed out by a standard programme procedure (Horak and Otte, 1985). The final data set, as prepared, is then stored on computer in a format whereby the deflection and horizontal distances are correctly related. A graphical presentation of such a data set is as shown in Figure 2.10. Such a data set of a deflection basin measurement can now be defined as edited for further data manulipulation. As will be seen in the next section such data manipulations can be done with various levels of sophistication.

5.2 Higher level deflection basin data manipulation.

The form in which a data set of a measured deflection basin is, as described in the preceeding section, ideal to calculate all the

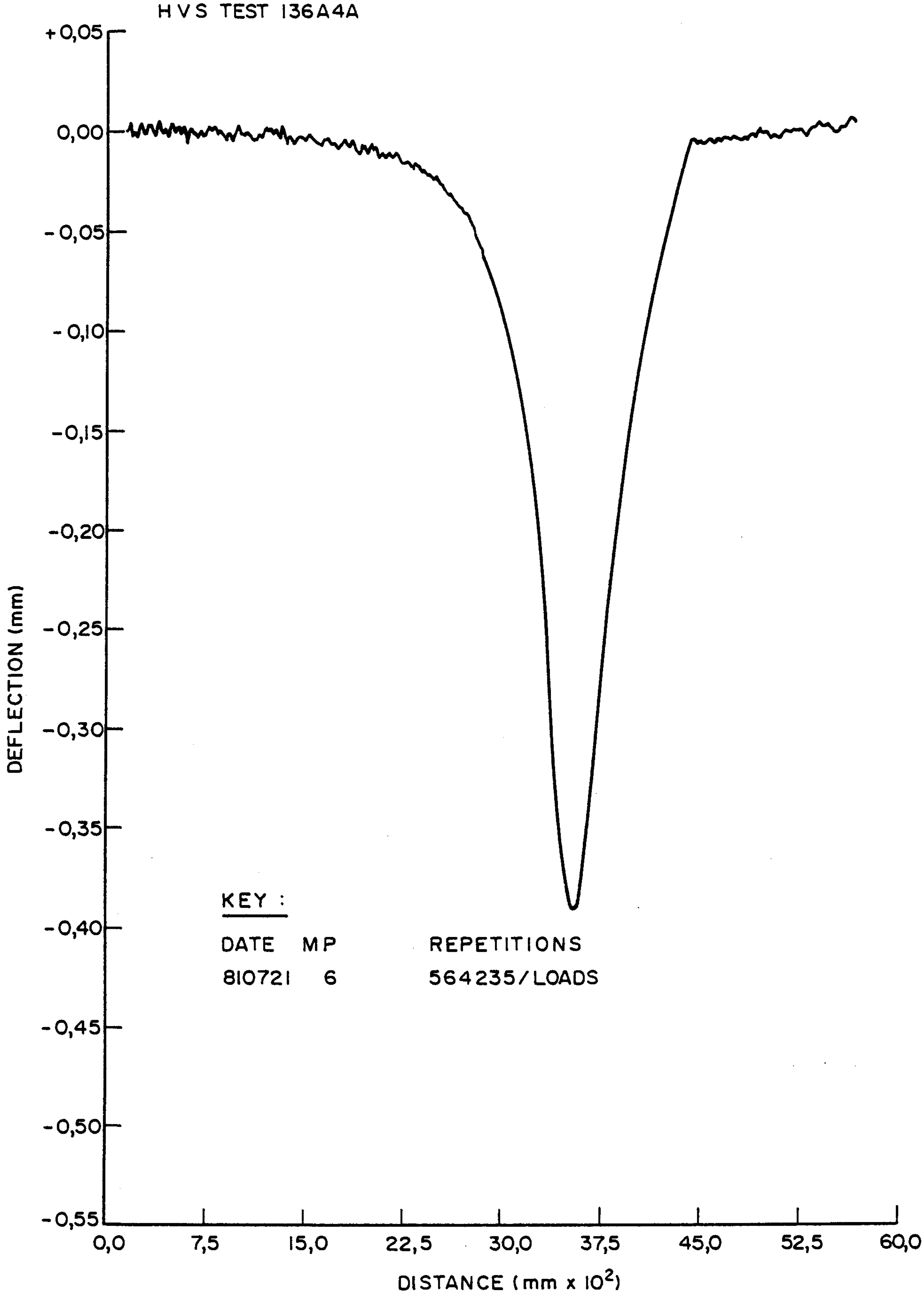


FIGURE 2.9

TYPICAL RSD RESULTS OF DEFLECTION BASIN MEASUREMENTS

940-4-4966/11 BS

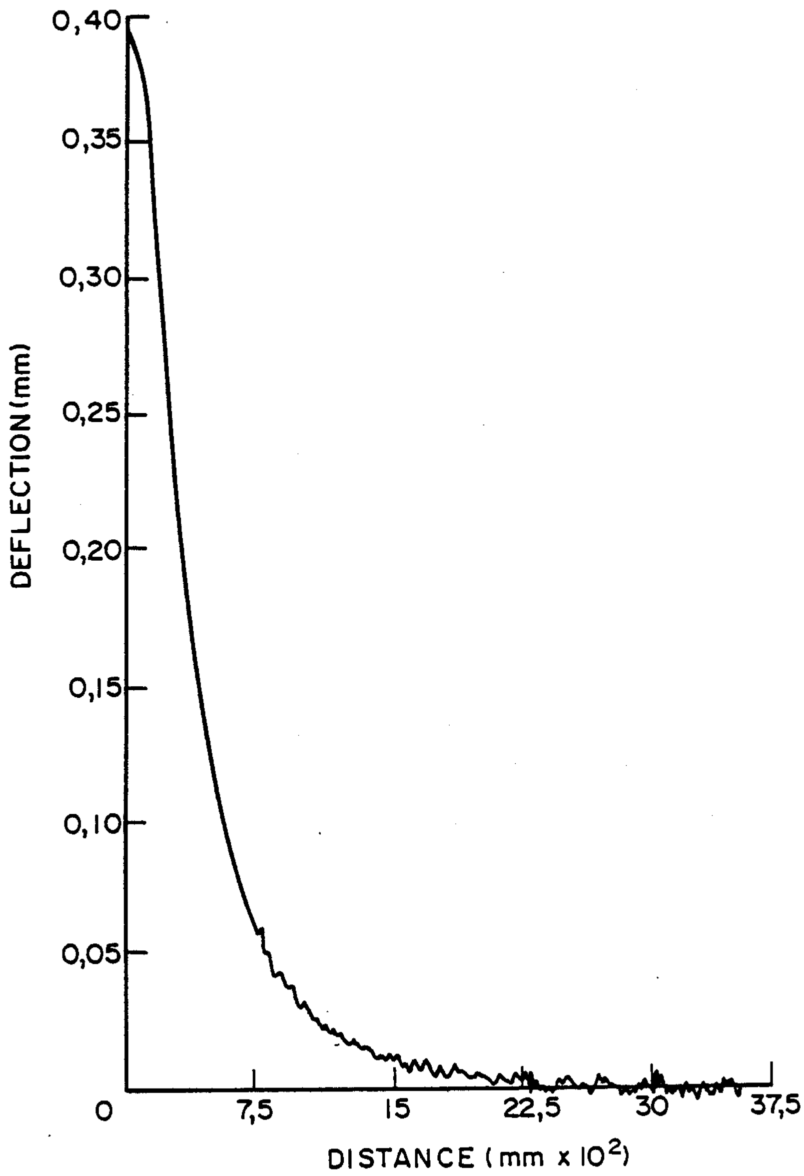


FIGURE 2.10

TYPICAL MANIPULATED RSD DEFLECTION BASIN HALF OF DATA SET deflection basin parameters as discussed in Chapter 1. The various deflection basin parameters and their respective formula as found in the literature survey are all shown in Table 1.1. In Table 2.1 the number of deflection basin parameters has been reduced to 11. All these deflection basin parameters listed here can be calculated by using deflection points at standard off-sets on a specific deflection basin data set. These deflection points are at the off-sets of 0, 127, 305, 610 and 915mm. These deflection points can easily be determined by a simple interpolation procedure (Horak and Otte, 1985) on such a prepared data set. This expands the description of the deflection basin considerable if compared to the traditional limited description of the deflection basin by calculating only the first two parameters listed in Table 2.1 (maximum deflection and radius of curvature).

The author also explored the area of a mathematical expression for the whole deflection basin. The measured deflection basins are discrete points and various mathematical and physical models available , were selected and tested to fit these discrete measuring points. The SPSS Statistical Package for the Social Sciences (Nie et al, 1975; Robinson, 1984) was used to do linear and non-linear regression analyses. This work is described in detail in Appendix A. When linear regression analysis procedures are used mathematical models can either describe the area of positive curvature or that of the negative curvature well, but not both areas simultaneously (see figure 2.7). This regression analysis method also cause a slight distortion in the area of positive curvature.

The use of the non-linear regression analysis procedures does lead to improved fittings of either the positive or negative curvature of the deflection basin (see figure 2.7). The mathematical models still either describe the positive or negative curvature well, but not both.

The physical model of a point load on an unlimited beam on elastic foundations gave the best fit using the non-linear regression analysis procedure. It also gave a better fit of the whole

TABLE 2.1 SUMMARY OF DEFLECTION BASIN PARAMETERS

Parameter	Formula	Measuring device
1. Maximum deflection	δ_0	Benkelman beam Lacroix deflectograph
2. Radius of curvature	$R = \dfrac{r^2}{2\,\delta_0(\delta_0/\delta_r - 1)}$ $r = 127$ mm	Curvaturemeter
3. Spreadability	$S = \dfrac{[(\delta_0+\delta_1+\delta_2+\delta_3)/5]\,100}{\delta_0}$ $\delta_1 \ldots \delta_3$ spaced 305mm	Dynaflect
4. Area	$A = 6\,[1+2(\delta_1/\delta_0) + 2*(\delta_2/\delta_0) + \delta_3/\delta_0]$	Falling weight deflectometer(FWD)
5. Shape factors	$F_1 = (\delta_0 - \delta_2)/\delta_1$ $F_2 = (\delta_1 - \delta_3)/\delta_2$	FWD
6. Surface curvature index	$SCI = \delta_0 - \delta_r$; where $r = 305$ mm or $r = 500$ mm	Benkelman beam Road rater FWD
7. Base curvature index	$BCI = \delta_{610} - \delta_{915}$	Road rater
8. Base damage index	$BDI = \delta_{305} - \delta_{610}$	Road rater
9. Deflection ratio	$Q_r = \delta_r/\delta_0$ where $\delta_r \simeq \delta_0/2$	FWD
10. Bending index	$BI = \delta/a$ where a = Deflection basin	Benkelman beam
11. Slope of deflection	$SD = \tan^{-1}(\delta_0 - \delta_r)/r$ where $r = 610$ mm	Benkelman beam

deflection basin, but still lacked accuracy in the small area of maximum deflection (see figure 2.7 or Appendix a for detail). The combined use of a mathematical model in this area solves the problem. The use of the physical model leaves the opportunity for detailed evaluation on a theoretical basis.This aproach was not pursued further in veiw of the fact that various deflection basin parameters , as described above, can be calculated with ease with a high degree of description of the full deflection basin. The practical and simple approach was therefore selected by this author to be pursued in this thesis rather than such a highly theoretical approach in spite of the challenging possibilities.

6. CONCLUSIONS AND RECOMMENDATIONS

(a) The measuring equipment, RSD and MDD, associated with the accelerated testing of the HVS is highly sophisticated and accurately measure deflection basins on the surface and in depth of pavement structures. The measuring procedure ensures statistically representative deflections of a pavement structure.

(b) The evaluation procedures of typical Benkelman beam and Dehlen radius of curvature meter measurements are not automated and limited deflection basin information in the vicinity of maximum deflection is gathered and processed. The data evaluation procedures of these rebound measurements are peculiar to the method and equipment.

(c) The measuring procedures of the RSD and MDD are automated and the whole deflection basin is recorded. The evaluation or data processing procedure make use of model curve fitting, but interest is concentrated on the small area of positive curvature while the large negative curvature of the basin is virtually ignored. The WASHO procedure of measurement is relatively free of plastic deformation effects and is preferred to the rebound method.

(d) For fundamental methods of pavement evaluation it is imperative that an accurate description of the deflection basin is obtained. Various deflection basin parameters can be used to reflect the whole deflection basin and associated structural relationships.

(e) Only the elastic side of a typical RSD or MDD deflection basin is processed to ensure that the data is relatively free of plastic deformation and possible interferences of reference points in the deflection basin.

(f) The data preparation procedure, even before model curve fitting is done, can provide as measured deflections (δ_0, δ_{127}, δ_{305}, δ_{610}, δ_{915} and δ_{2000}) which is normally used in the calculation of the majority of deflection basin parameters. It is suggested that this approach should be followed for normal data processing.

(g) Higher level data manipulation using linear and non-linear curve-fitting procedures were used to express the discrete set of measuring points of a deflection basin mathematically. The most successfull model was that of a point load on an elastic foundation using non-linear regression analysis techniques. This shows great promise ,but is not persued further here due to the obvious ease and accuracy of the simpler approach mentioned above.

CHAPTER 3

ANALYSIS OF DEFLECTION BASINS MEASURED DURING ACCELERATED TESTING

CHAPTER 3: CONTENTS

		PAGE
1	INTRODUCTION	3.2
2	MEASURED DEFLECTION BASIN PARAMETERS	3.3
2.1	Behaviour of deflection basin parameters with repetitions	3.3
2.1.1	Bitumen base pavement	3.3
2.1.2	Granular base pavement	3.7
2.1.3	Cemented base pavement (equivalent granular state)	3.10
2.1.4	Light structured granular pavement	3.14
3	BEHAVIOUR STATES	3.16
4	RUT RELATIONSHIPS IN THE FLEXIBLE BEHAVIOUR STATE	3.18
5	SUMMARY AND CONCLUSIONS	3.19

1. INTRODUCTION

In the preceding chapters it was described how the deflection basin can be described by means of various deflection basin parameters. The use of these parameters with the road surface deflectometer (RSD) was proposed as a viable analysis technique as the RSD accurately measures the full deflection basin. It was also shown how the analysis of the measured RSD deflection basins can be simplified to calculate all the deflection basin parameters as shown in Table 1.1.

Up to date,the RSD is associated only with the accelerated testing facility ,the fleet of heavy vehicle simulators (HVS's). As a vast number of pavement sections have been tested over the years, there exists potentially a vast data bank of information on RSD measured deflection basins. The original data processing of the accellerated tests were however done with other goals in mind. The result is that this RSD data is stored on magnetic tape in an ackward format. Detail background information of specific tests is further needed to aquire the neccessary information in the raw data form. Due to this cumbersome process of getting the data it was therefore decided to concentrate the effort on tests on typical representative pavement types tested where there is a high confidence in the knowledge of the detail of the test information. These tests selected would then act as a pointer for possible detailed analyses as a seperate project.

Accelerated tests carried out with the Heavy Vehicle Simulator (HVS) fleet on four different pavement structures were selected for a detailed analysis of the deflection basin data measured. The four HVS tests selected are representing pavements with a bituminous base on a cemented subbase (Opperman et al., 1983), a typical granular base on a cemented subbase (Horak and Maree, 1982), a typical light granular pavement structure (granular base and subbase) (de Beer 1982 and Van Zyl and Triebel, 1982) and a cemented base and subbase (Opperman, 1984 and Kleyn et al., 1985). The latter three have thin asphalt surfacings ($\leq$ 40 mm) which is typical of South African roads. The test on the cemented base pavement has particular significance too: it is the only HVS test where, after the completion of the initial test, the test section was overlaid and the test was continued on the overlaid section.

The curve fitting procedure described elsewhere (Horak, 1985 and Horak and Otte, 1985) was used by this author to calculate the deflection basin parameters from the deflection basins as measured during the HVS tests. Due to the small sample size of the relevant data sets the discussion on the relationships between the structural parameters of each individual layer and the calculated deflection basin parameters mean that no meaningfull regression analyses could be done. The discussion on this is therefore more general and directed towards the graphical evidence presented. The behaviour of the various deflection basin parameters related to the behaviour of the various pavement types under accelerated testing is discussed in detail. Suggestions are made as to the usefulness of these deflection basin parameters to describe pavement behaviour states and layers. The fold out at the back (Table 1.1) referring to the various deflection basin parameters should be used for ease of reference.

2 MEASURED DEFLECTION BASIN PARAMETERS

2.1 Behaviour of deflection basin parameters with repetitions

2.1.1 Bitumen base pavement

The various parameters, as calculated under a 40 kN dual wheel load, were related to the actual number of repetitions of the HVS test at Paradise Valley (Opperman et al., 1983). Parameters were grouped together in terms of units and range and all presented versus actual repetitions as shown in Figure 3.1. Although the actual test consisted of 2,3 x 10^6 actual repetitions, only about 1,5 x 10^6 actual repetitions are shown in Figure 3.1.

