DEFLECTION CHARACTERISTICS FOR FLEXIBLE ROAD AND AIPORT PAVEMENTS IN KENYA

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ABSTRACT

As traffic traverses a flexible pavement the axle loads induce a downward deflection of the pavement surface. The pavement however, bounces back as soon as the load passes a section. This downward deflection is measured by tracing the profile of the surface behind loaded wheels of a vehicle moving at creep speed. The magnitude of this deflection has been related to the strength of the pavement and has therefore been used for the assessment of pavement structural condition. This paper reports findings of deflection measurements on low and high volume road pavements and airport pavements in Kenya.

Keywords: Flexible, Pavements Deflections

1. INTRODUCTION

1.1 Geometry of the Deflected Profile

Several parameters have been picked in the deflected profile of a flexible pavement upon passage of a test vehicle. The parameters include maximum deflection (d_o) and radius of curvature (R). Analyses of deflected profiles have shown Equation 1 as the relation of deflection and radius of curvature (Roads Department, 1988 and Mwea, 2001). It is seen that after the measurement of d_x and corresponding values of x, the value of the radius of curvature may be obtained.

$$d_x = (2Rd_o^2) / (2Rd_o+x) \qquad \text{(Eq.1)}$$

Where:

- ***d***

$\boldsymbol{d_x}$ is the deflection at a distance x (m) from the loaded point (0.01mm units)
$\boldsymbol{d_o}$ is the maximum deflection, usually at the loaded point (0.01mm units)
R is the radius of curvature (m)

1.2 Pavement Structural Assessment from Deflection Measurements

The Roads Department (1988) has developed the concept of equivalent modulus (Eq) from Burmisters two-layer analysis. The expression for defining equivalent modulus is given in Equation 2. The various variables in Equation 2 are determined by Equations 3 through 9.

$$Eq = (10^{\alpha-1}\, E1\, (R/ d_o)^{\alpha/2})*10^2 \qquad \text{(Eq.2)}$$

$$\alpha= 1/(1+\log (E_1/1018)) \qquad \text{(Eq.3)}$$

$$E_1 = ((R/0.056)* (d_o /58000)^{A}))^{(1/B)}*10^2 \qquad \text{(Eq.4)}$$

$$A = (1-X)/(1-Y) \qquad \text{(Eq.5)}$$

$$X = 0.86 \log h – 0.474 \qquad \text{(Eq.6)}$$

$$Y = 0.493 \log h - 0.71 \qquad \text{(Eq.7)}$$

$$B = 1\text{-}A \qquad \text{(Eq.8)}$$

$$(E_1/E_2)^{(x-y)} = Rd_0/3248 \qquad \text{(Eq.9)}$$

Where:

Eq is the equivalent modulus of existing pavement (kN/m^2)
E_1 is the elastic modulus of the upper layer (kN/m^2)
E_2 is the elastic modulus of lower infinite layer (kN/m^2)
h is the thickness of top layer: i.e. existing pavement (mm)

From the above it is observed that when the value of the thickness of the pavement structure (h) and the maximum deflection d_o are known then Eq may be determined. A value of $150*10^3$ kN/m^2 is interpreted to mean failure and the need for an overlay. Long-term measurements by Gichaga (1979) and Murunga (1983) established that deflections increased with age and pavement air temperatures. Additionally the deflections for the slow lanes were always larger than those for the fast lanes.

2. DEFLECTION MEASUREMENTS ON THE FLEXIBLE ROAD PAVEMENTS

Gichaga (1979), Murunga (1983) and Atibu (1986) established eight test sites on the high volume roads for long term monitoring and evaluations. Two test sites along the low volume road along (Gatura – Mataara road) were later added to the testing program (Mwea, 2001). The construction details for these test sites are shown on the Table 1. The table shows these historical sites, namely ES1 through ES10 with an exception of ES8, which was not visited during this research. Table 2 shows the maintenance interventions during the study period.

Table 1. Construction details for the test sites.

Site	ConsDate	Location	Surfacing	Base	Subbase	Subgrade
ES1	1961	Fox cinema Nairobi-Thika (A2)	DSD	300mm Cement stab gravel	450mm Natural gravel	Natural gravel
ES2	1965	Langata road	DSD	225mm Hand Packed stone	225mm Natural gravel	Rock
ES3	1977	Airport-Lusaka Road (A107)	100mm AC	130mm GCS	200mm crushed stone	300mm Imp. subgrade
ES4	1977	Lusaka Airport Road (A107)	100mm AC	165mm GCS	165mm GCS	300mm Imp. subgrade
ES5	1977	Airport - Athi River Road	100mm AC	200mm Hand packed stone	200mm soft stone	300mm Imp. subgrade
ES6	1977	Airport - Athi River Road	135mm AC	200mm Hand packed stone	200mm soft stone	300mm Imp. subgrade
ES7	1981	Limuru Road (km27)	50mm AC	150mm DBM	250mm Natural gravel	Red coffee soil
ES9	1974	Gatura- Mataara (km 75)	DSD	130mm Crushed stone	100mm gravel	Murram
ES10	1974	Gatura- Mataara (km 80)	DSD	150mm Lateritic Gravel	-	Murram

Legend: Cons. Construction, Stab – Stabilised, Imp. Improved DSD – Double surface dressing, AC – Asphalt concrete

Table 2. Traffic and maintenance details for high volume test sites.

	Traffic		Maintenance	
Test Site	Years of service	CESAx10^6	Years of service	Type of maintenance
ES1	29	16.0	25	135mm AC overlay
ES2	24	4.5	22	Slurry Seal
ES3	12	12	8	35mm AC overlay
ES4	12	7.32	8	35mm AC overlay
ES5	12	28.57	8	35mm AC overlay
ES6	12	28.57	-	-
ES7	8	8.52	-	-

Legend: AC – Asphalt concrete, CESA – Cumulative equivalent standard axles

The equipment for measuring deflection was the benkleman beam developed in the USA and used in the AASHO road test between 1958 and 1960. In the Roads Department (1988) specifications, which was also used in this study, a two axle loaded truck was used for deflection measurements. The rear axle load was adjusted to 63.50 kN. Figure 1 shows the wheel and beam arrangements. The truck was driven at creep speed and halted at 10 centimetre intervals to 50 centimetres and then to 1500 centimetres. This enabled the deflection of the surface to be measured as the beam rotates about the pivot.