In general, deflections (δ_0 to δ_{915}) show a tendency to increase slightly with number of actual repetitions. This is as expected. What is significant though is that δ_{305} and δ_{610} show

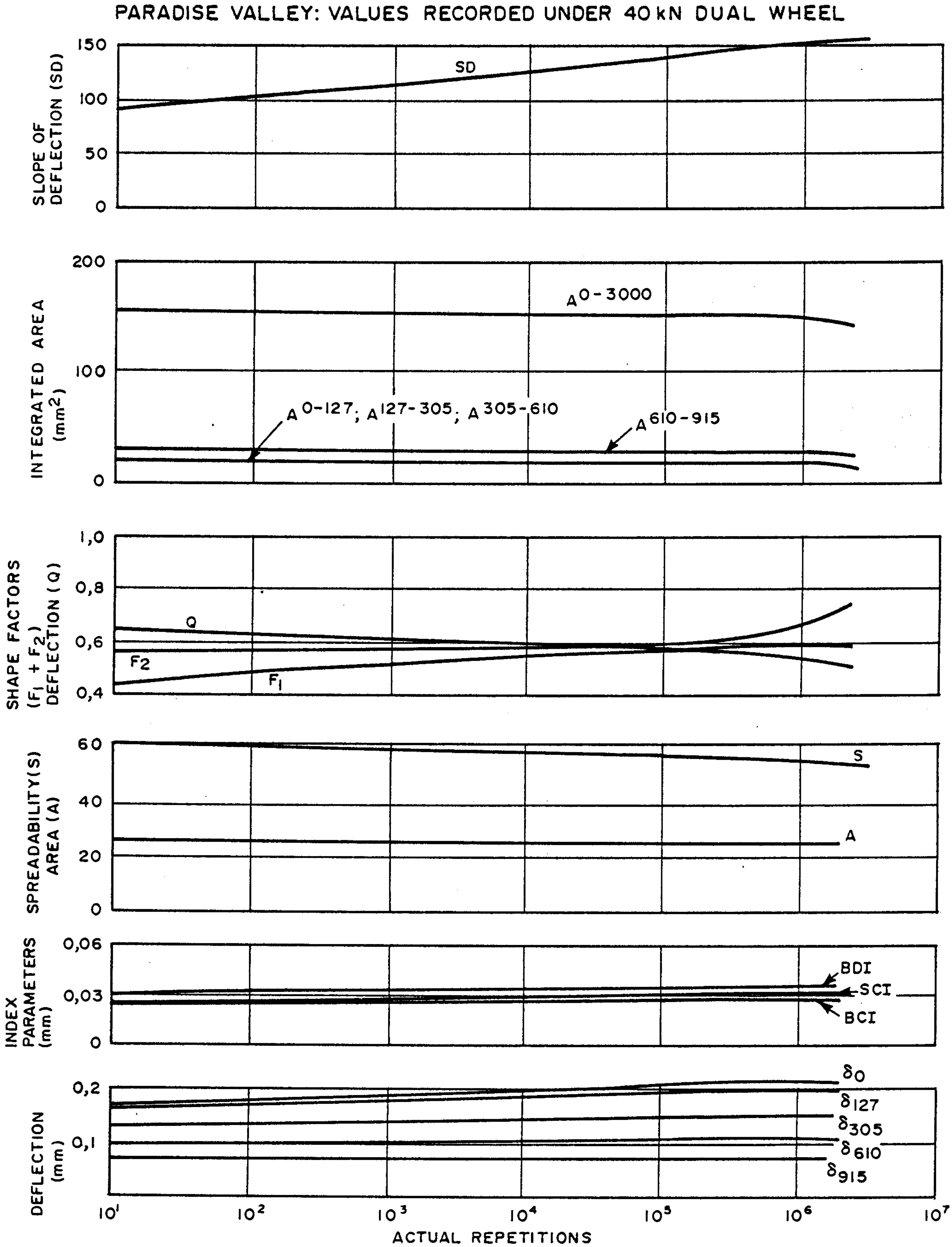


FIGURE 3.1

DEFLECTION BASIN PARAMETERS VERSUS ACTUAL REPETITIONS FOR A BITUMEN BASE PAVEMENT some relative increase in deflection to δ_{915}, which is virtually constant. This may reflect a change in elastic moduli of the subbase and selected layer (Tam, 1985). In the region of 10^5 to 10^6 actual repetitions all deflections (δ_0 to δ_{915}) level off. If these results are compared to the change in calculated effective elastic moduli as shown in Figure 3.2, it is confirmed that the subbase, selected layer and subgrade show the greatest change in this region (10^5 to 10^6 actual repetitions).

The index parameters, surface curvature index (SCI), base damage index (BDI) and base curvature index (BCI) also reflect the same changes as their calculations are based on the values of δ_0, δ_{305}, δ_{610} and δ_{915}. SCI shows only a slight increase in value with increase in actual repetitions. This reflects a bitumen base which is structurally strong, with minimal change in effective elastic moduli deeper in the pavement structure (Kilareski et al., 1982). This corresponds with the change in effective elastic moduli as shown in Figure 3.2 for the subbase, selected layer and subgrade over the range where their elastic moduli showed the greatest change. The slightness of change in BDI and BCI are indicative of the scale of change of the effective elastic modulus of the cemented subbase which still had a high residual effective elastic modulus at the end of the test.

Parameters such as spreadability (S) and area (A) reflect the whole of the deflection basin. The major portion of the deflection basin, particularly away from the point of maximum deflection, reflects changes in the lower layers (Tam, 1985). Figure 3.2 indicates that the changes in effective elastic layers were relatively small. This may contribute to S and A being relatively constant.
Shape factors (F1 and F2) and deflection ratio (Q) are not specifically related to any structural layers. In the calculation of these parameters the shape of the deflection bowl, or change thereof (e.g. becoming more peaked), would be reflected. This apparently did not happen.

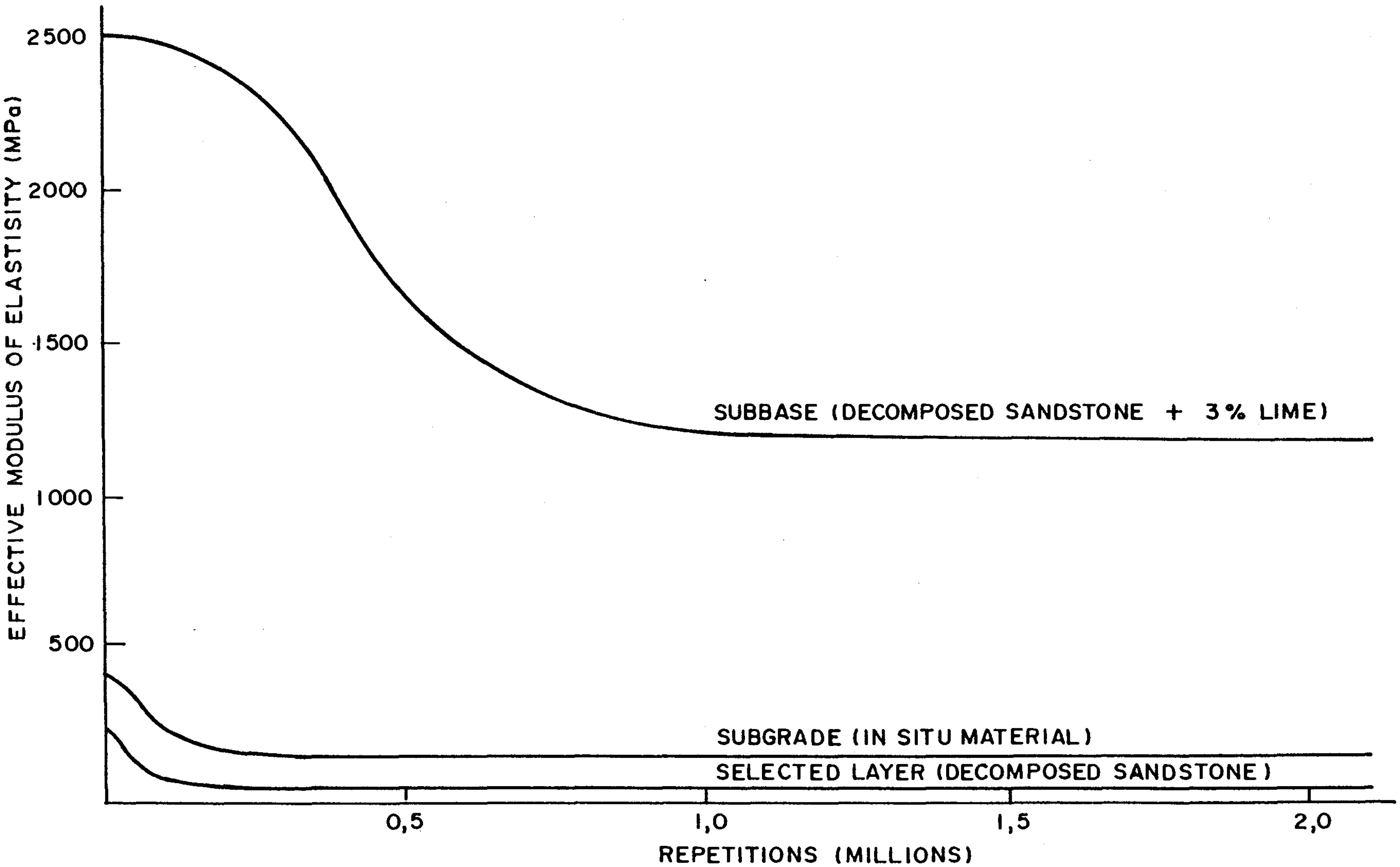


FIGURE 3.2

CHANGE IN EFFECTIVE E-MODULI OF THE DIFFERENT LAYERS IN THE HVS TEST 162A3 AT PARADISE VALLEY (OPPERMAN et al., 1983)

The statement above is enhanced when the fitted deflection basin functions (Horak, 1985) are integrated over various segments (e.g. 0 to 127 mm, etc.). Virtually no changes took place until about 1 x 10^6 actual repetitions. A^{0-3000}, which is the integrated area under the whole deflection basin, shows very clearly only a slight change after 1 x 10^6 actual repetitions.

Slope of deflection (SD) shows a gradual rise in value as actual repetitions increase. This parameter can unfortunately not be related to any change of any particular structural layer. It is however an indication of a deflection basin which is becoming gradually more peaked, but can, due to the low range it is covering, not be defined as peaked.

2.1.2 Granular base pavement

The various parameters, as calculated from deflection basin measurements under a 40 kN wheel load, are presented for the HVS test at Erasmia (Horak and Maree, 1982) in Figure 3.3. The deflection basin showed definite changes with the increase in actual repetitions. Although δ_{915} showed virtually no change, δ_{610} and δ_{305} to δ_0 in particular showed the greatest changes. This clearly indicates changes in the base and subbase effective elastic moduli (Tam, 1985). In Figure 3.4 it can be shown that the base and the subbase in particular did change considerably in terms of effective elastic moduli up to around 2 x 10^5 actual repetitions.

This change in the base and subbase effective elastic moduli is clearly reflected by the change in surface curvature index (SCI) (Anderson, 1977). The base damage index (BDI) and base curvature index (BCI) changed to a lesser extent, indicating changes in the effective elastic moduli of the selected layer and the subgrade (Kilareski and Anan, 1982). Radius of curvature (R) showed a definite reduction as actual repetitions increased which confirms the indication of the SCI values. Spreadibility (S) and area (A) did however not reflect any

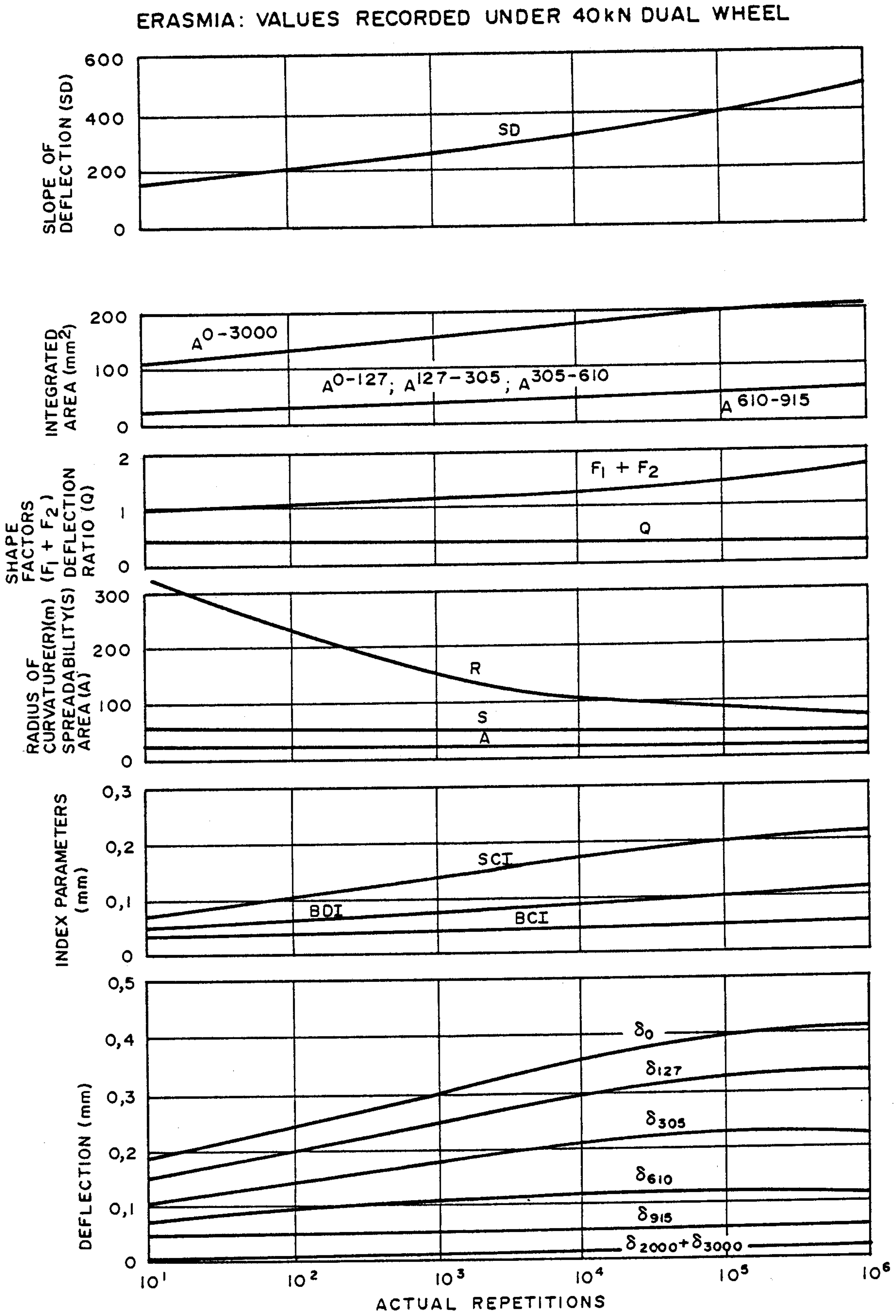


FIGURE 3.3

DEFLECTION BASIN PARAMETERS VERSUS ACTUAL REPETITIONS FOR A TYPICAL GRANULAR BASE PAVEMENT

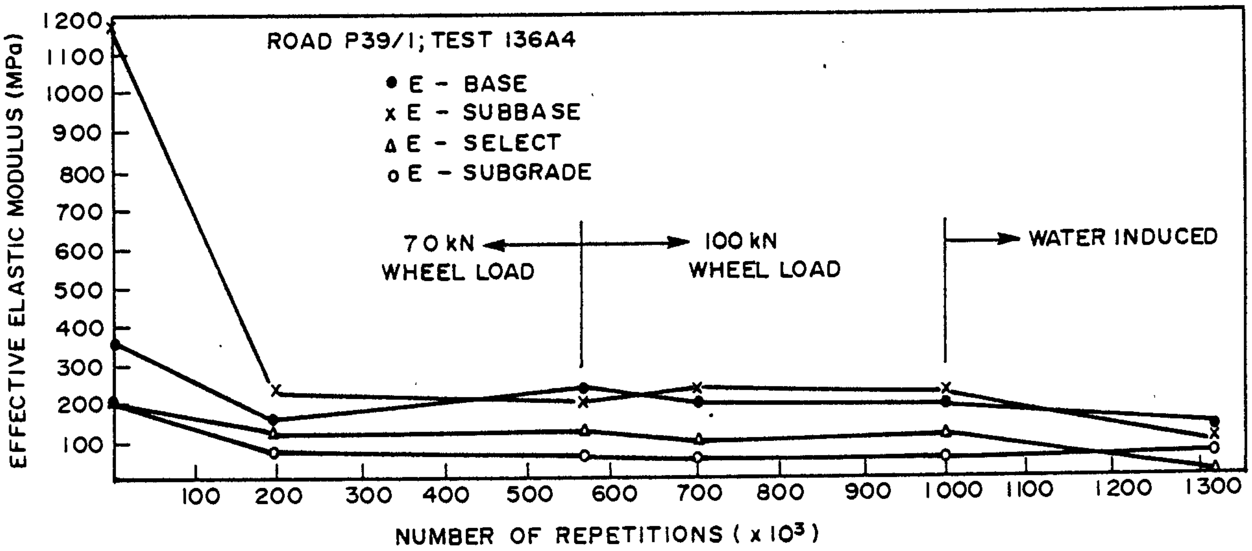


FIGURE 3.4

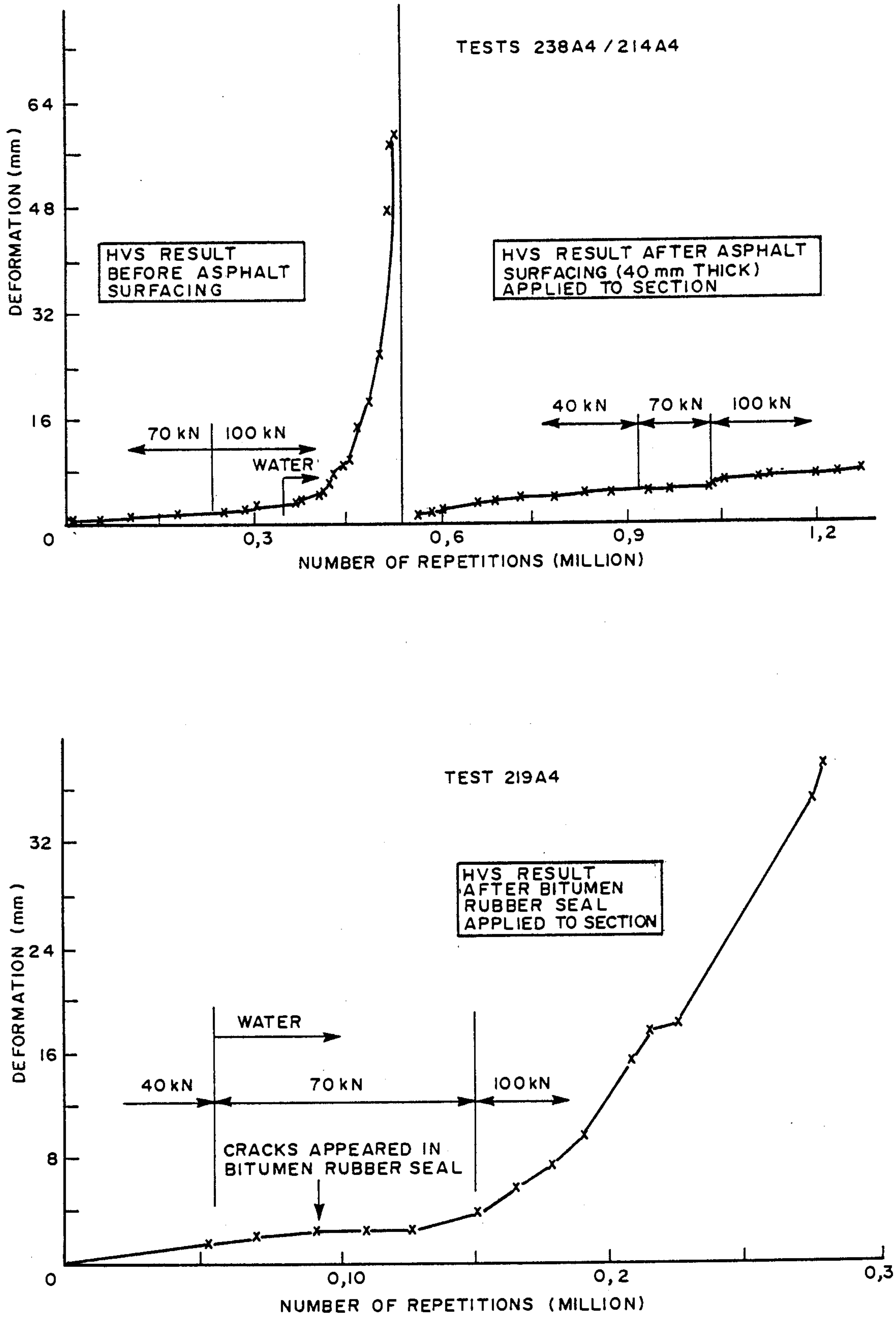
AVERAGE EFFECTIVE ELASTIC MODULI VALUES FOR A 40 kN DUAL WHEELLOAD (Horak and Maree, 198) change. Their values were slightly higher than those of the bitumen pavement but were also constant.

The parameters, shape factors (F1 and F2) were the same and showed a steady increase in value with the increase in actual repetitions. This is generally the same as the behaviour for bitumen base pavements, indicating a more peaked deflection basin developing. Deflection ratio (Q) stayed rather constant, reflecting no particular change of any structural layer.

The integrated area under sections of the function of the fitted deflection basin showed a slight increase in values for all the sections. The slope of deflection (SD) indicated by its change that the deflection basin is becoming more peaked. The initial value was higher than that of the bitumen base pavement, which clearly indicates that this granular base pavement had a much more peaked deflection basin from the start. This clearly reflects the lower effective elastic moduli of the granular base too.

2.1.3 Cemented base pavement (equivalent granular state)

The deflection basin parameters were determined for the cemented base pavement at Hornsnek (test 214A4) which was in a flexible state, exhibiting equivalent granular behaviour from the start of the test (see Figure 3.5). This is a rehabilitated or overlaid pavement (Opperman, 1984). In Figure 3.6 the deflection basin parameters are shown in relationship to the actual repetitions.
The deflections generally increase as measured at various points on the deflection basin (δ_0 to δ_{915}). The deflection furthest out (δ_{915}) showed only a slight increase with the increase in actual repetitions. It is however δ_{610} and δ_{305} which showed the greatest increase, reflecting changes in effective elastic moduli of the deeper layers (Tam, 1985). The δ_{127} value actually showed less increase than the δ_{305} value. This is typical behaviour of a granular base, as described earlier with a cemented subbase. Opperman (1984) reported that due to the



b) EFFECT OF BITUMEN RUBBER SEAL

FIGURE 3.5

AVERAGE DEFORMATION OF THE ROAD SURFACE AFTER REHABILITATION MEASURES (ROAD 30 HORNSNEK) (OPPERMAN, 1984)

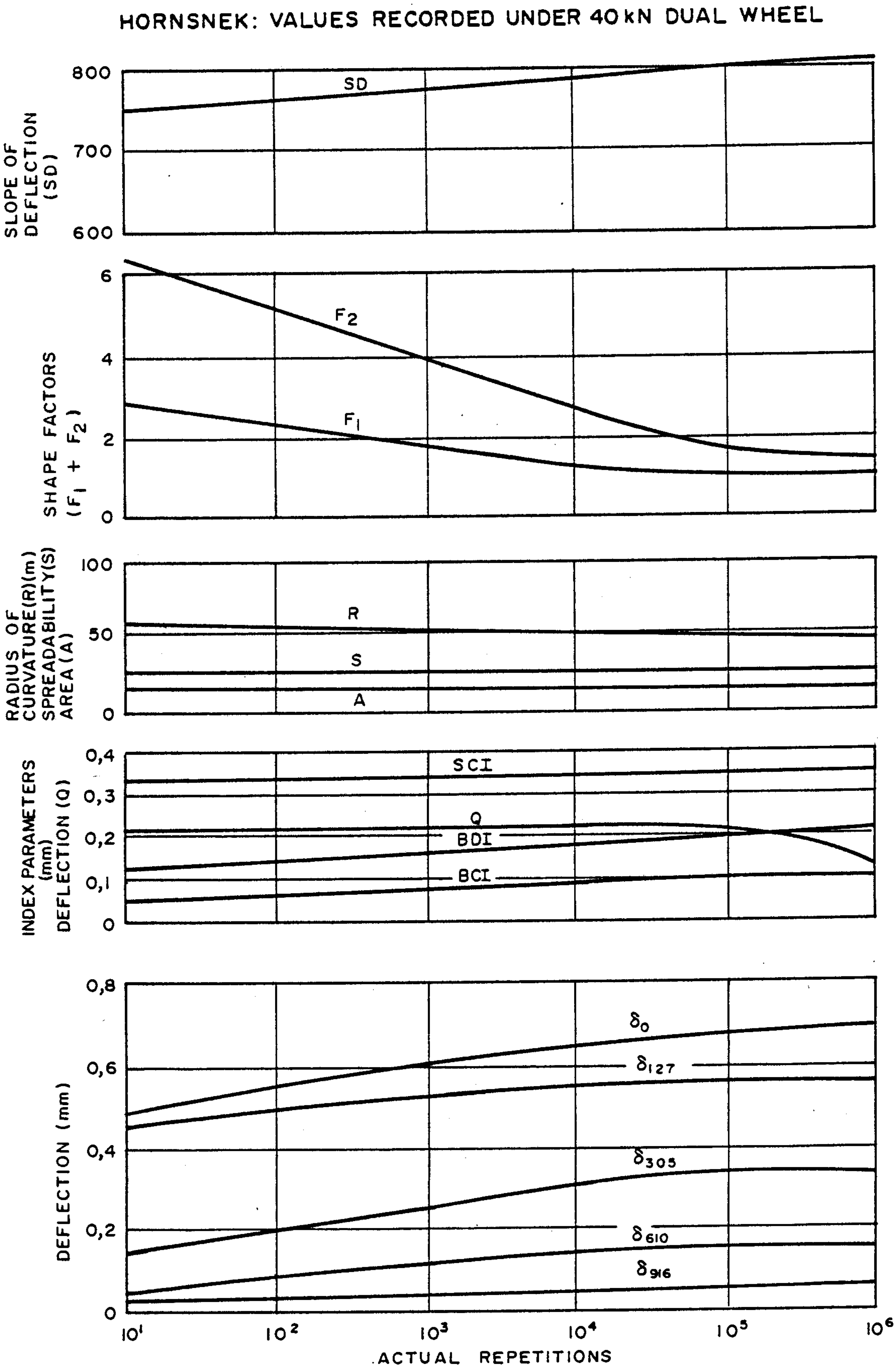


FIGURE 3.6

DEFLECTION BASIN PARAMETERS VERSUS ACTUAL REPETITIONS FOR A TYPICAL CEMENTED BASE EXHIBITING EQUIVALENT GRANULAR BEHAVIOUR crushing of the upper cemented base and limited cracking of the cemented subbase layer under the soft interlayer this is exactly the case.

The index parameters (SCI, BDI and BCI) confirm the observation of the change in the lower layers (subbase selected layer and subgrade) while the base is rather stable. The surface curvature index (SCI) stayed virtually constant with the increase in actual repetitions. The initial value is however in the same order where the granular base pavement, discussed earlier, ended. The base damage index (BDI) and base curvature index (BCI) showed a steady increase with the increase in actual repetitions. These values also started off at the level where the typical granular base pavement, discussed earlier, ended. This definitely confirm the changes in effective elastic moduli of the cemented subbase, selected layer and subgrade. The deflection ratio (Q) shown with the index parameters in Figure 3.6 is virtually constant up to 10^5 actual repetitions whereafter it declines. Unfortunately it does not indicate any specific relevance to structural change in the various layers.

The parameters, spreadibility (S) and area (A), like in the case of the granular base and bitumen base pavement, are constant and more or less in the same range. This again leads to the conclusion that these parameters are not sensitive to change in the various structural layers.

Radius of curvature (R) shows only a very slight increase, which is not unusual. It should be pointed out though that the level at which it starts off in itself is already low. It is interesting though that the shape factors (F1 and F2), contrary to the behaviour of the bitumen base pavement and the granular base pavement in particular, showed a decrease in value with the increase in actual repetitions. The starting values of this pavement in the equivalent granular behaviour state, is more than those of the granular base towards the end though. In line with the reasoning for the other pavement types earlier, the conclusion must be drawn that the deflection basins are becoming less peaked or more level. This line of reasoning is confirmed by the behaviour of the slope of deflection (SD) too. It started off at a very high value ($\pm$ 750), indicating a peaked deflection basin from the start as R indeed indicated. The rate at which SD increased with actual repetitions is even less than that of the bitumen base pavement discussed earlier. This points to the fact that there is some form of remoulding or change in balance of the pavement, becoming a "deep" pavement (Kleyn et al., 1985). This would definitely reflect more of the quality of the subgrade, which controls the outer edges of the deflection basin (Tam, 1985).

2.1.4 Light structured granular pavement

The deflection basin parameters as calculated for the light structured granular pavement on Main Road 18 near Malmesbury, are shown in Figure 3.7 versus the number of actual repetitions. As shown at the top of this figure, water did enter this test section. In general this wet phase leads to quite different behaviour of the deflection parameters as the pavement in this state failed rapidly. Discussion centres on the dry phase in order to correlate the general behaviour with that of the other pavement structures tested and discussed here.

In Figure 3.7 it can be seen that in the dry phase, deflection (δ_0 to δ_{915}) showed a general increase with the increase in actual repetitions. Even δ_{915} showed a definite increase, indicating that this can be termed a "deep" structure reflecting changes of the subgrade (Tam, 1985). Maximum deflection (δ_0) also shows an increase relative to δ_{305} indicating changes in the upper layers too. In the wet phase δ_{915} was the least affected due to the effect of a clayey subgrade (De Beer, 1983).

The index parameters, base damage index (BDI) and base curvature index (BCI), which reflect the effect of subbase and subgrade layers (Kilareski and Anani, 1982) showed an increase in value with actual repetitions in the dry phase. This indicates a lowering of the effective elastic moduli of these layers. The

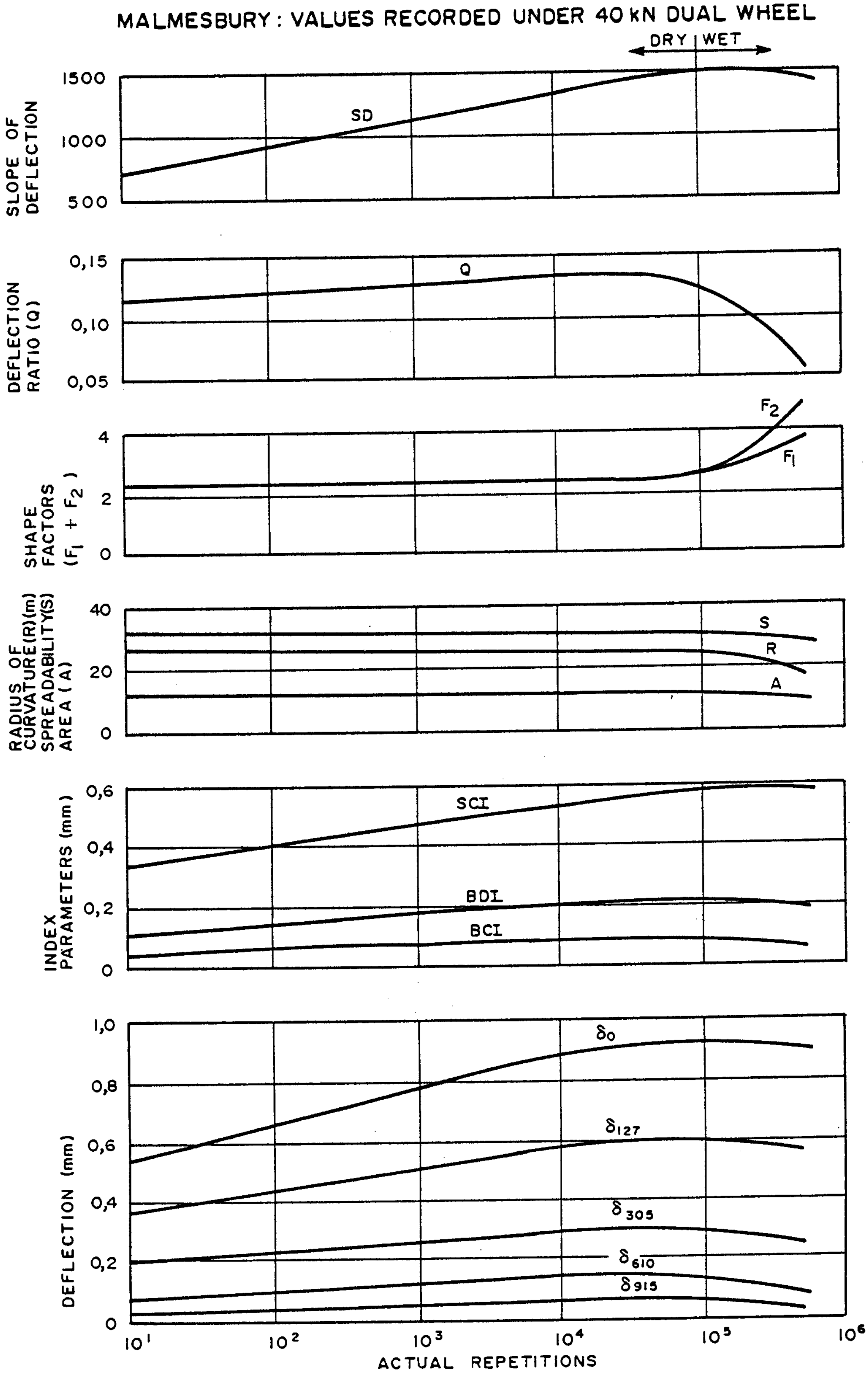


FIGURE 3.7

DEFLECTION BASIN PARAMETERS VERSUS ACTUAL REPETITIONS FOR A TYPICAL LIGHT STRUCTURED GRANULAR PAVEMENT surface curvature index (SCI) also indicates a lowering in effective elastic moduli of the base layer by the increase in SCI in the dry phase. In the wet phase the value of SCI levels off while BDI and BCI even show a lowering of their values. This is most probably due to plastic deformation or even shear deformation in the base (Van Zyl and Triebel, 1982) and the effect of the clayey subgrade (De Beer, 1983).

The radius of curvature (R), spreadibility (S) and area (A) parameters stayed virtually constant in the dry phase of this test. The low initial value of R, does however indicate a weak base with less support than with a cemented subbase. Shape factors (F1 and F2) and deflection ratio (Q) also either stayed constant during the dry phase of the test or were meaningless in terms of any structural changes. This author believes that the changes in the abovementioned deflection basin parameters only reflect the distressed state of the failing pavement at that stage without any specific reference to structural layers.

The slope of deflection (SD) showed an increase in value during the dry phase of the test. This may indicate a change in the form of the deflection basin, as the SCI values also indicate, in the region of the loaded wheel. Radius of curvature (R) however does not confirm this behaviour but this may be related to measuring problems (Horak, 1984). The initial value of SD is higher than that of the typical granular base pavement which indicates that the deflection basin was indeed rather peaked from the start. This again reflects on the lack of quality of the base and subbase.

3. BEHAVIOUR STATES

The behaviour of the different pavement types is controlled by the behaviour of the individual layers and by the interaction between the layers forming the pavement structure (Freeme et al., 1986). The definition of the states of behaviour is given by Freeme (1983). These states range from very stiff to very flexible as can be seen in Table 3.1.