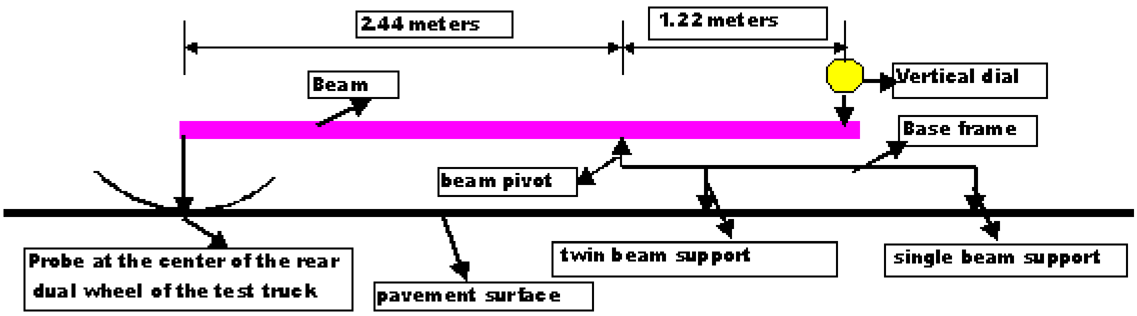


Figure 1. Benkleman beam arrangement.

For every test site twenty readings were done. The performance of a pavement section is associated with the weaker areas of section. These are the sections, which are unable to carry the axle loads and subsequently deform. The deflection (d) measured by the benkleman beam arrangement is associated with the structural performance of the pavement. If the average value of d were taken, then evaluation of the pavement using this average value would result in using a deflection whose value is exceeded in half of tested sections. This would result in failure of the pavement structure. To counter this a characteristic deflection D_{90} is obtained from Equation 10. To reduce the probability of failure to ten percent, f is taken as 1.3 (Roads Department 1988).

$$D_{90} = d' + fsd' \qquad \text{(Eq.10)}$$

Where:

D_{90} is the characteristic deflection
d' is the mean deflection and f is a factor,
sd' is the sample standard deviation

3. DEFLECTION MEASUREMENTS FOR AIRPORT PAVEMENTS

Deflection measurements were conducted on an airport located approximately at the equator and 37^0 east. Due to the wide pavements, the runway and the taxiway were divided into 7.5 meter wide "roadways". These roadways were used for the deflection measurements. This gave rise to sixteen test points at each cross section of the runway on four roadways and eight test points on the taxiway on two roadways. The test points are shown on Figure 2.

The surfacing of the airport pavements consisted of and initial 50 mm asphalt concrete constructed in 1974. An overlay of 40 mm asphalt concrete was applied in 1986. The base and subbase for the pavement structure consisted of 200mm graded crushed stone on 200 mm gravel respectively. The pavement structure rested on 500 mm of selected subgrade. At the time testing the pavement had been on service for 24years and signs of distress had manifested at various sections of the pavement. The major signs of distress consisted of extensive cracking, water pooling and large longitudinal and lateral deformations. This made flight manoeuvres at the airport dangerous.

Pavement deflection was measured using the same equipment as that for the highway test sites. At each test point five readings were made for construction of the deflection profile. Eighty readings were therefore made at each cross section of the runway. The taxiway had forty readings on each cross section readings. In both runway and taxiway test cross sections were spaced at 100m intervals over the four kilometre lengths. In total the runway had 37 test sections with 2960 readings while the taxiway had 38 test sections with 1520 readings. With this high volume of data, statistical analysis was justified to arrive at pavement deflection characteristics.

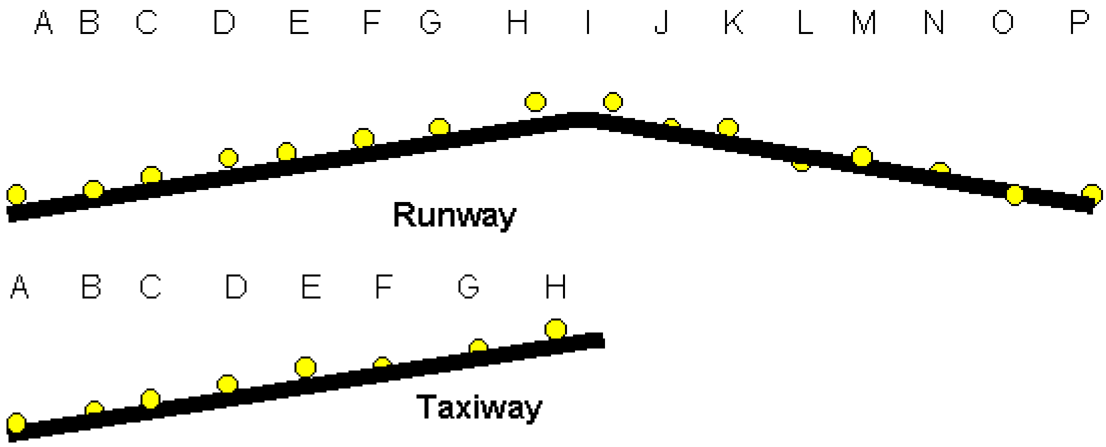


Figure 2. Test points for deflection tests.

Table 3 shows test results for four roadways of the runway and two roadways of the taxiway. As for the highway test results the characteristics deflection (D_{90}) is defined as the mean deflection plus 1.3 standard deviation. The equivalent modulus (Eq) was calculated on the basis of the deflections and radius of curvature (R). The table shows longitudinal and lateral variation of the three parameters at half kilometre intervals.

Table 3. Deflection radius of curvature and equivalent modulus.