TABLE 3.1 Definition of states of pavement behaviour

State	Approximate deflection range (mm)	Comments
Very stiff	< 0,2	Pavement behaviour predominantly controlled by high modulus (> 2 000 MPa) layers acting as slabs
Stiff	0,2 to 0,4	Pavement behaviour controlled by layers with reasonably high moduli (> 500 MPa. Some layers could be cracked but blocks tend to be larger than 2 m in diameter
Flexible	0,4 to 0,6	Pavement behaviour controlled by material in the granular state. Moduli of load-bearing layers in the range of 200 to 500 MPa (excluding thin surfacing moduli). Cementitious layers can be further cracked into smaller blocks
Very	> 0,6	Pavement behaviour controlled by materials flexible in the granular state usually with low moduli (< 200 MPa). Pavements tend to be susceptible to the ingress of water

TABLE 3.2 Ranges for behaviour states

Behaviour	Deflection basin parameter ranges				
	δ	SD($*10^{-6}$)	SCI	BDI	BCI
Very stiff	<0,2	<50	<0,01	<0,01	<0,01
Stiff	0,2 to 0,4	50 to 400	0,01 to 0,2	0,01 to 0,1	0,01 to 0,05
Flexible	0,4 to 0,6	400 to 750	0,2 to 0,4	0,1 to 0,15	0,05 to 0,08
Very flexible	>0,6	>750	>0,4	>0,15	>0,08

Using Table 3.1 the various HVS tests described earlier were classified according to their behaviour states. It is clear that the Paradise valley test with the bituminous base is still in the stiff behaviour state at the completion of the test. The granular base test at Erasmia changed from a stiff behaviour state to a flexible behaviour state after 100 000 actual repetitions. The cemented base in the equivalent granular state at Hornsnek,

changed from a flexible to very flexible behaviour state virtually from the start of the test. The same description holds for the light structured granular pavement tested at Malmesbury.

The ranges of the various deflection basin parameters that were identified as the most reliable indicators earlier were determined for each of the behaviour states. It is believed that by looking at more deflection basin parameters than only maximum deflection that a better definition of the behaviour states is possible. These ranges are shown in Table 3.2 .

4 RUT RELATIONSHIPS IN THE FLEXIBLE BEHAVIOUR STATE

In the previous section it was indicated that the granular base or pavements in the equivalent behaviour states were the only pavements that could be classified as being in either a flexible or very flexible behaviour state. For these pavements identified, the standard relationships between rut and actual repetitions are available.

The rut measurements of these pavements were correlated with the equivalent axle repetitions (E80s) and the measured deflection basin parameters in a stepwise multiple regression analysis. An exponent of n = 3 was used in the calculation of the E80 repetitions (Maree et al., 1982). In the stepwise multiple regression a R-square value of at least 0,75 was set or three variables in the relationship for acceptance. This resulted in a data matrix of 12 by 30, representative values of the granular base pavements in the flexible and very flexible behaviour states.

Two relationships were determined;

Relationship 1.... Rut = -13,696 + 5,698 (F1) + 4,153 (F2)

$R^2 = 0,79$

Where, Rut is measured in mm, F1, F2 are shape factors.

Relationship 2.

Rut = -15,205 + 6,002 (F1) + 0,444 (F2) + 3,935 (E80s).
Where; Rut, R^2 = 0,82; F1 and F2 areas described above, E80s are equivalent 80 kN acles in millions (10^6) repetitions.

The significance of these relationships is that granular base pavements in the flexible or very flexible behaviour states can be identified and rut can be calculated from deflection basin measurements. It is also significant that the two shape factors F1 and F2 which did not correlate well with changes in state or material state the base layer in particular, does correlate well with the permanent deformation behaviour. Most of the permament deformation in granular bases originate in the base layer (Maree et al., 1982).

5 SUMMARY AND CONCLUSIONS

Four accelerated tests with well documented results were selected to calculate deflection basin parameters. These tests represent typical bitumen base, granular base, cemented base and light pavement structure granular pavements. The latter three pavement types were in the flexible behaviour state while the bitumen base pavement was in the stiff behaviour state.

Due to the limoted amount of HVS tests analysed the findings reported earlier in chapter 2 were used as reference in the discussion. It was therefore only investigated to what extent the typical deflection basin parameters confirmed the indications of other researchers.

Considerable more HVS tests needs to be analysed with the proposed deflection basin parameters (Table 1.1). If statistically significant numbers of tests of each pavement type is available it is suggested that a proper regression analysis be done to see which deflection basin parameter correlate the best with important structuralparameters such as effective elastic moduli with the increase in repetitions.

In spite of the limited survey an attempt was made to make statements about the generalised relations between the deflection basin paramaters and effective elastic moduli changing with the number of repetitions. In line with findings reported in chapter 2 it can be stated that the index parameters (SCI, BDI and BCI) gave relatively clear indications of their relations with respectively the base, subbase and selected layers and their respective effective elastic moduli changing with the number of repetitions. Othe r deflection basin parameters gave rather vague relations if any which limits their use at present with this limited survey.

From the results of the various tests and measured deflection basins, it is possible to give a better description of the behaviour states. It is suggested that not only maximum deflection should be used to indicate in which behaviour state a pavement is, but preferably the index parameters (SCI , BDI , BCI) as indicated in the Table 3.2.

Rut can be calculated from deflection basin parameters F1 and F2 and optionally the E80s too. This is true for granular base pavements in the flexible or very flexible behaviour states for this small sample size analysed.

The author stronly recommends that in future a larger sample size of tested pavements be analysed in order to broaden the data base facilitating proper statistical analyses. This will be of particular benefit to possible relationships between deflection basin parameters and structural parameters of individual layers. With the new data manipulation programmes for HVS data this will in future be a relatively simple operation. A dedicated and concerted effort will however be needed for the data retrieval of the past tests. This author only sees his role in this regard as having investigated the possibilities for this present purpose.

CHAPTER 4

LITERATURE SURVEY ON MATERIAL CHARACTERIZATION AND STRUCTURAL ANALYSIS USING DEFLECTION BASINS

CHAPTER 4: CONTENTS

		PAGE
1	INTRODUCTION	4.2
2	GENERAL	4.4
3	METHODS INCORPORATING TWO-LAYER LINEAR ELASTIC THEORY	4.5
4	METHODS INCORPORATING THREE OR MORE LAYERS	4.8
4.1	Methods using up to two deflection basin parameters	4.11
4.2	Methods using at least three deflection basin parameters	4.20
5	CONCLUSIONS AND RECOMMENDATIONS	4.34

1 INTRODUCTION

The full deflection basin can be measured and the various deflection basin parameters give a better description of it. This was discussed in detail in the previous chapters. The main aim of this thesis is to use such measured deflection basin parameters in the analysis procedure. In Figure 4.1 the South African mechanistic rehabilitation design or analysis procedure is illustrated (Freeme, 1983). The pavement class and pavement type description of steps 1 and 2 will not be discussed here , but detail can be found elsewhere (TRH4, 1985a). In the previous chapter it was shown how the pavement behaviour state can be enhanced by using measured deflection basin parameters. Step 4 where the pavement layer state is described is also discussed in detail elsewhere (Freeme, 1983). In Appendix B a brief summary is given on how condition surveys enhance this identification of pavement layer state.

The main emphasis of this literature survey in this chapter is to indicate how measured deflection basins can also enhance step 5 (see Figure 4.1). The layer thickness of the pavement can be determined from as-built plans or by profile trenching. In the mechanistic design process effective elastic moduli are fundamental inputs in the analysis procedure in order to ensure that the evaluation procedure in step 6 (Figure 4.1) can be executed successfully. In South Africa the material classification as outlined in TRH14 (1985b) and TRH4 (1985a) is based on fundamental behaviour and strength characteristics. This description of materials form the basis for the description of the effective elastic moduli of each material type in a specific behaviour state (Freeme,1983). Various destructive and non-destructive tests are normally executed to ensure a correct definition of pavement material layers in terms of the effective elastic moduli used in the final non-simplified analysis procedure. Measured deflection basins , as a non-destructive testing means, can enhance the confidence in the assigned effective elastic moduli. In this chapter a detailed description

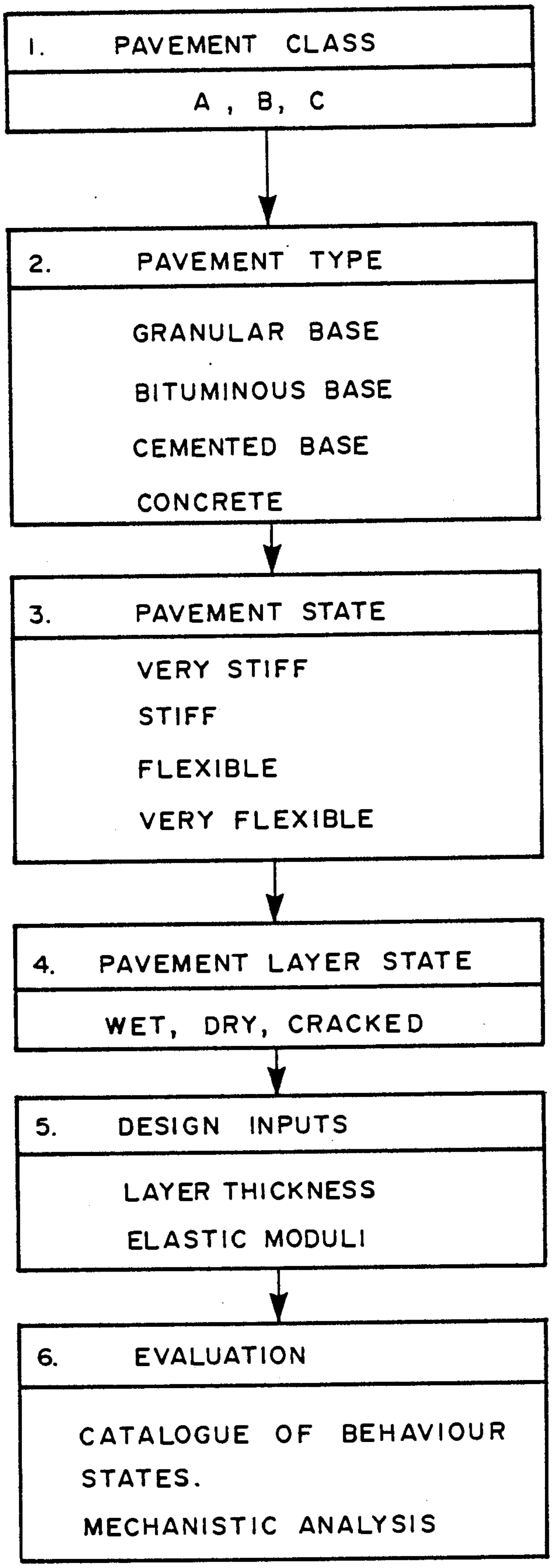


FIGURE 4.1

FLOW DIAGRAM OF THE MECHANISTIC PAVEMENT REHABILITATION DESIGN METHOD (FREEME, 1983) is therefore given given of various analytical methods of material characterization using deflection basin measurements as main input as found in literature.

2 GENERAL

In the characterization of materials of flexible pavements, various deflection based analysis methods are used. There are two main groups of methods. The first group is empirically based procedures with limited information on the full deflection basin and with limited deflection criteria. The second group uses basic elastic behaviour to determine the material properties. The majority of procedures in this latter group are based on linear elastic layered theory while the non-linear elastic behaviour of pavement materials is approximated by using linear elastic layered pavement models or finite element computer programs. The emphasis in this discussion is on the linear elastic group of methods of analysis. These procedures use computerized iterative, graphical, tabulated or nomograph solutions to analyse the pavement structures.

In a review of methods using layered elastic theory to model pavements Whitcomb (1982) observed the following common characteristics -

(a) The methods use two to five deflection basin values in the analysis.
(b) These models have two to five layers corresponding with the number of elastic moduli which are back-calculated.
(c) Some of the methods are developed for a specific non-destructive testing device.
(d) There are two basic methods of analysis: computerized iterative solutions and graphical fittings or nomographs.

Whitcomb (1982) also lists the following shortcomings of the layer elastic theory in its application to characterize pavement materials:

(a) An inability to analyse effects of loads at discontinuities (e.g. cracks and edges)

(b) The validity of the assumptions with regard to interface conditions

(c) Their inability to handle inertial forces or vibrations

(d) Non-linear stress and strain behaviour of materials (granular materials).

Of these limitations the last is considered to be the most restrictive. It is generally accepted that most materials used in pavements, in particular unbound granular materials, exhibit non-linear stress and strain behaviour. This leads to the fact that the elastic modulus of such materials is a function of the stress level and the result is that the elastic moduli will change both with depth and lateral position. Patterson et al. (1974) states that when there are more than two layers in a pavement, the number of unknown variables increases by three per layer and the system is generally insoluble unless the response characteristics can be defined more fully. Patterson et al. (1974) also observed that the theoretical models have an inherent weakness in terms of the accuracy in which it can represent the behaviour of the real pavement. They summarised the expected errors in peak surface deflection to be -30 per cent to +60 per cent when material moduli were measured in the laboratory. In more vigorous studies the errors were generally reduced to approximately 20 per cent. Patterson et al. (1974) noted that another limitation of the deflection influence surface technique is that is not very accurate with regard to the layers very near the surface. The limitations of the modelling of the loaded areas cause some doubt as to the accuracy of the deflections within an area of 0,5 m of the loaded area.

The discussion on techniques or methods for determining elastic moduli will be divided into two groups. The first group looks at two-layer models and the second group at three-layer or multilayer pavement models.

3. METHODS INCORPORATING TWO-LAYER LINEAR ELASTIC LAYER THEORY

Two-layer linear elastic layer theory can be used for preliminary analysis and interpretation of generalized trends. This may appear to be an over-simplification of more complex pavement structures, but the fact remains though that most surfaced and unsurfaced (low volume) roads have a typical two-layer pavement structure. This is particularly true in South Africa where surfacings are generally thin (average thickness of 25 mm). Considerable use is also made of chipsealing. In analysis methods this thin asphalt layer is often ignored owing to its limited influence on structural analysis.

The two-layered linear elastic model by Burmister (1945) is usually used as the basis for most of these methods. Yoder and Witczak (1975) state that the stress and deflection values obtained by Burmister are dependent upon the strength ratio of E1/E2 of the layers , where E1 and E2 are the moduli of the reinforcing and subgrade layers respectively. Maximum deflection values for a flexible plate (which represents a tyre load) are determined as follows (Yoder and Witczak, 1975):

$$\delta_o = \frac{1{,}5\ pa}{E2} F2$$

where p = unit load on circular plate
a = radius of plate
E2 = modulus of elasticity of lower layer
F2 = dimensionless factor depending on the ratio of the moduli of elasicity of the subgrade and pavement as well as the depth to radius ratio

Wiseman et al. (1977) proposed the simplification of pavement structures by using the Burmister (1945) two-layer system or preferably the Hogg (1944) model. The Hogg model is that of an infinite plate on an elastic subgrade. Greenstein (1982) used these models for pavement evaluation of low-cost roads. In Figure 4.2 falling weight deflectometer(FWD) deflection basin measurements are used to arrive at values of Ep/Es (E1/E2)) for relative values of pavement thickness to load radius ratio and the specific values of poisson ratio (ν_ρ and ν_s). This nomograph is

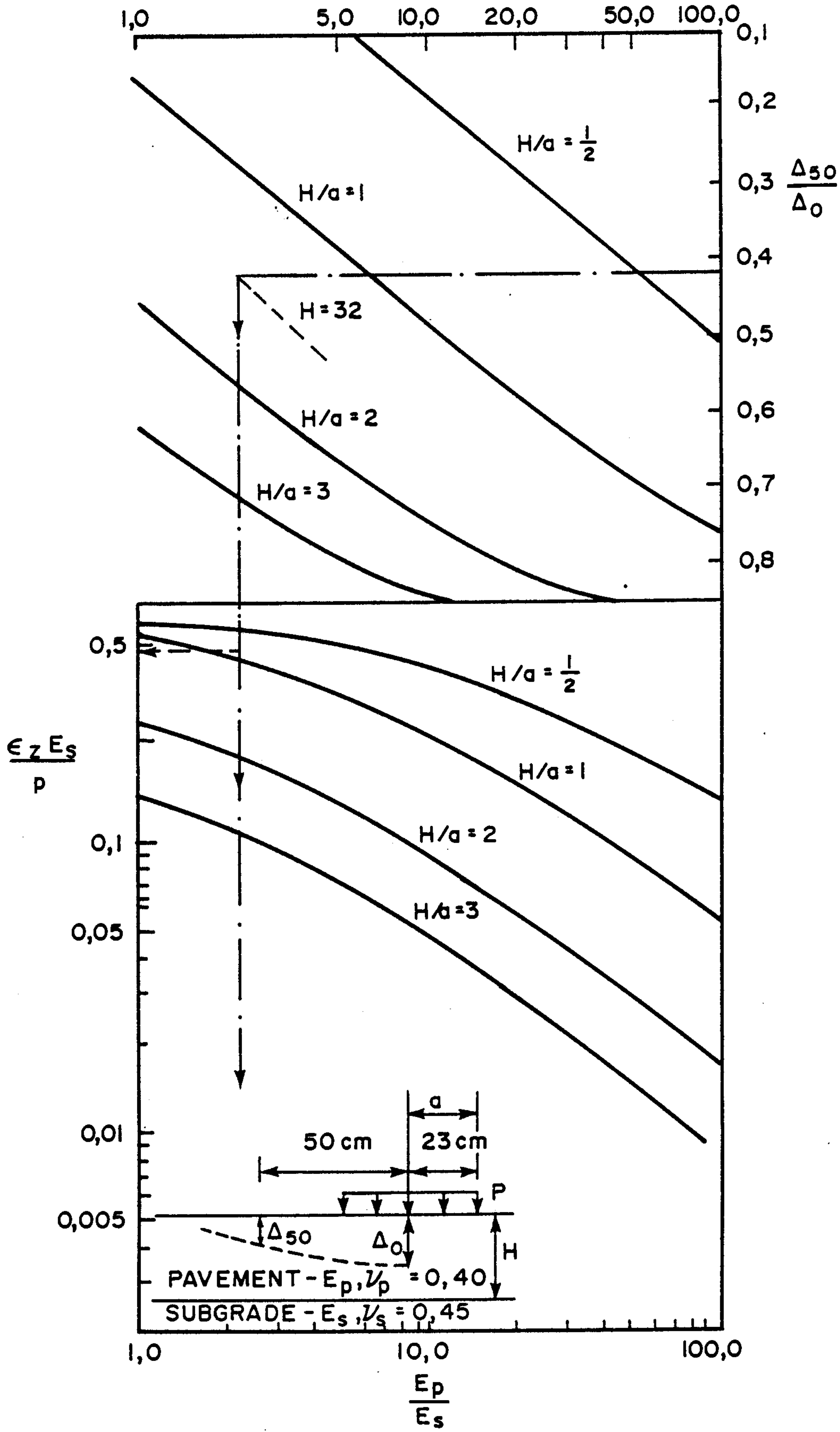


FIGURE 4.2
TWO-LAYER MODEL USED FOR
LOW-COST ROADS (Greenstein, 1982)

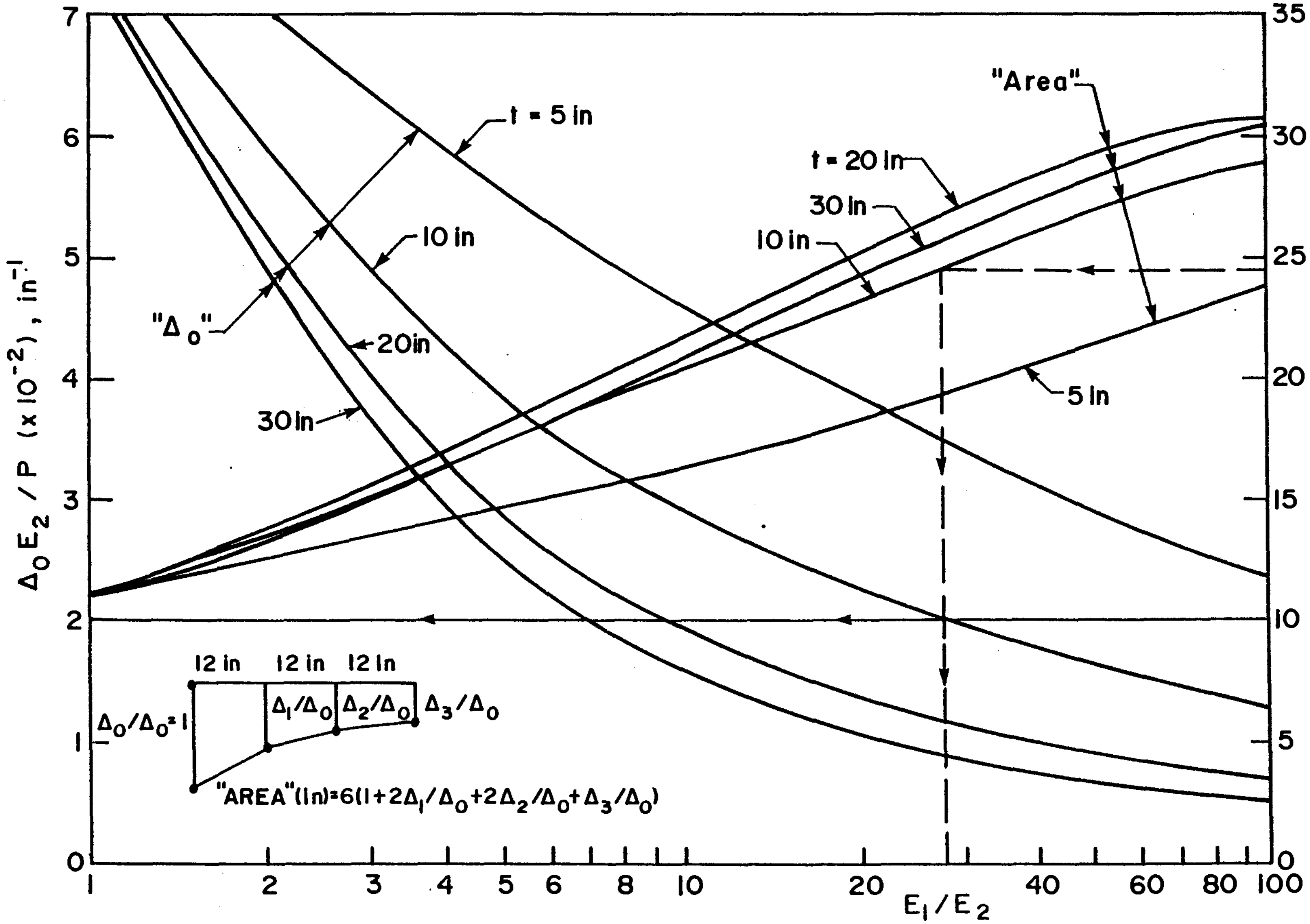
based on that derived by Wiseman et al. (1977). Estimate values of E1 and E2 are reached which may be used as a basis for further analysis. Berger and Greenstein (1985) simplified the use of the Hogg model so that it can be used on pocket calculators and deflections as measured with a Benkelman beam.

The general relationship of the ratio E1/E2 is used in other graphical or nomographical procedures using the Burmeister model (1945). In Figures 4.3 and 4.4, Hoffman and Thompson (1981) show how four deflection basin parameters, δ_o, A, F1 and F2, (see Table 1.1) are related to this basic elastic moduli ratio (E1/E2). These figures were developed for the Falling Weight Deflectometer (FWD) and Idaho Department of Transportation (IDOT) road raters. Other similar relationships were developed by Swift (1972) and Vaswani (1971) for the use of dynaflect deflection basin measurements.

In conclusion it can be stated that the elastic moduli based on Burmister's (1945) two-layered linear elastic theory can be back-calculated with confidence using the deflection basin measurements. The specific measuring device and deflection basin parameter selected will result in a specific graphical solution. It is suggested that if this method is used it should preferably be used with granular bases and be treated as a method of estimating effective elastic moduli. The Hogg model (1944) certainly also proved to be a viable method particularly for cemented or bituminous base pavements. It should however also be seen as an approximation procedure of more complex pavement structures.

4 METHODS INCORPORATING THREE OR MORE LAYERS

Linear elastic layer theory is widely used to determine effective elastic modulus values of the various layers based on back calculated deflection basin measurements. Although there are a wide variety of such computer models available with varying levels of sophistication only those relevant to the linear elastic theory



NOTE: DEFLECTIONS RESULTING FROM FALLING WEIGHT DEFLECTOMETER LOADING OR IDOT ROAD RATER LOADING MUST BE USED

FIGURE 4.3

VARIATION OF THE "AREA" AND THE MAXIMUM DEFLECTION FACTOR IN A TWO-LAYER LINEAR ELASTIC MODEL (Hoffman and Thompson, 1981)

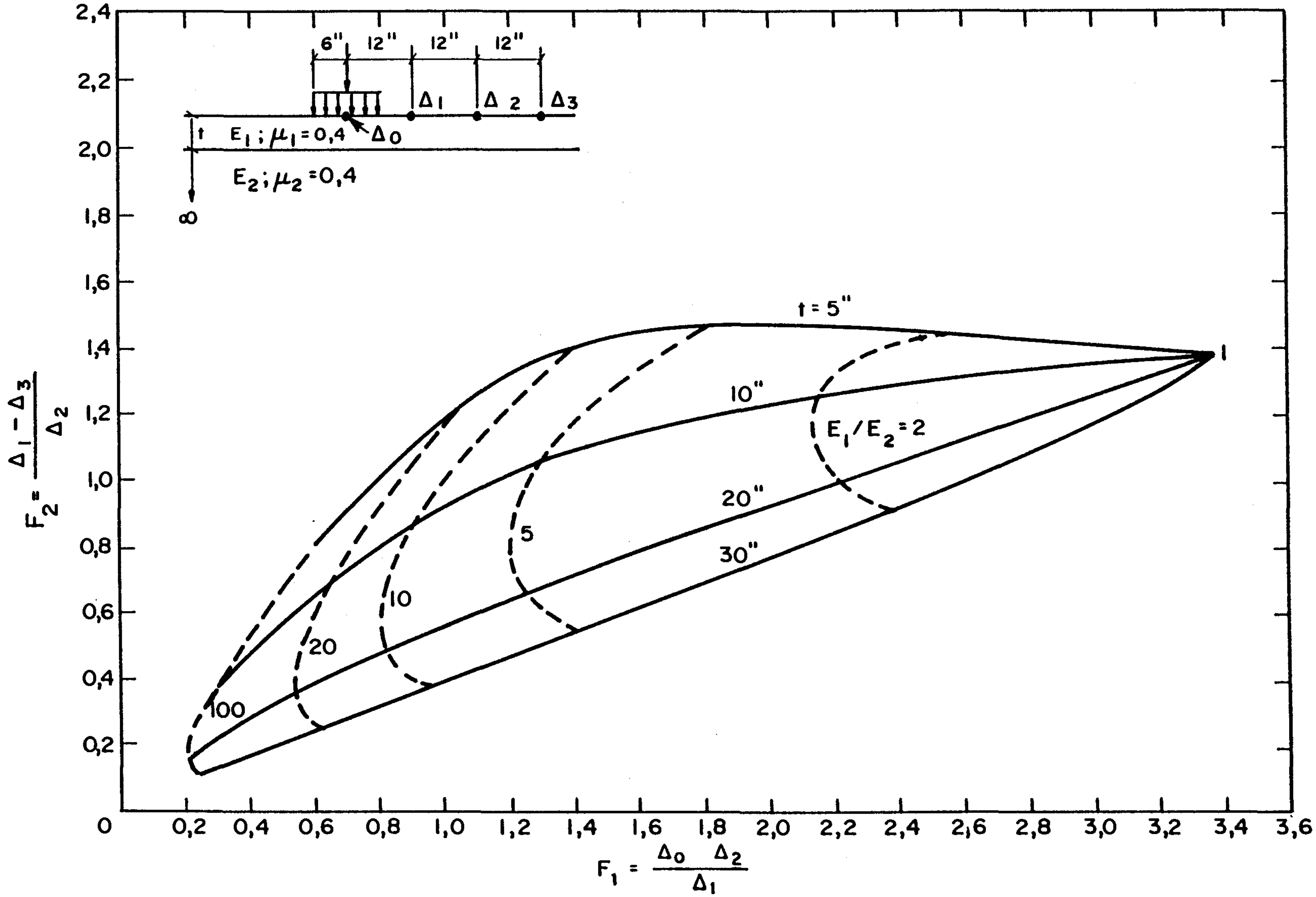


FIGURE 4.4

SHAPE FACTORS OF THE DEFLECTION BASIN IN A TWO-LAYER will be referred to. Methods using deflection basin measurements or parameters as basic input are discussed briefly below.

4.1 Methods using up to two deflection basin parameters

Grant and Walker (1972) describe a linear elastic layer theory approach to arrive at effective elastic moduli for a three-layered pavement structure. The deflection basin parameters δ_o and R are used. Modular ratios of surfacing-over-base and base-over-subgrade are calculated with the CHEVRON computer program and plots in relation to δ_o and R are derived, as shown in Figures 4.5 and 4.6 These relationships were developed for thin surfacings with granular bases, which are typical in South Africa. Modular ratios of the base-over-subgrade were defined as varying between two and four and the initial estimates of the subgrade modulus was derived from the relationship:

$$E_{subgr}=K*CBR$$

where K is a factor varying between 3,5 and 10,3 with the latter used in most cases.

Typical elastic moduli values are fixed for the asphalt surfacings as shown in Figures 4.5 and 4.6 with typical layer thicknesses for the surfacing and base layers. A Poisson ratio of 0,3 was used for the base and subgrade. Grant and Walker (1972) therefore conclude that "The above method could be used to obtain an estimation of the subgrade and base moduli for roads with thin asphalt surfacings".

The new Shell Pavement Design Manual (Shell, 1978) forms the basis of the method described by Koole (1979). The deflection basin parameters maximum deflection (δ_o) and deflection ratio (Q), determined from Falling Weight Deflectometer (FWD) measurements, are used. A three-layered linear elastic system is modelled. The pavement structure is characterized by eight variables:

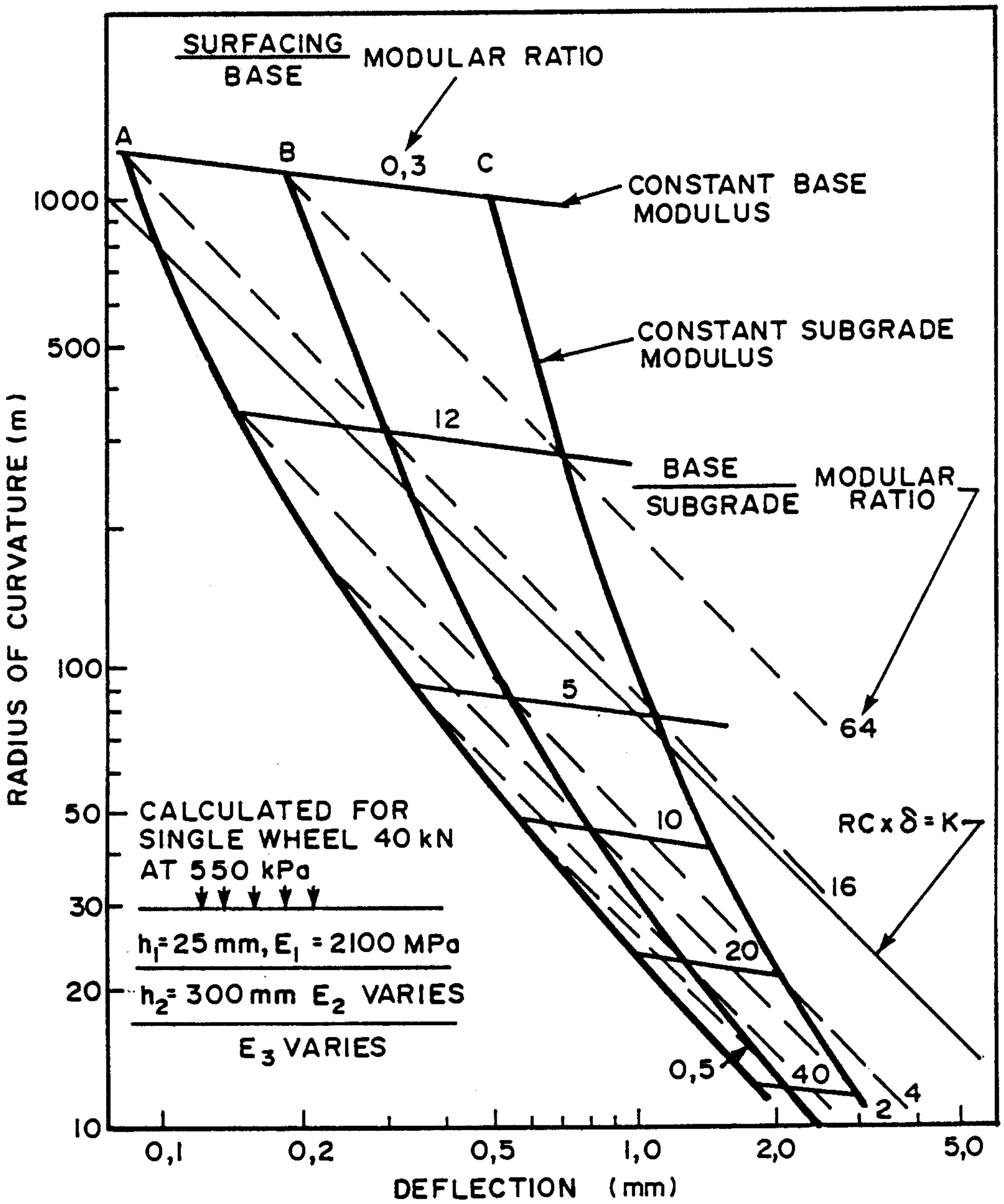


FIGURE 4.5

RELATIONSHIPS BETWEEN DEFLECTION AND RADIUS OF CURVATURE FOR CONSTANT BASE MODULUS, CONSTANT SUBGRADE MODULUS AND CONSTANT BASE/SUBGRADE MODULAR RATIO (SINGLE LOADED AREA (Grant and Walker, 1972)

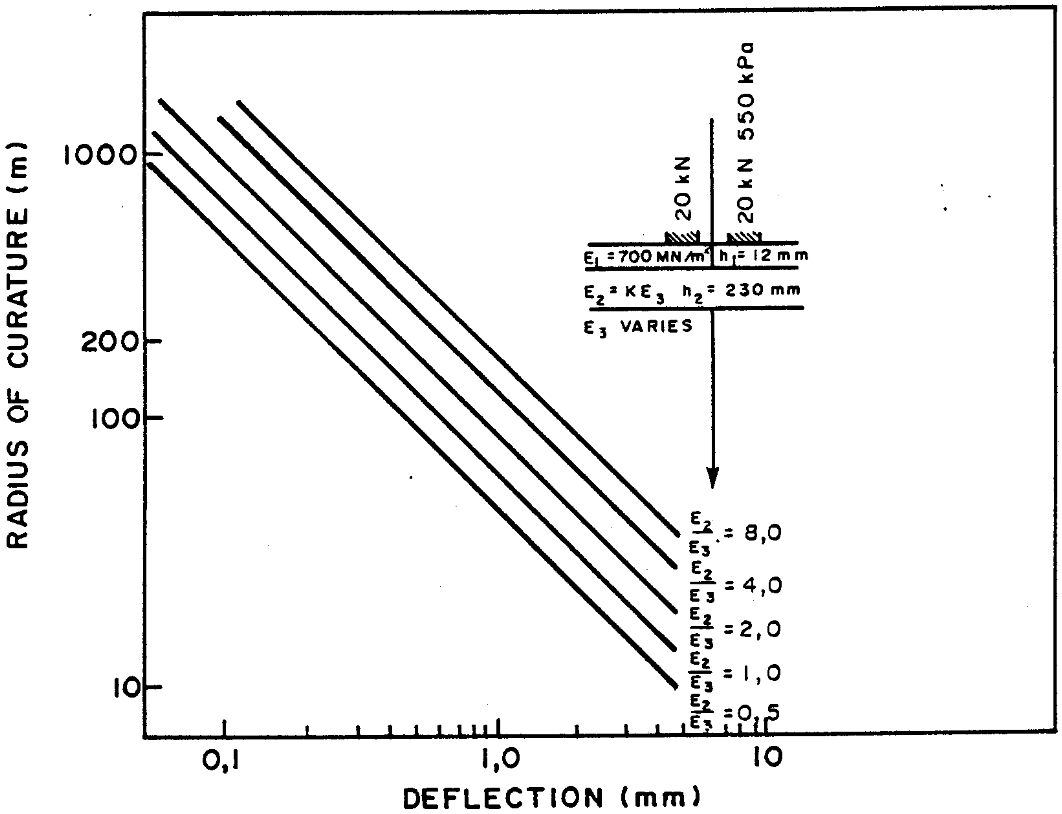


FIGURE 4.6

PRACTICAL MEASUREMENTS OF DEFLECTION AND RADIUS OF CURVATURE RELATED TO THEORETICAL BASE/SUBGRADE MODULAR RATIOS (DUAL LOADED AREAS) (Grant and Walker, 1972)

h_1, h_2, E_1, E_2, E_3, ν_1, ν_2 and ν_3.