Runway						
X-Section (m)	Thickness (mm)	Parameter	Section of Roadway			
			A-D	E-H	1-L	M-P
200 to 700	150	D_{90} (.01 mm)	53	44	47	58
		R (m)	159	379	253	213
		Eq $(kN/m^2)*10^2$	4008	18460	8738	7436
700 to 1100	175	D_{90}	43	40	35	46
		R	288	290	56	244
		Eq	8361	8190	1646	6559
1200 to 1600	175	D_{90}	45	33	42	45
		R	302	438	262	305
		Eq	9356	14985	7066	9504
1700 to 2100	150	D_{90}	36	27	34	33
		R	311	490	330	341
		Eq	10594	20815	11614	12068
2200 to 2600	150	D_{90}	42	36	41	41
		R	202	250	221	206
		Eq	5352	7039	6206	5481
2700 to 3100	150	D_{90}	58	49	55	56
		R	167	252	274	183
		Eq	5968	12346	17037	7034
3200 to 3600	125	D_{90}	58	49	55	56
		R	167	252	274	183
		Eq	5965	12346	17037	7034
3700 to 3800	125	D_{90}	57	25	20	53
		R	62	560	818	239
		Eq	1077	35567	64794	12027

Taxiway				
X–Section (M)	Thickness (mm)	Parameter	Section of Roadway	
			A-D	E-H
325 to 800	225	D_{90} (.01 mm)	68	56
		R (m)	76	150
		Eq $(kN/m^2)*10^2$	1205	2698
825 to 1300	225	D_{90}	49	38
		R	193	416
		Eq	3697	10249
1325 to 1800	225	D_{90}	39	42
		R	319	174
		Eq	7042	3151
1825 to 2300	225	D_{90}	35	76
		R	343	487
		Eq	7569	16649
2325 to 2800	225	D_{90}	33	34
		R	196	332
		Eq	3563	7177
2825 to 3300 1300	225	D_{90}	55	48
		R	114	144
		Eq	2326	3178
3325 to 3800	150	D_{90}	55	47
		R	68	109
		Eq	1179	2058
3825 to 4000	150	D_{90}	43	32
		R	172	359
		Eq	4102	12767

4. ANALYSIS AND DISCUSSION OF ROAD PAVEMENT TEST RESULTS

4.1 Deflection Variation with Time

Historical deflection data has been assembled from past studies by Gichaga (1979), Murunga (1983), Atibu (1986) and Wa-Kyendo (1991). Figure 3a shows the variation of deflection with load for the high volume test sites while Figure 3b shows deflection verse time for the low volume sites. The main features from the deflection history for each site can be summarised as follows:

4.1.1 High volume roads

ES1: After an initial high deflection of 50×10^{-2} mm, the deflections remained bound between 20 and 35 x 10^{-2} mm up-to a loading of 8.51×10^{6} ESA. The mean deflection then rose to 39×10^{-2} mm. This deflection reduced to 31×10^{-2} mm before a 135mm overlay was placed. The deflection then reduced to 24×10^{-2} mm remaining low and at the final loading of 15.71×10^{6} ESA the deflection had reduced further to 16×10^{-2} mm.

ES2: Deflection measurements started at low loading of 0.34×10^{6} ESA. The deflection remained in the mid forties as the loading increased to 2.56×10^{6} ESA when a slurry seal was then applied and deflection reduced from 45 to 31×10^{-2}mm.

ES3 and ES4: The two pavements consist of 330mm combined cohesionless subbase and base. The surfacing consisted of 100mm-asphalt concrete. However, the low loading on ES4 kept the deflections consistently low over the years. Thus at five and half years the mean deflection for ES3 was 53mm, while that for ES4 was 24mm. It was however noted that despite the large deflection ES4 also remained in good condition. Upon application of overlay at a loading of 2.49×10^{6} ESA, the deflection reduced from 29 to 21×10^{-2}mm.

ES5 and ES56: The mean deflection at ES5 during the elastic working stage of the pavement was 41×10^{-2} mm. This rose to 62×10^{-2}mm before strengthening at a load of 18.31×10^{6} ESA. The overlay intervention reduced the deflection to 39×10^{-2}mm. This deflection continued to decrease and at a loading of 31.70×10^{6} ESA after thirteen years of service the deflection reduced to 29×10^{-2}mm. Prior to getting loaded to 19×10^{6} ESA, the pavement at ES6 was performing well with deflection ranging from21 to 31×10^{-2} mm when the deflection rose to 62×10^{-2} mm. ES5 deflections were on the average 61% higher than those of ES6. The difference in deflections was however found to lessen with age. The higher deflections are attributable to the effect of low-speed heavy axle-loads on the steeper ES5 test site. This was also manifested in the rutting which was higher on the steep ES5 test site.

ES7: This site was still in its elastic stage at 9.08×10^{6} ESA loading and eight and half year's service. The deflections fell between 34 and 48×10^{-2}mm.

4.1.2 Low volume roads

ES9 and ES10 ES9 and ES10 test sites are on low volume tea roads. The testing on these two sites was separated by seven months. Within that period the deflection at ES9 decreased from 60 to 43×10^{-2} mm while at ES10 it increased from 49 to 80×10^{-2} mm. The two sites have lower strength pavements compared to the high volume sites. Hence their deflections were higher. The sites did not manifest surface defects. This was despite their seventeen years of service. The low traffic combined with the ability of surfacing to remain intact preserved the pavement surface.