Two assumptions are made: Poisson's ratio is taken as being equal to 0,35 for all the layers and the effective modulus of the unbound base layer is a function of its thickness, h2, and the subgrade modulus E_3. The relationship of Dorman and Metcalf (1963) is used:

$E_2 = k * E_3$

where: $k = 0,2\ h_2^{\ 0,45}$; $2<k<4$ and h_2 is measured in millimeters.

The computer program BISAR is used to prepare a typical graphical chart such as that shown in Figure 4.7. In this case typical values of E_3=100 MPa and h_2=300 mm are used. The value of h_1 is determined from coring results or as-built records. By varying these values of E_3, E_2 and a resulting E1, deflection basin parameters maximum deflection (δ_o) and deflection ratio (Q_r) are matched and values of effective moduli are thus obtained. Koole (1979) states that estimate values of effective moduli for cement-treated bases derived from past experience or measurements should be used as input in the BISAR computer analysis. The nomograph in the Shell Pavement Design Manual (Shell, 1978) is a well-known procedure to determine estimate values of the effective moduli for the asphalt layer.

Snaith et al. (1980) analyse and arrive at effective elastic moduli for a three-layered pavement structure using the familiar relationship between CBR and subgrade elastic modulus and the relationship between the effective moduli of the base layer (granular) and the subgrade. These are the same relationships as those described in the methods proposed by Grant and Walker (1972) and Koole (1979).

Snaith et al. (1980) use the deflection basin parameter, δ_o, as measured with a Benkelman beam. A simple method of

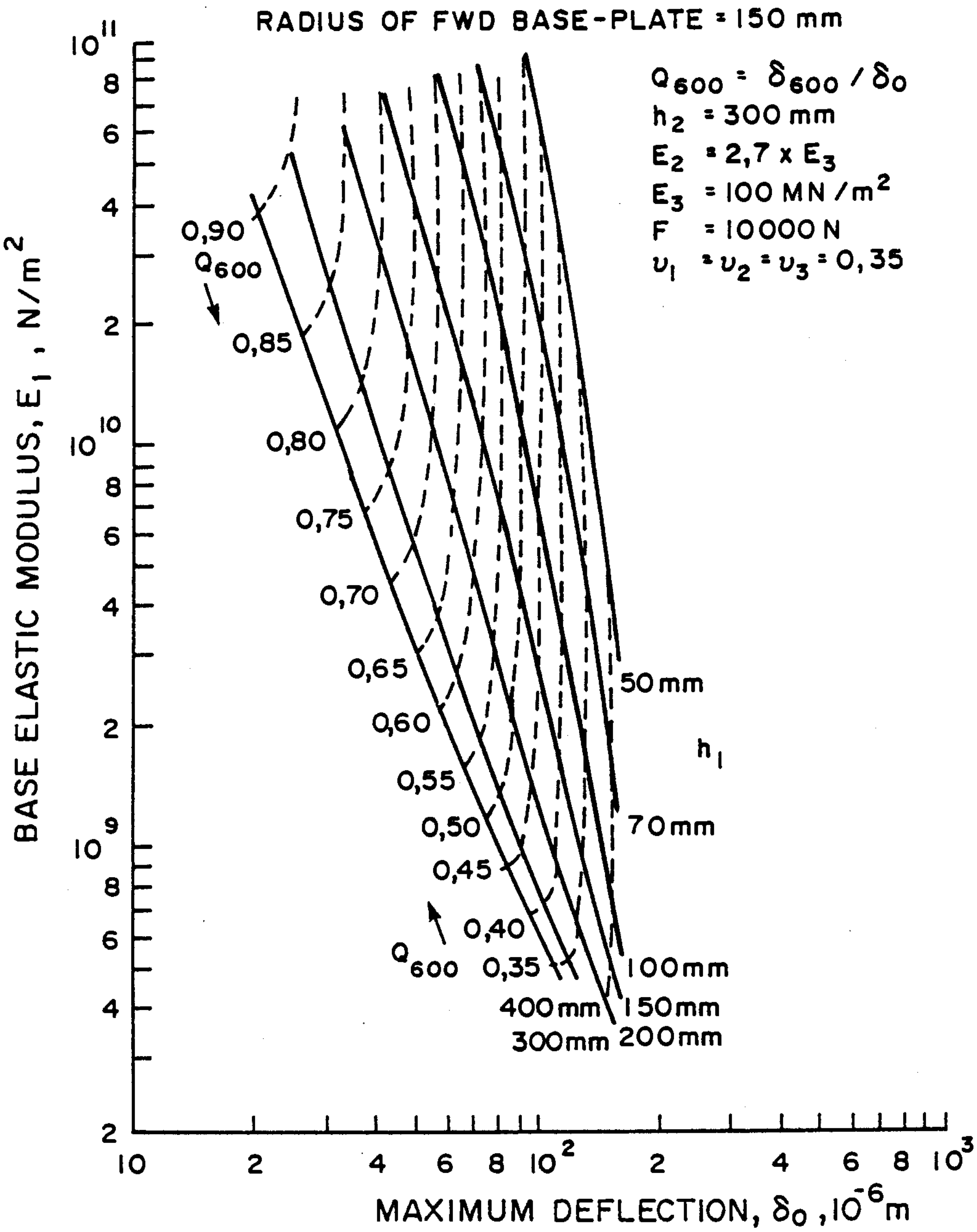


FIGURE 4.7
DEFLECTION INTERPRETATION CHART
(Koole, 1979)

approximating equivalent thicknesses (Odemark, 1949) is used. The solution consists of two steps:

(a) transformation of the multi-layered system into a single layer with equivalent thickness, and

(b) use of the solutions for distributed loads on the surface of a linear elastic semi-infinite mass.

In order to change the layered system into an elastic half-space the equivalent thickness (H_e) is calculated for each layer. The principle is that the equivalent layer has the same stiffness as the original layer, so as to give the same pressure distribution underneath the layer. This results in the following relationship:

$$H_e = h_1 \left[\frac{E_1 * (1-\nu_2^{\,2})}{E_2 * (1-\nu_1^{\,2})} \right]^{\frac{1}{3}}$$

If the value of the Poisson's ratio is the same for both materials, the expression is reduced to:

$$H_e = h_1 \left[\frac{E_1}{E_2} \right]^{\frac{1}{3}}$$

For a uniform vertical loading on a circular area, the deflection δ_o at any depth Z at the centre of the loaded area is given by

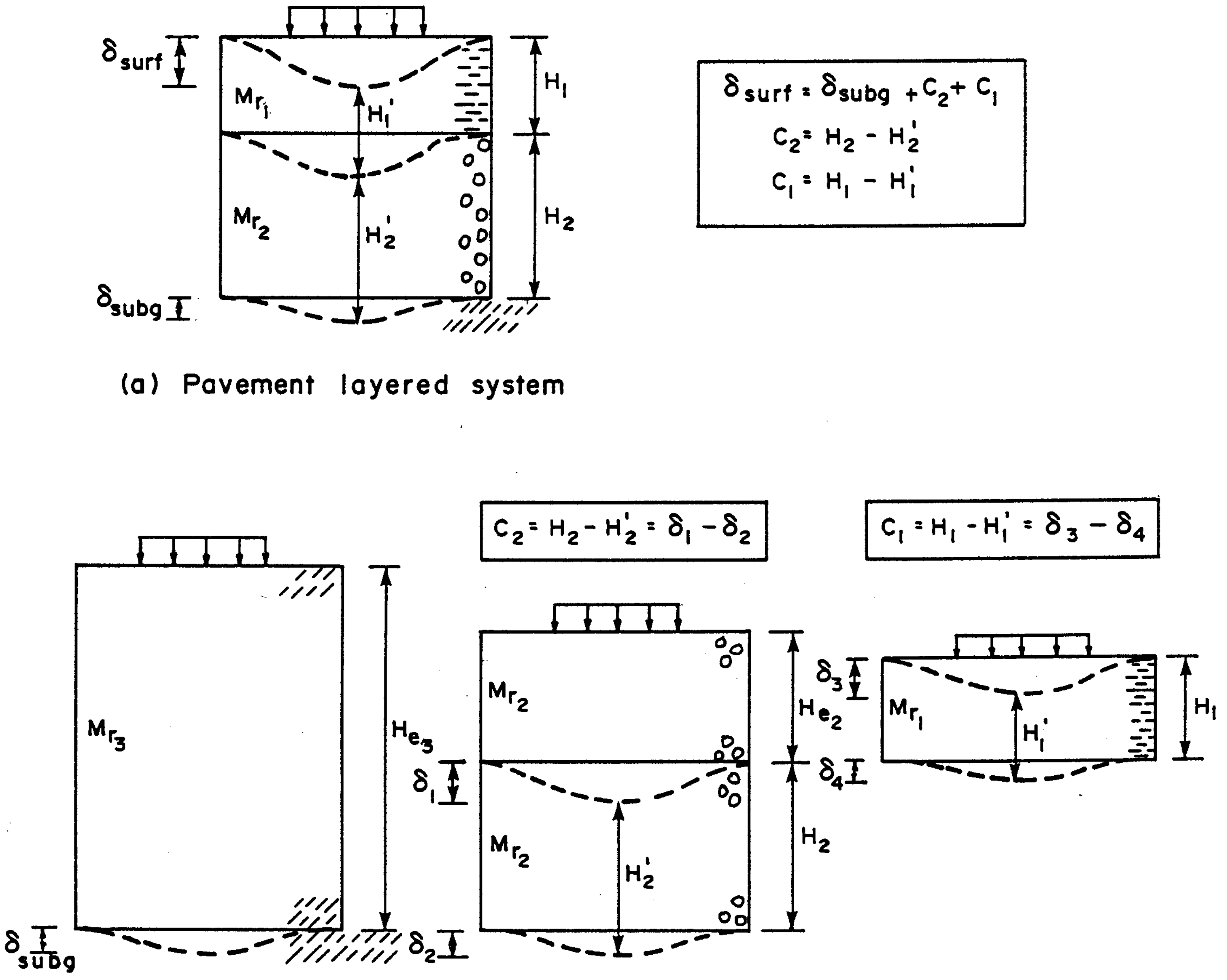
$$\delta_Z = \frac{(1+\nu)}{E} pa \left[\frac{1}{(1+(Z/a)^2)^{\frac{1}{2}}} + (1-2\nu)(1+(Z/a)^2)^{\frac{1}{2}} Z/a \right]$$

Estimate values of effective elastic moduli are used as input values to determine the value of δ_o, which is compared with the measured value. The algorithm to arrive at this value is shown in the sketches in Figure 4.8. Snaith et al. (1980) use an electronic calculator to do the calculations and use the criterion of:

$$|\delta_{surf} - \delta_o(\text{measured})| < 0{,}05\ (\delta_o\ (\text{measured}))\ \text{for iterations.}$$

After a minimum of three deflection values have been found and plotted on graph paper, it is possible to estimate the required modulus value. It is not clear from the discussion by Snaith, et al. (1980) whether these deflection values refer to δ_o alone, but this is assumed to be so owing to lack of indications to the contrary. A check calculation can be done with this estimate value. It is not clear how the load distribution measured under dual tyres is represented in the calculations. Snaith, et al (1980) obviously only consider one loaded area. Dehlen (1962b) also used Odemark's approximate solution for a multi-layered system. Dehlen (1962b) states that; "Because of doubtful validity of Odemark's theory away from the axis of load, no attempt was made to compute deflections or curvatures between dual wheels and, in the case of such tests, computations were made simply for a single circular load of the same area as the dual wheel imprint." The results of such computations are shown in Figure 4.9. Dehlen (1962b) further indicates that differences exist between Elastic Theory and practice, mainly in that deflections are more concentrated about the load than indicated by theory, and that they decrease more rapidly with depth.

Molenaar (1983) analysed a three-layered linear elastic system using measurements taken with a falling weight deflectometer (FWD). Molenaar (1983) also makes use of Odemark's (1949) equivalency theory. He states: "... the equivalent layer thickness is a magnitude which is meaningful and easily understood. A pavement with a high H_e will last longer than a pavement with a low H_e". The equivalent layer thickness (H_e) is determined as follows:



(a) Pavement layered system

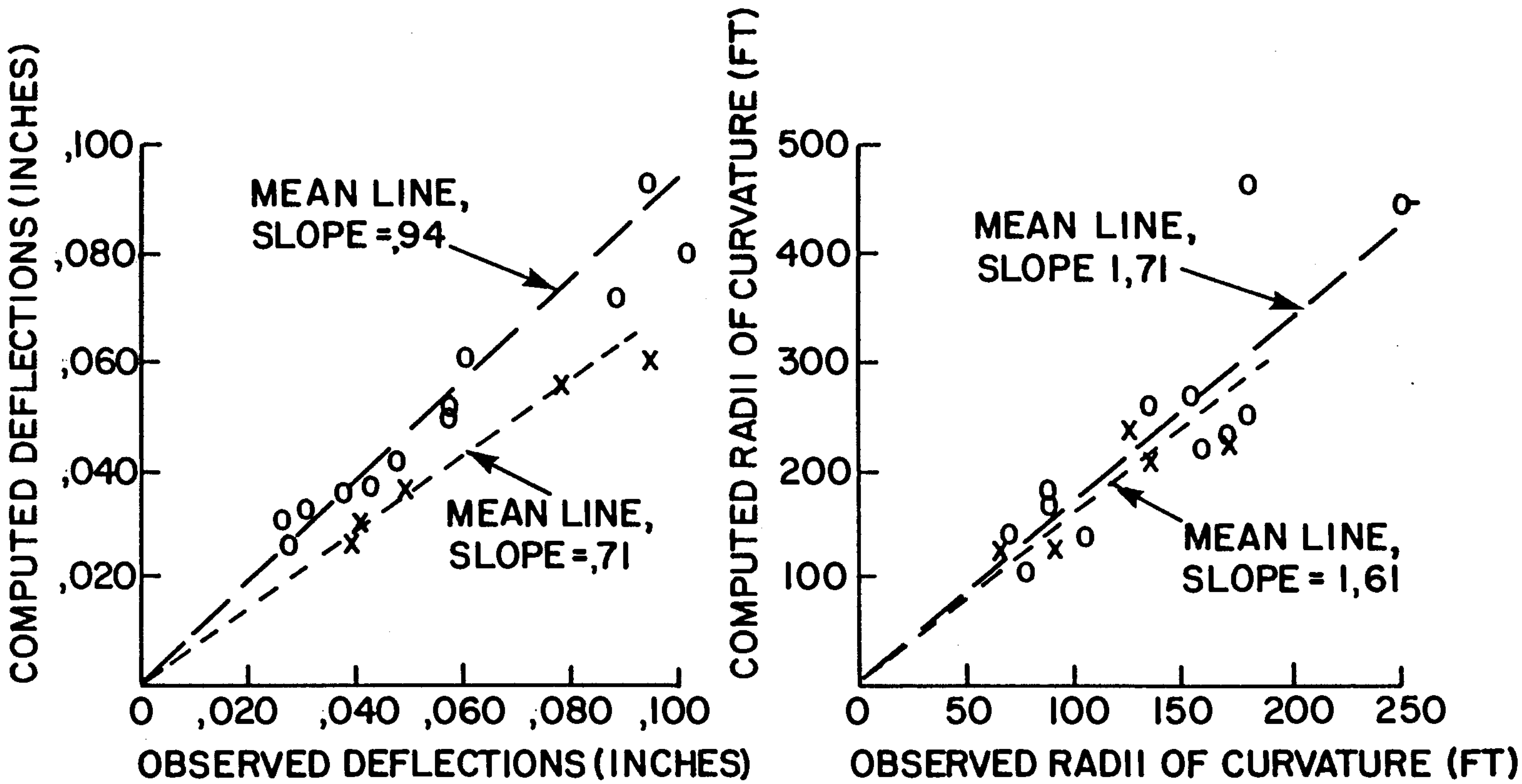
(b) Equivalent subgrade thickness layer

(c) Equivalent unbound thickness layer

(d) Bituminous layer

FIGURE 4.8

COMPONENTS OF A SURFACE DEFLECTION
(Snaith, et al., 1980)



—O— MEASUREMENTS MADE BETWEEN DUAL WHEELS COMPUTATIONS FOR SINGLE LOAD OF SAME AREA

—X— MEASUREMENTS AND COMPUTATIONS FOR SINGLE WHEEL

FIGURE 4.9

COMPARISON OF OBSERVED AND COMPUTED DEFLECTIONS AND RADII OF CURVATURE (Dehlen, 1962 b)

$$H_e = 0,9 \sum_{i=1}^{L-1} h_i \left[\frac{E_i}{E_s} \right]^{\frac{1}{3}}$$

where h_i = thickness of layer i in meter

E_i = elastic modulus of layer i in N/m^2

E_s = elastic modulus of the subgrade in N/m^2

L = number of layers.