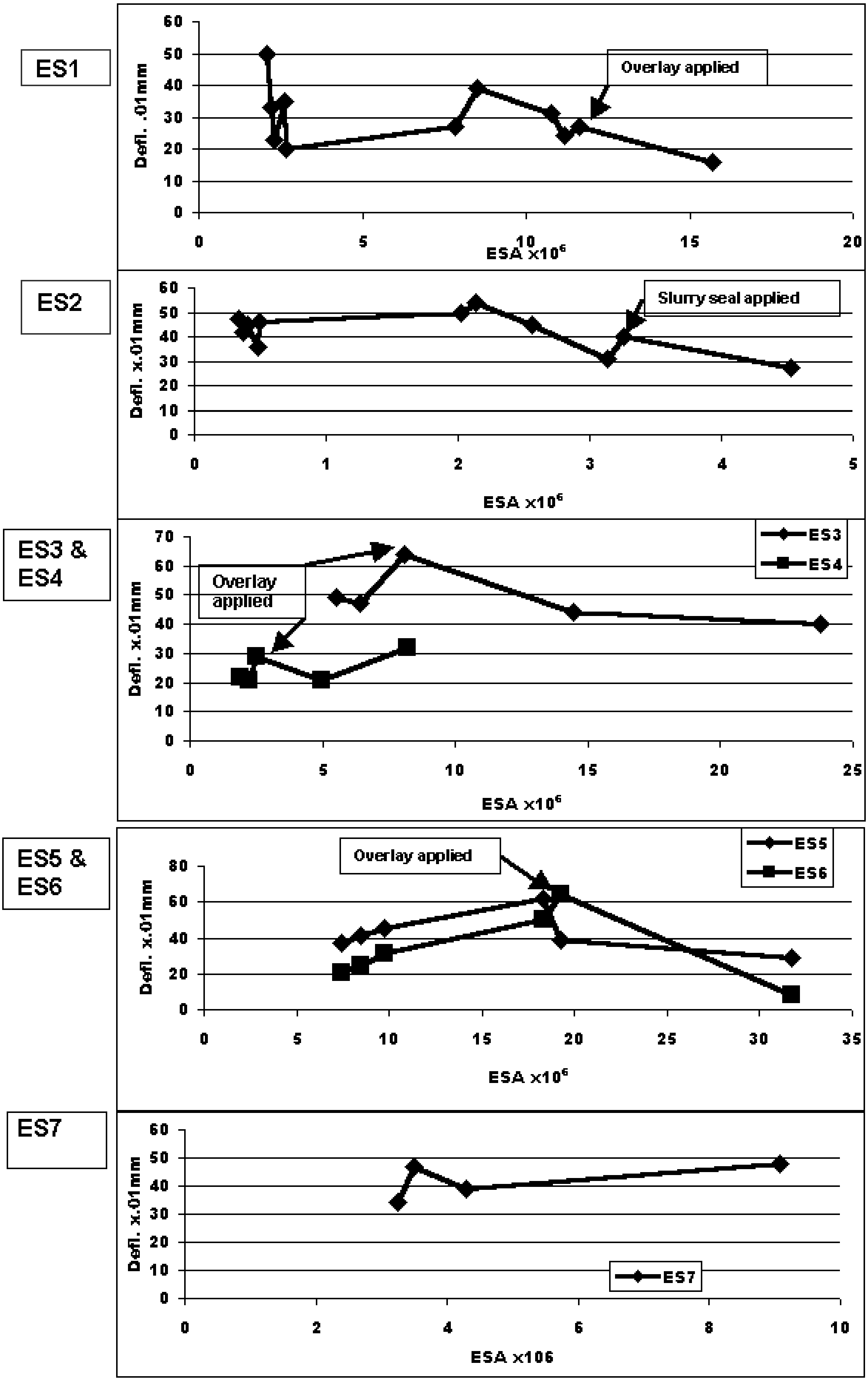


Figure 3a. Deflection vs equivalent standard axles for high volume roads.

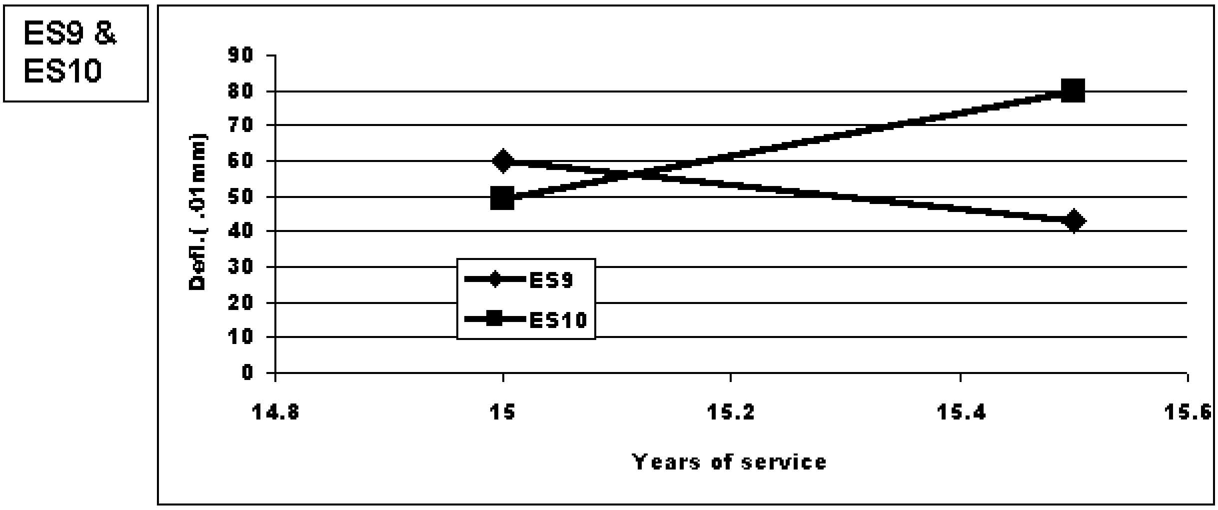


Figure 3b. Deflection vs years of service for low volume roads.

4.2 Longitudinal Deflection Profiles

Figure 4 shows half-deflection longitudinal profiles for the various test sites. The profiles have been compared where the test sites share common pavement structure features below:

4.2.1 High volume roads

ES1 and ES2: These were old sites with a service life of 28 and 27 years respectively. ES1 had carried 15.71x106 ESA while ES2 had carried 4.53x10^6 ESA at the time of deflection measurements. The deflection at ES2 was however found to be larger with a smaller radius of curvature. The 135mm-overlay intervention on ES1 was therefore more effective than the slurry seal in ES2 in improving the structural integrity of the pavement.

ES3 and ES4: These two sites are on dual carriage roads. They were constructed in 1977 at the same time. Over the years of service ES3 had carried more load than ES4. Though maintenance intervention was done at the same time, the deflection at ES3 was bigger than that at ES4. Correspondingly the radius of curvature at ES3 is smaller than that at ES4.

ES5 and ES6: These two sites have the same traffic and had carried the heaviest traffic at the time of testing of all the sites. The equivalent standard axles carried at each of these sites of 31.70x10^6 ESA placed the traffic with the highest category in the Kenyan design manual. This was class T1 (Roads Department 1987). However the ES5 was located on steep section and the slow lane deflection profile was found to be deeper with smaller radius of curvature.

ES7: This site is on a natural subgrade. Historical data showed that at the time of testing the mean deflection was within the elastic phase magnitudes recorded at the beginning of monitoring. However the deflection profile for the right hand lane was found to be generally deeper than that of the left lane.

4.2.2 Low volume roads

ES9 and ES10: These two sites are on low volume tea roads. Their deflection profiles when compared to those of high volume roads would appear to be large and therefore the sites required a periodic maintenance intervention. However, visual examination of the site showed that the surface condition was good with no cracks and no permanent deformation. The deflection criteria would therefore appear to be unsuitable performance indicator for the low volume roads.

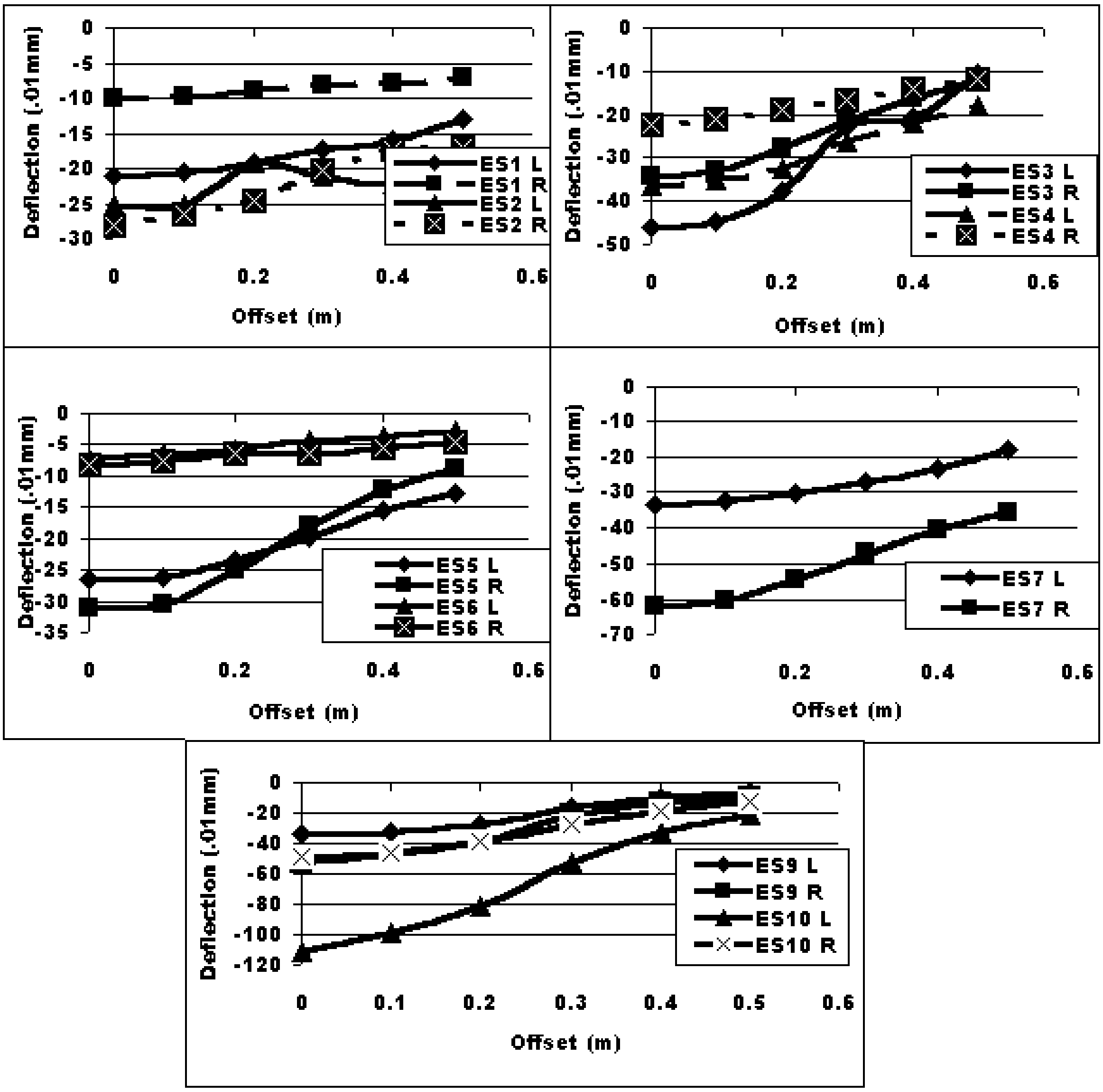


Figure 4. Deflection profiles for the test sites.

5. GENERAL OBSERVATIONS

The interpretation and comparison of deflection data is complex due to a large number of variables that affect deflection. Thus, besides the above presentation of historical performance, deflection and by extension the structural strength of the pavement depends on initial design capacity and routine maintenance. The maintenance includes minor resealing and patching, which seals off the cracks and the incidence of weakening the base and underlying pavement structures. The deflection is also affected by improvement of drainage, which again increases strength and reduces deflection (Mwea 2001).

An overall deflection comparison for all the sites is difficult. The difficulty in interpretation can be seen in Figure 5, which shows the relation of deflection with loading for all the sites. The Figure shows no noticeable trend. However comparison of deflections for individual sites immediately before resealing or overlaying immediately after resealing or overlaying shows an immediate drop in deflection (Figure 6). The order of drop varies but an average of 35% was found.

A regression analyses of R vs. D_{90} was carried out for all the test sites (Equation 11). From the coefficient of determination (0.74) and the low analysis of variance (ANOVA) significant. F of .000 it can be seen that the R is significantly related to the D_{90}. This was despite the large variation in the pavement structures.

$R = 596949D_{90}^{-1.95}$ (r^2=0.74) (Eq.11)

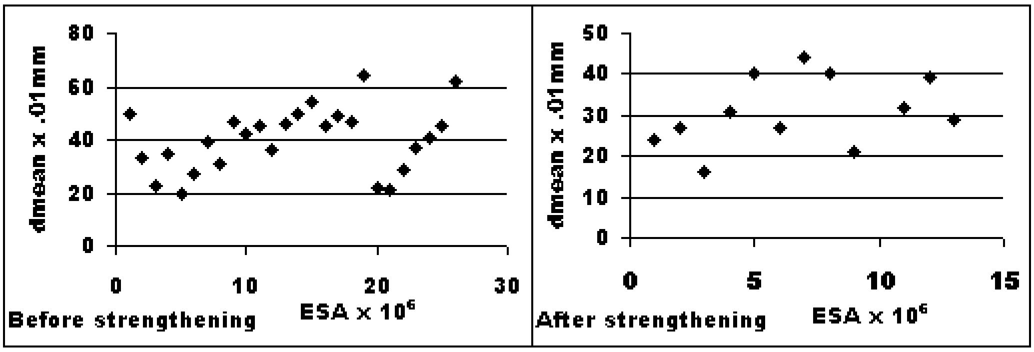
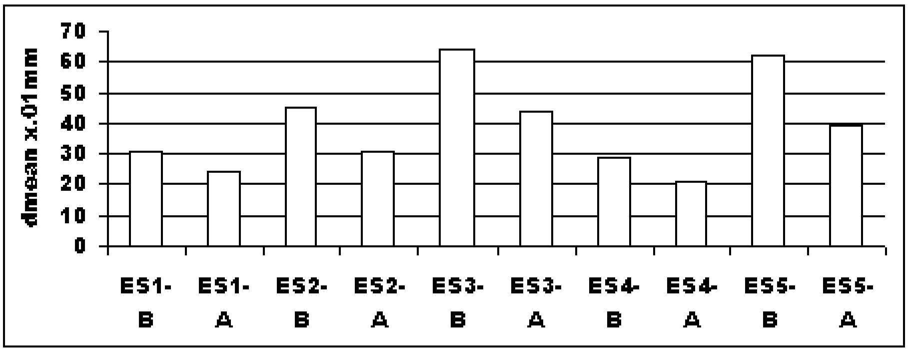