Using the deflection basin parameter, surface curvature index (SCI), relationships between the SCI and versus H_e are determined as shown for various subgrade moduli (E_s) in Figure 4.10. This is for a three-layered pavement structure. The subgrade modulus is also calculated directly from the falling weight deflectometer (FWD) deflection basin as follows:

$$\log E_s = 9,87 - \log \delta_r$$

where r = 2 meters from the loading centre and δ_r is measured in mm. (The load force is 50 kN, loading time is 0,02 second).

Elastic moduli of the various layers can thus be back-calculated, if the layer thicknesses are known, using a dure similar to that described by Snaith et al. (1980), and using the general moduli relationship between base and subgrade and the Shell nomograph procedure as described by Koole (1979).

4.2 Methods using at least three deflection basin parameters

For a more effective use of the whole measured deflection basin in the determination of effective elastic moduli, at least the same number of deflections should be used as the number of layers in the pavement structure. This general approach is described in a FHWA report (1984). Normally a maximum of five deflection basin measurements extending radially away from the centre of loading are used. In Figure 4.11 a four-layered elastic system is illustrated. When a load of known intensity is applied over a known area, deflections are created at some distance from the centre of the loaded area. It is normally

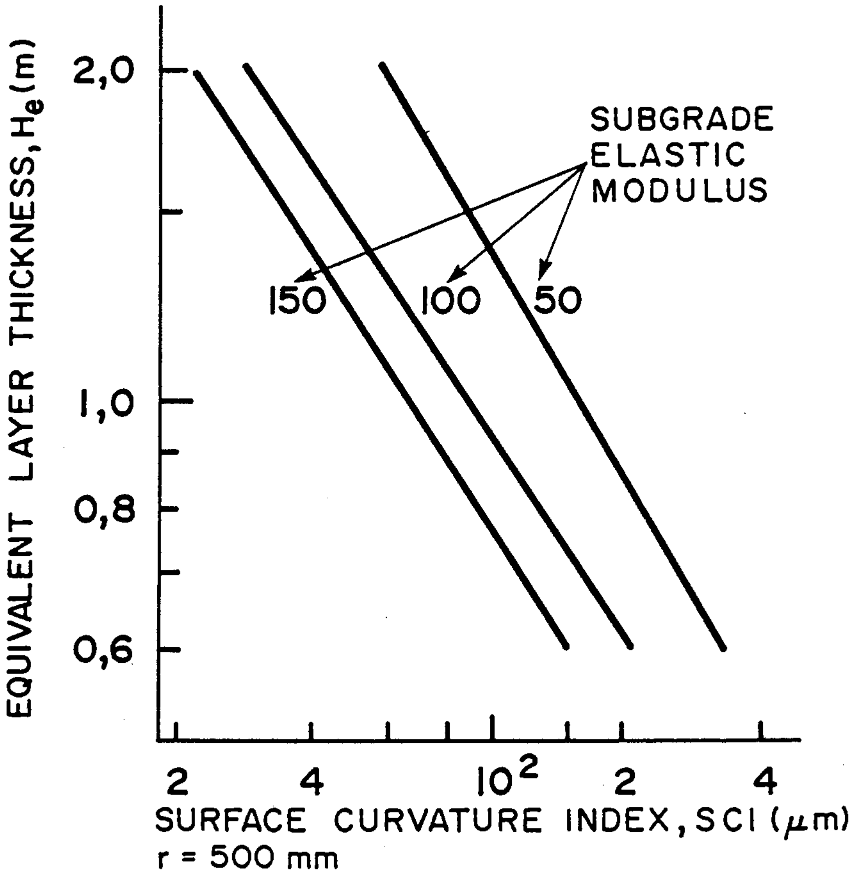


FIGURE 4.10
RELATION BETWEEN THE SURFACE CURVATURE INDEX (SCI) AND THE EQUIVALENT LAYER THICKNESS (H_e)
(Molenaar, 1983)

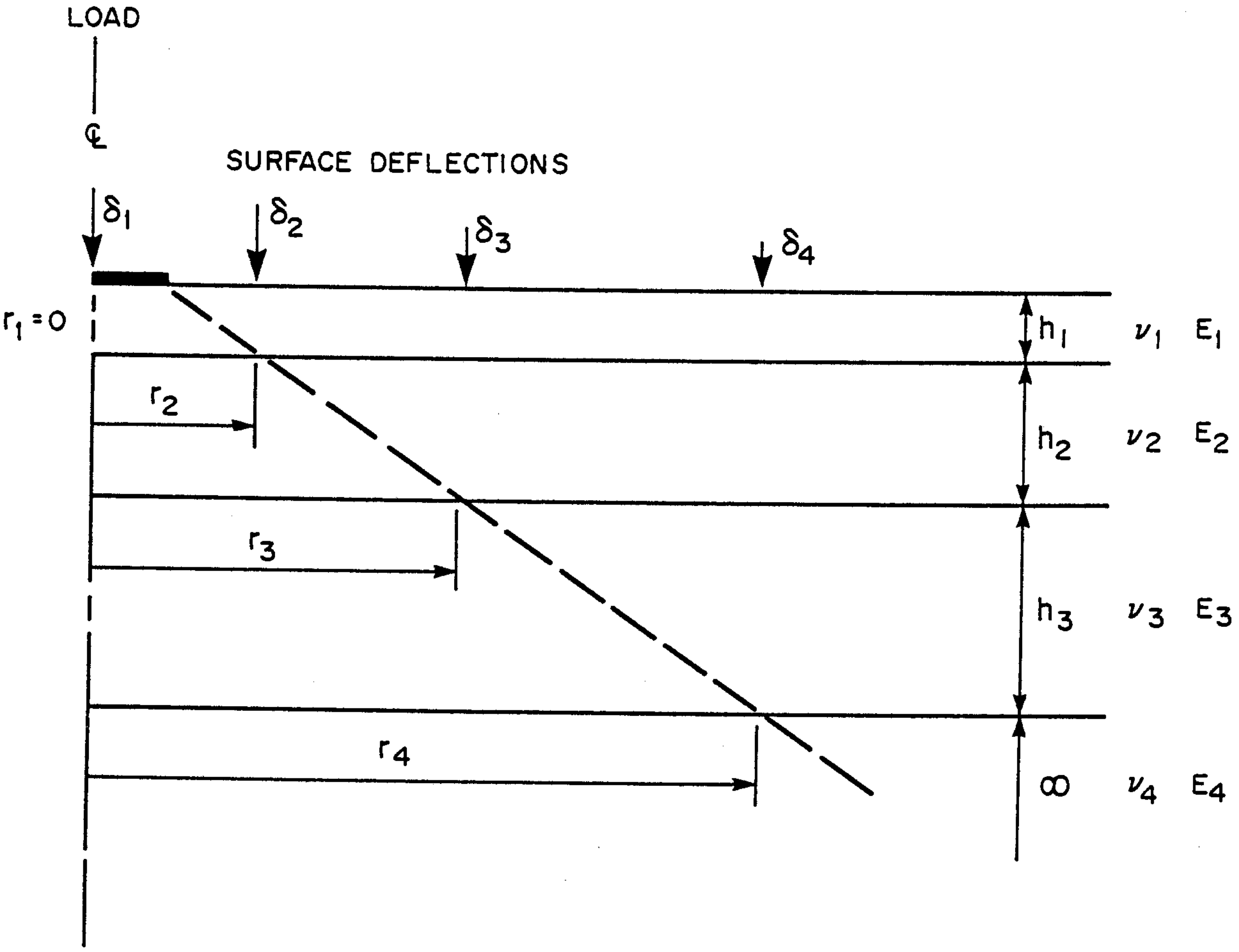


FIGURE 4.11

FOUR LAYER ELASTIC REPRESENTATION OF A PAVEMENT SYSTEM (FHWA, 1984) assumed that the load is distributed through the pavement system by a truncated cone (as shown by the dashed line in Figure 4.11).

Based on this concept, the deflection δ_4 at a distance r_4 from the centre of the load is due to the "elastic" compression of layer 4 since layers 1,2 and 3 are outside the influence cone created by the load. Likewise, the deflection δ_3, at distance r_3 is due to the compression of layers 3 and 4; the deflection at distance r_2 is due to compression in layers 2,3 and 4 and the deflection δ_1, is due to compression in all layers. Thus by back-calculating one should work radially inwards from δ_4 at r_4 towards the centre of loading in matching calculated deflections with measured deflections. This will result in the determination of effective elastic moduli of deep layers (subgrade) first and then of the other layers progressively upwards to the surface layer.

Using road rater data Kilareski et al. (1982) expressed this as follows:

$$\delta_1 \approx f1\ (E_1,\ E_2,\ E_3,\ E_4)$$
$$\delta_2 \approx f2\ (E_2,\ E_3,\ E_4)$$
$$\delta_3 \approx f3\ (E_3,\ E_4)$$
$$\delta_4 \approx f4\ (E_4)$$

This forms the basis of the successive approximation procedure. Estimate or seed values, as discussed previously, are assumed to start the iteration process. Using the BISAR computer program, the deflection values δ_1^1, δ_2^1, δ_3^1 and δ_4^1 corresponding to the assumed values of moduli are calculated. These calculated values are compared with the measured deflections, δ_1, δ_2, δ_3 and δ_4 at the measuring points as shown in Figure 4.11.

The correction method of the effective elastic moduli E_i is expressed as follows:

$$E_i \text{ new} = {}^1E_i \text{ old } \frac{(\delta_i^1 + \delta_i / \delta_i)}{2}$$

where i = 1 to 4.

The corrections are made to the subgrade first (E_4 and δ_4^1, δ_4 values) and a new deflection value is determined after which E_3 is adjusted and then E_2 and lastly E_1 by calculating the new corresponding deflection value (δ_i^1) each time. This completes one iteration after which the process is repeated until the following criterion is met:

$$\text{Error}_i = \left[\frac{\delta_i - \delta_i^1}{\delta_i^1}\right] *100$$

where Error_1 = 5 per cent allowed for δ_1^1 and Error_2 and Error_3
and Error_4 = 1 per cent allowed for δ_2^1 δ_3^1 and δ_4^1.

This is all done by an interactive computer program developed by Anani (1979).

With regard to the uniqueness of solutions, Kilareski et al. (1982), state that unique values of E_4 can be determined from δ_4 calculations and similarly unique values of E_3 from δ_3 calculations. In order to ensure unique values of E_2 and E_1 from δ_2^1 and δ_1^1 calculations, a certain range of E_1/E_2 ratios was assumed initially. This value was chosen as 0,7 as determined from laboratory resilient modulus testing on core samples.

MODCOMP I is a computer program developed by Irwin (1981) to interpret the moduli of elasticity of pavement layers from surface deflection data. The program can handle up to eight layers in the pavement using the CHEVRON n-layer code. Up to six surface deflections measured on the deflection basin are used. Measurements with vibration or impact devices are preferred in order to exclude plastic creep associated with Benkelman beam measurements. The principle of this analysis procedure is quoted as follows:

"At large radial distances the surface deflections are primarily the result of deformation in the deeper layers. Thus, the magnitude of the moduli of shallow layers has very little influence on the surface deflections of the pavement at large distances from the load." The back- calculation procedure is then the same as that described by Kilareski et al. (1982).

Seed values or estimate values of elastic moduli are used as input. An interesting feature of this approach is that the deflection basin measurements are first interpolated with a curve-fitting subroutine (CRVFIT). This is done so that there are measured deflection values, corresponding radially to a 38-degree cone intersecting the pavement layer interfaces as shown in Figure 4.12. No reason for the choice of the 38-degree cone is given. Adjustments to the modulus of the layer underneath the relevant interface are made using the following equation:

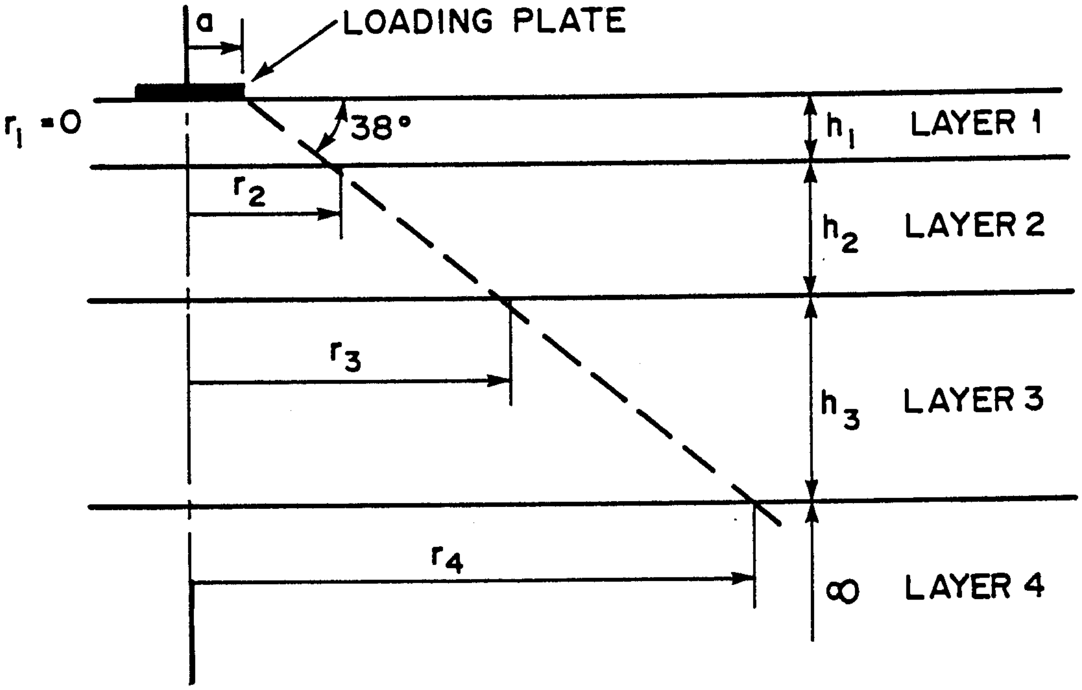
$$E_{new} = E_{seed}\left(1+2\left[\frac{\delta_i^1 + \delta_i}{\delta_i}\right]\right) \quad ?$$

where δ_1^1 = calculated deflection at the i-th radius

δ_i = measured deflection at the i-th radius.

The factor 2 is used to accelerate the iteration process, but E_{new} is not allowed to become negative. Using the E_{new} value (all other values unchanged) the deflection at that point is recalculated. A linear interpolation is then made in log-log space so that a value of the modulus for the layer, $E_{interp.}$ is obtained, based upon the measured deflection. From sensitivity analysis data, Irwin (1981) suggests that the tolerance level for computation purposes in MODCOMP I be five to ten times lower than the standard error of the measurement system ($\pm$ 0,02 mm).

The principle of modulus-at-depth as a function of the distance from the loaded area of the deflection basin is also the basis of the method used by Patterson and van Vuuren (1974). The use of the surface deflection basin is seen as a first method of determining effective elastic moduli. Deflection basin measurements were obtained with a modified Benkelman beam with a



CONE/LAYER INTERFACE INTERSECTIONS DEFINE RADII FOR CALCULATIONS

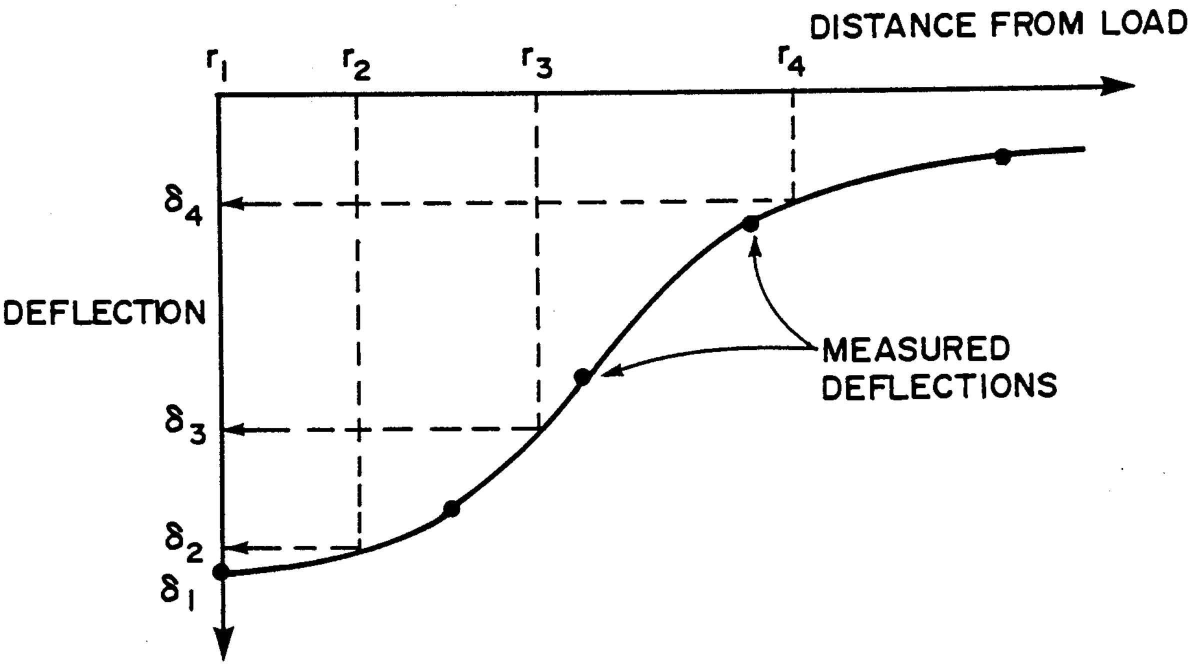


FIGURE 4.12
DEFLECTION BASIN SHOWING MEASURED DEFLECTIONS AND CRVFIT SUBROUTINE INTERPOLATIONS (Irwin, 1981)

deflection transducer for a "continuous" output. Layer thicknesses and material estimates were used as input in a trial and error process using the CHEVRON and ELSYM programs. The same basic approach as described by Irwin (1981) and Kilareski et al. (1982) was followed to determine and fix effective elastic moduli values by using deflection basin values contracting radially. Figure 4.13 shows a typical result of matching measured and calculated deflection basins and derived effective elastic moduli.