Figure 5. Deflection vs load scatter for all the test sites.



Legend A – After strengthening, B – Before strengthening

Figure 6. Comparison of deflection before and after strengthening.

The deflections at the various offset positions on the left lane were regressed with those of the right lane. The regression assumed that deflection for the left lane could be calculated from Equation 12. The values of ANOVA significant F were below 0.05 showing a significant relationship.

$d_l = A + B*d_r$ (Eq.12)

Where:

$\boldsymbol{d_l}$ is the deflection for the left lane

$\boldsymbol{d_r}$ is the deflection for the right lane

Statistical results for test sites are shown on Table 4. Gichaga (1979) found ES1 deflections for the left and right lane were well related. He likened the behaviour to that of a concrete slab. He attributed this to the cementing action of the stabilised murram, which was used in the base. Results from the Table 4 shows that good linear correlations were also obtained in pavements with cohesionless bases. The binding effect combined with the inter-particle friction of the pavement materials make the deflections on the left lane correlate well with those on the right lane for all types of pavement structures.

Based on the deflection and pavement layer thickness for the various test sites the Eq has been calculated and are presented in Figure 7. The Roads Department (1988) thresholds for well performing pavements have been presented alongside the field values to enable comparison of the performance and also check on the structural state of the sections.

Table 4. Left lane and right lane deflection relationships.

Cohesionless bases								Cohesive bases			
High volume sites				Low volume sites				High volume sites			
Site	A	B	R^2	Site	A	B	r^2	Site	A	B	r^2
ES2	18.23	0.50	0.76	ES9	5.89	0.72	0.97	ES1	-9.96	2.48	0.94
ES3	9.91	1.07	0.93	ES10	-11.0	2.27	0.97	ES7	8.44	0.50	0.95
ES4	-2.78	1.99	0.97								
ES5	10.62	0.59	0.98								
ES6	-0.49	0.68	0.83								

Save for test site ES5 on the high volume roads the other sites were found to be well performing. The test site ES5 is on the hilly section of the Jomo Kenyatta Airport-Athi River town road. The slow climbing road was found to have an equivalent modulus below the $150*10^3$ KN/m^2 threshold. In addition high rutting and cracking was noted.

The deflection data For ES9 and ES10 shows that the pavements are weak. However visual inspection showed that the sections were performing well. The low traffic on this road, estimated at below 500 average annual daily traffic (AADT) had not damaged the pavement structure over the years of service. Deflection as tool for pavement evaluation on these sections can therefore be misleading without an accompanying conditional survey

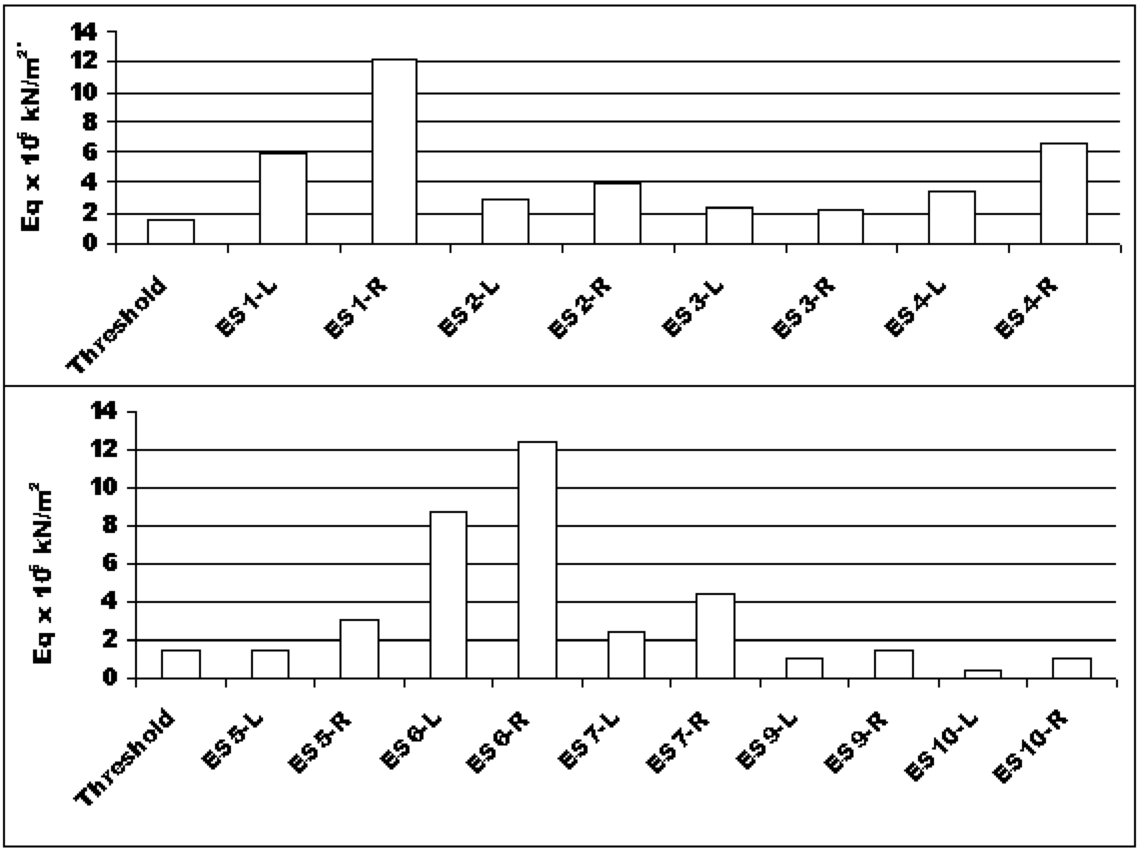


Figure 7. Performance of the experimental sites.

6. AIPORT PAVEMENT TEST RESULTS

6.1 Runway Deflection Test Results

It was observed that the mean characteristic deflection (D_{90}) ranged from $20x10^{-2}$ mm to $58x10^{-2}$ mm with an average of $43x10^{-2}$ mm. The overall scatter of the deflection is shown in Figure 8. The radius of curvature ranged from 56m to 818m. The overall scatter the radius of curvature is also shown on Figure 8.

The structural integrity of the pavement was assessed by estimation of the equivalent modulus (Eq) based on deflection data and pavement layer depths, confirmed by drilling. Figure 9 shows that a negligible percentage was less than 150×10^3 kN/m^2, the threshold at which the structural strength of flexible pavements are critical. Consequently the Eq criteria also passed the pavement as structurally sound

In general the two parameters, which define the deflection profile show a normal distribution. The large variation shown by the large scatter, are indicative of the variation of strength. The scatter can also be seen from the regression of the radius of curvature against the characteristic deflection. For a uniform pavement the deflection would be basically similar and the R versus D_{90} regression would yield a high coefficient of determination. Instead the overall regression analysis of R versus D_{90} gave Equation 13 with a coefficient of determination (r^2) of 0.35 and a low analyses of variance (ANOVA) significant, F of .003.

$$R = 21.4 \times 10^3 D_{90}^{-1.20} \qquad \text{(Eq.13)}$$

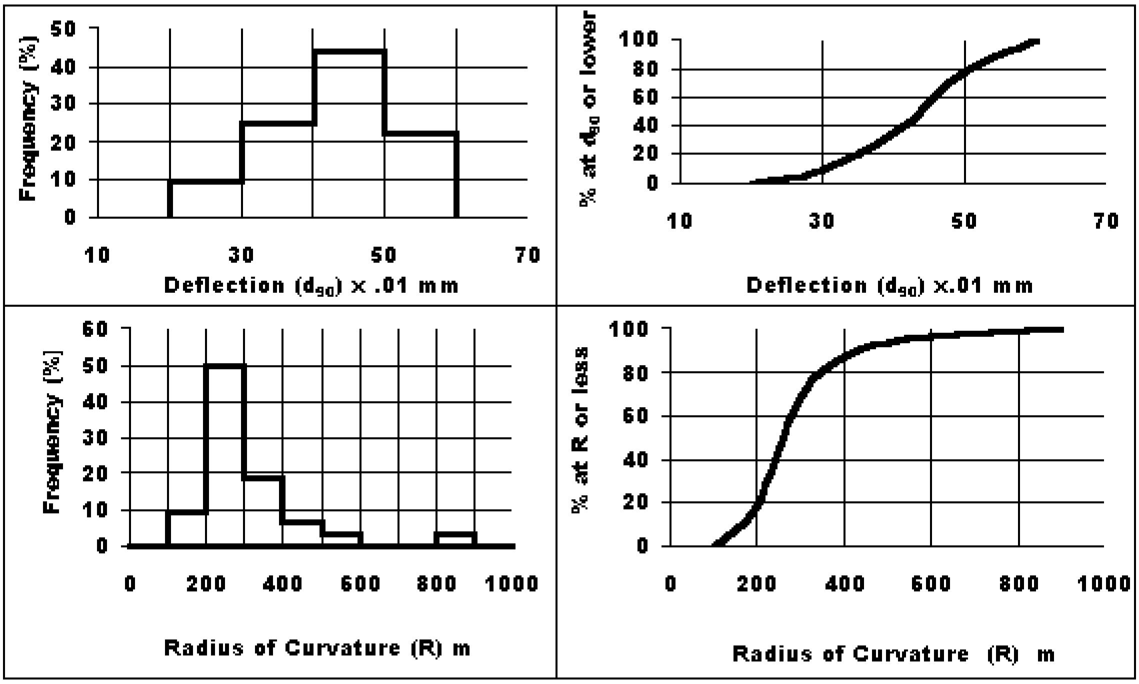


Figure 8. Deflection and radius scatter for the runway.

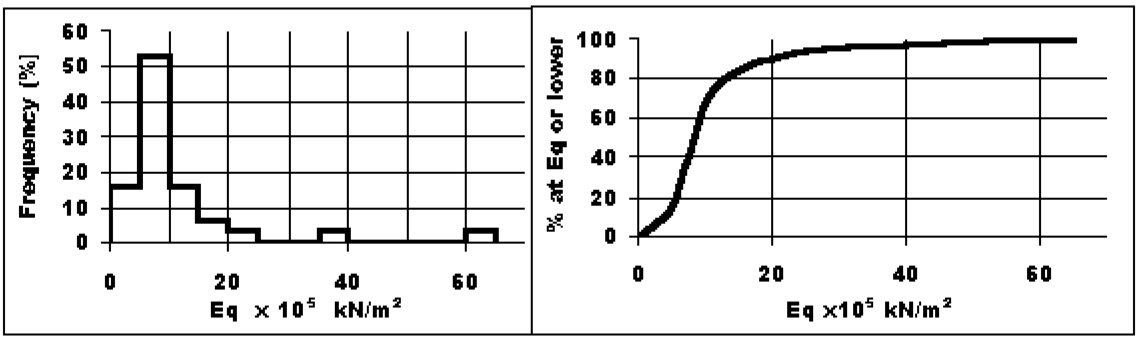


Figure 9. Equivalent modulus (Eq) scatter for the runway.

6.2 Taxiway Deflection Test Results

The equivalent modulus computation gave the distribution shown on Figure 10. The figure 3 shows that only about 12.5% of the taxiway pavement had an equivalent modulus less than the $150*10^3$kN/m^2 threshold. The equivalent modulus criteria therefore passed the pavement as structurally sound.