Husain and George (1985) use the CHEVRON program to do deflection matching. The measured deflection basin of either Dynaflect orfalling weight deflectometer (FWD) can be used. The deflection equation is inverted by a non-linear pattern search technique to determine the values of the layer moduli that would best fit the observed surface deflections. The elastic modulus of the asphalt concrete layer is corrected for temperature and the base and subbase moduli are corrected for stress dependency. The algorithm used, starts from the surface layer and proceeds to the subgrade. A general gradient technique is used in order to minimize the sum of squared errors.

The principle of deflection at distance from load applied being a function of material characteristics also forms the basis of the model used by Marchionna et al. (1985). In general the model is as follows;

$$\delta_i = f(CM_{1,1} \ldots, CM_{K,J}, \ldots, S_1 \ldots, S_K, F)$$

where

δ_i = deflection relative to the point located at distance r_i

$CM_{K,J}$ = Jth mechanical characteristic of the material relative to the Kth layer

S_K = thickness of the Kth layer

F = load applied

The falling weight deflectometer (FWD) is used with seven measuring positions and an elastic four layered pavement

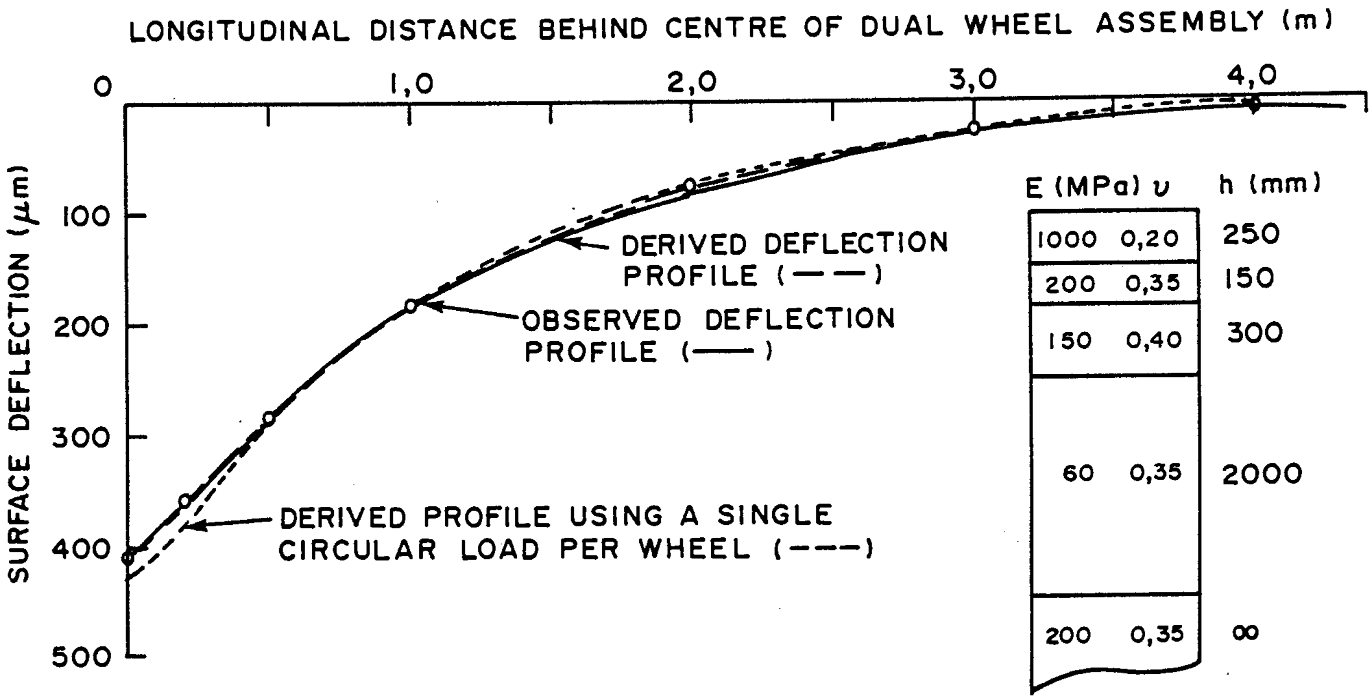


FIGURE 4.13

SIMULATION OF LONGITUDINAL SURFACE DEFLECTION INFLUENCE LINE AND DERIVED MODULI

(Patterson and van Vuuren, 1974)

structure is used. Bound or treated materials are linear elastic in this model, but granular bases and subbases and subgrade layers are treated as non-linear elastic. The analysis is carried out with the method of finite elements and the Non-linear Structural Analysis Program (NONSAP), developed at the University of California, is used. Various subroutines were added in order to use a technique of multiple regressions to find the expression of the δ_i functions that best interpreted the data available and met the conditions described.

The method of equivalent thicknesses described by Odemark (1949) forms the basis of the procedure described by Ullidtz and Peattie (1982). Surface deflections are measured with the FWD at the centre of loading (δ_o) and a minimum of two other positions (δ_1, and δ_2). It is specified that δ_2 be measured at a distance (radially from the load centre) of at least five times the load area radius (a); it must also be more than the thickness of the equivalent layer thickness (H_e). The latter is calculated in accordance with the description by Koole (1979). He gives the reason for the distance specification is that "at large distances the surface deflections are approximately inversely proportional to the modulus of the subgrade".

Layer thicknesses are determined by coring and estimated values of elastic moduli are assigned to each layer. Values of surface deflections are calculated by using computer programs like BISTRO or CHEVRON, or Finite element method programs. Ullidtz and Peattie (1980) suggest the use of programmable calculators using the equivalent layer thickness approach.

The equivalent surface modulus E_o (the modulus of the half-space that would give the same surface deflection as the multi-layer structure) is calculated as follows at corresponding distance r_2 where δ_2 is measured :

$$E_o = \frac{(1-\nu^2)\ p\ a^2)}{r_2\ \delta_2}$$

where the symbols are as previously described by Snaith et al. (1980) for the two-layered pavement system. For the prescribed value of r_2 the equivalent surface modulus (E_0) is approximately equal to the subgrade modulus. The subgrade estimate modulus value is adjusted and the value δ_2^1 is calculated and compared with the measured value δ_2.

The stiffness of the asphalt layer is determined using the ratio of the deflections δ_0^1. This ratio is highly dependent on the stiffness of the upper layer. If the calculated value of the ratio is less than the measured value, the estimated value of the stiffness of the asphalt layer must be increased because a high ratio corresponds with a high stiffness in the asphalt layer. The stiffness of the intermediate layer (normally granular) is determined from δ_0. Stubstad and Harris (1984) state that the latter is the least accurate value.

Laboratory-derived ratios of E1/E2 and E2/E3 are used as tolerances or guides. Some of these relationships used by Shell and described by Koole (1979) are also used. In the later versions the program ELMOD on the HP85 microcomputer are used to analyse a four-layered pavement structure. In this program higher precision was reached if the following conditions were met:

(a) The structure should contain only one stiff layer; if there are more they should be combined into one layer for the purpose of structural evaluation
(b) Moduli should decrease with depth
(c) The thickness of the upper (stiff) layer (h_1) should be greater than half the radius of the loading plate (a/2)
(d) When testing is done near a joint or a large crack or on gravel roads, the structure should be treated as a two-layer system.

Non-linearity of materials was also accommodated for by using a mainframe computer program ISSEM4 which can model a four-layer pavement. Ullidtz (1982), however, gives guidelines on how to accommodate stress softening (nonlinear) subgrade layers by constructing diagrams based on calculations done with finite element computer programmes. This was done for a two-layer system but formed the basis of the iteration process to determine moduli values for the subgrade.

Adjustments were made for temperature, time of loading and other environmental influences throughout the year. These were built in as standard features of the ELMOD program described above. In conclusion on this method Epps and Hicks (1982) state that: "The method proposed by Ullidtz and Peattie (1980) comes closest to a "universal" technique, but required the use of a computer. As the capabilities of small computers increase and as their use becomes more widespread, this requirement will be less and less of a deterrent to its use."

Stubstad and Corner (1983) made use of this latter method using the FWD to predict damage potential on Alaskan highways during spring thaw. The similarity with stabilized subbases is illustrated by their statement; "... it was immediately noticed that the same tendency toward virtually no deflection at large distances from the FWD load was occurring during the early spring of 1982. Such a phenomenon can only occur if the underlying layers have a high stiffness or modulus of elasticity. It was thus decided to use layered elastic theory to determine the effects of thaw depth and other material characteristics on the seven FWD deflections."

Bush (1980) developed an evaluation procedure for light aircraft pavements based on a layered elastic model. The CHEVDEF program used was developed to determine the set of modulus values that provide the best fit between a measured deflection basin and a computed deflection basin when given an initial estimate of the modulus values, a range of modulus values and a set of measured deflections. Deflections were measured with the Model 2008 Road Rater. Two force levels were used (5 000 and 7 000 lb) in order to predict the non-linear stress-dependent behaviour of the subgrade material. Results from the CHEVDEF program gave the relationship for the deviator stress and the modulus for the subgrade materials.

Thompson and Hoffman (1983) used the ILLI-PAVE computer programme to develope deflection basin algorithms for conventional flexible pavements and full depth asphalt concrete pavements. The effect of non-linearity of the subgrade of such three layered pavement structures is taken into consideration.

Tam (1985) use the same approach as Kilareski, et al. (1982) to calculate effective elastic moduli by relating the deeper layers to the outer defelctions. Thus by fixing effective elastic moduli from the bottom upwards, deflections are fitted radially inwards. Tam (1985) however make use of the BISAR or BISTRO computer programs rather than CHEVRON. The reason is illustrated in Figure 4.14 where the calculated deflection basins for the specific pavement structure and loading conditions are shown as calculated by BISTRO and CHEVRON. It is clear that the CHEVRON program has a slight discrepancy in the area between the edge of the loaded wheel and + 400 mm from the centre of loading. This tendency was confirmed by Taute et al. (1981) on stiffer pavement types, too. The discrepancy is more pronounced on lighter flexible pavement.

Before the iteration procedure is applied with the typical linear elastic approach, Tam (1985) takes the non-linearity of the subgrade into consideration. This is done by subdividing the subgrade into rather thick layers (0,6 to 1,0 m). The effective elastic modulus of each layer is then calculated as follows:

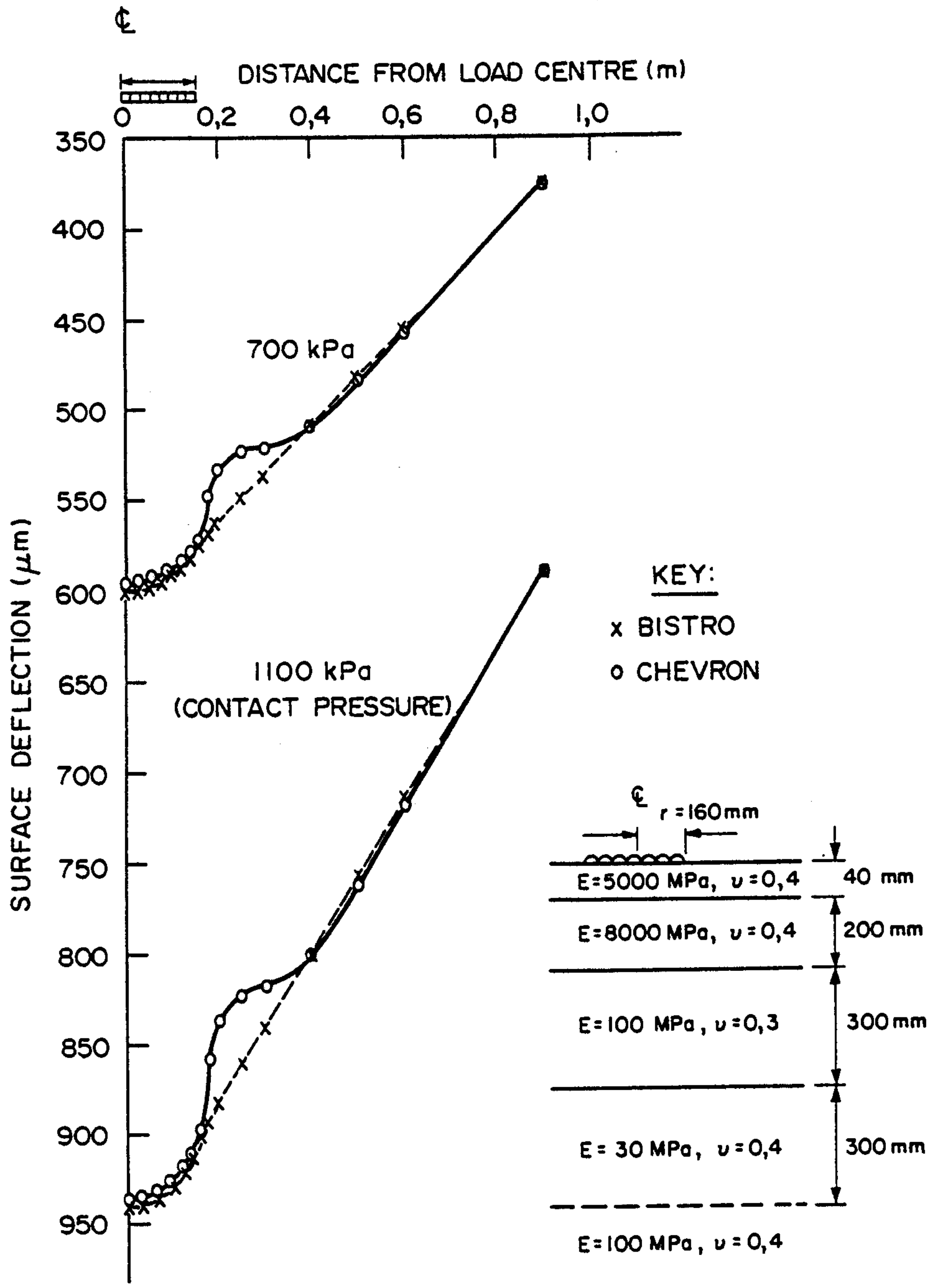


FIGURE 4.14

COMPARISON OF DEFLECTIONS CALCULATED BY BISTRO AND CHEVRON PROGRAMS

(Tam, 1985)

$$E = A \left| \frac{P_o^1}{q} \right|^B$$

where E is the elastic modulus (MPa)

P_o^1 is the initial effective everburden stress (MPa)

q is the deviator stress due to applied load (MPa)

A is the amplification factor (MPa)

B is the nonlinear power coefficient.

The seed values of A and B are selected as 50 MPa and 0,2. By using the deflection measured at a distance of 1,2 m from the load centre, A is first adjusted and then B. When all the subgrade layers have been fixed, the normal iteration procedure, as described earlier, starts.

5 CONCLUSIONS AND RECOMMENDATIONS

Estimate values of effective elastic moduli, based on laboratory and field observations, form the basis of the majority of back calculation procedures discussed. The mechanistic procedure used in South Africa forms a excellent basis for these estimate values of effective moduli as discussed. It is believed that by extending the field observations, in particular the HVS tests, this initial estimation procedure can be refined even further.

The analysis procedures have been described at length and for this reason only recommendations related to the South African position will be made. The linear elastic layered computer programs currently in use in South Africa are capable of back-calculating effective elastic moduli values from deflection basin data. Normally there is a maximum of five layers, which require at least five deflection measurements extending radially away from the centre of loading. Various researchers have suggested that the basic relationship of deflection measured radially from the load centre is a function of the various individual layers or combinations thereof. Some have even suggested an angle of intersection of the interfaces of layers in depth to determine optimal deflection measuring points radially. It is clear from the methods discussed that the re-iterative back-calculation procedure should work radially inwards to match calculated deflections with measured deflections. This would result in the determination of effective elastic moduli for the surface layer. It is evident that this approach focuses on the ideal linear elastic condition and that factors like non-linearity, nonuniformity and stress dependence are mostly ignored. Finite element method computer programs are becoming more readily available and can be used; non-linear models can also be investigated. It is suggested that correlation studies should be done for each type of pavement in order to establish these relationships. Methods to accommodate the non-linearity of the subgrade with the linear elastic computer models available exist and can be investigated.

In none of the methods discussed was much emphasis placed on cementitious bases and subbases and their analysis procedures. It is believed that enough information from HVS tests is available to analyse these pavements accurately with linear elastic layered computer programs. This would include, for example, information on the various stages of cracking of these layers. This also stresses the need for the analysis procedures to be related to the pavement category or class and behaviour state.
As was suggested in Chapter 3 the vast amount of information gained from RSD and MDD results makes it possible to calculate effective elastic moduli by various means for various pavement structures, loading conditions, moisture conditions and equivalent axle repetitions. This may lead to extensive regression analyses done in order to relate deflection basin parameters for the suggested pavement classes to the structural conditions related to structural life and behavioural states.

Serious consideration and analysis should be given to incorporating methods based on Odemark's (1949) equivalent layer thickness theory. It has proved to be a method that can satisfy mechanistic design method criteria. In order to establish this, extensive regression analyses need to be done with the above-mentioned available data.