The overall scatter of deflection and radius of curvature values is shown on Figure 11. A large scatter of the deflection profile parameters can be seen. A regression of radius of curvature and characteristic deflection gave a coefficient of determination of 0.19 and a high ANOVA significant F of 0.091 (Equation 14)

$$R = 9.573x10^3\ D_{90}^{-1.02} \qquad \text{(Eq.14)}$$

A comparison of radius of curvature versus deflection for the runway and the taxiway is shown on Figure 12 showing similar trends

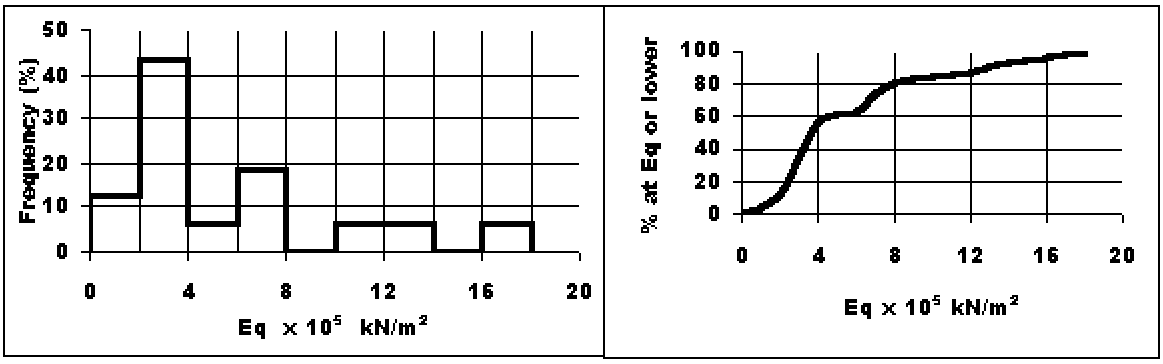


Figure 10. Equivalent modulus (Eq) scatter for the taxiway.

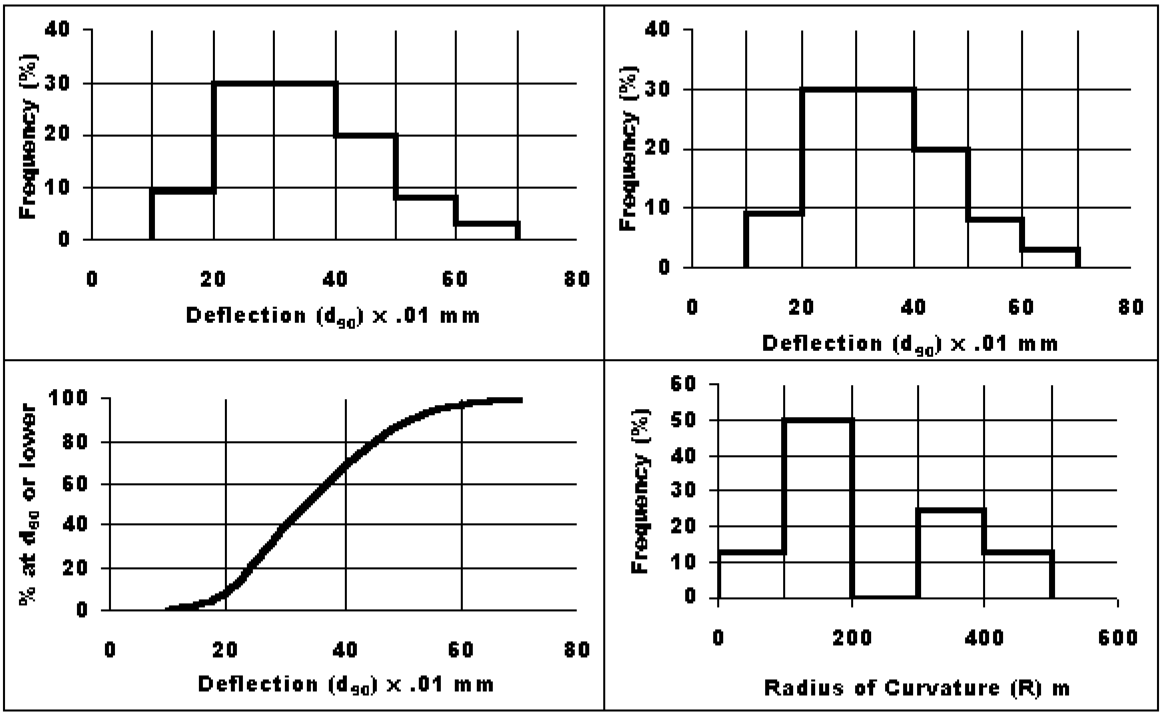


Figure 11. Deflection and radius of curvature scatter for the taxiway.

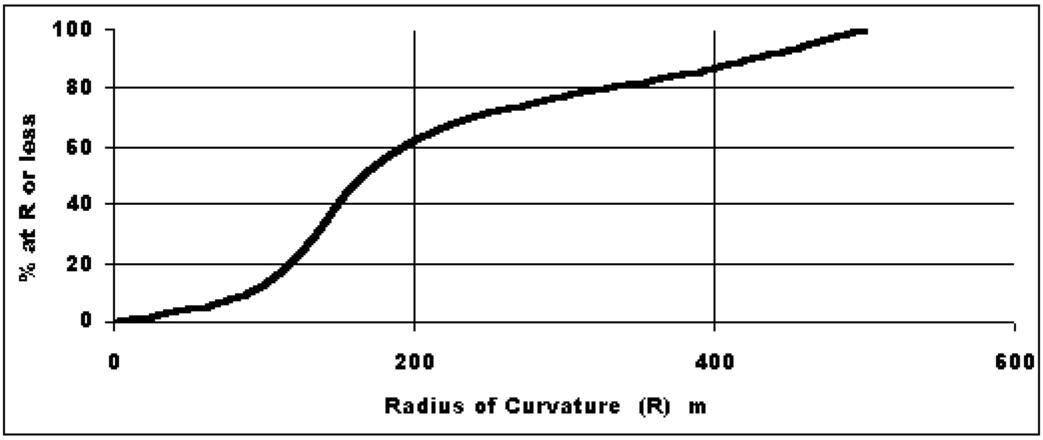


Figure 12. Radius of curvature vs deflection for the runway and taxiway.

7. CONCLUSIONS

The Equivalent Modulus was found to give a good indication of pavement condition for high volume roads, while visual condition indices are more appropriate for low volume roads

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