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DEFLECTION CRITERIA FOR FLEXIBLE PAVEMENTS

by

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ABSTRACT

The long-term performance of a flexible pavement is related to the magnitude of the transient deflections which occur under traffic. Measurements made over the past 15 years on experimental pavements have shown that it is possible to relate the deflection measured under a 'standard' wheel-load to pavement life in terms of the cumulative commercial traffic carried.

Deflection criteria curves of this type have been developed for pavements with crushed stone, rolled asphalt, tarmacadam and cemented bases, under rolled asphalt surfacings. The data reported in this Paper have been collected from the Laboratory's full-scale pavement design experiments, and particularly from the Alconbury Hill experiment constructed in 1957. The deflection measurements have been made with the Deflection Beam.

The deflection criterion curves for pavements with crushed stone, rolled asphalt and coated macadam bases are identical within the limits of the experimental accuracy, but acceptable deflection levels are much lower on pavements with cemented bases.

The Paper discusses the way in which deflection criterion curves can be used to estimate future performance and future maintenance requirements for flexible pavements.

1. INTRODUCTION

The passage of a loaded wheel causes a road pavement to deflect transiently, the magnitude of the deflection depending on the stress-strain characteristics of the construction materials and the underlying soil.

The life of a pavement is dependent in a general sense on the magnitude of the transient strains and deformations generated by traffic. As a consequence it is possible to relate empirically the surface deflection of any particular form of pavement under a selected 'standard' wheel-load, with the probable future performance under road traffic. The development of such empirical relations must entail long-term observations made over the life-span of various forms of pavement construction. However the potential value to the road engineer in predicting future maintenance requirements, particularly under increasing traffic, is very considerable.

With this in mind the Road Research Laboratory, some fifteen years ago, developed a standard method of deflection measurement and embarked on a programme of deflection studies aimed at the production of criterion curves relating deflection with pavement performance. Most of the field measurements have been made on the Laboratory's full-scale pavement design experiments on which condition surveys are carried out at regular intervals for the purpose of producing design standards.

The work described in this Report is broadly divided into two parts. The first relates to the original full-scale road experiments constructed in the period 1950-55. The second is devoted to the Alconbury Hill experiment, constructed on Trunk Road A.1 in Huntingdonshire in 1957, and for which the complete deflection histories of the various sections have been studied.

2. THE TRANSIENT DEFLECTION OF PAVEMENTS UNDER LOAD

The transient deflection observed at the road surface when a wheel-load passes is the sum of component transient deformation in the various pavement layers and in the soil foundation. These in turn depend on the elastic properties of the materials involved, which may be influenced by temperature and other environmental factors.

The distribution of wheel-loads in the traffic carried by a normal road, whose strength is subject to the effects of a range of temperature and moisture conditions, generates a very wide spectrum of transient stresses and strains within the pavement. The heaviest wheel-loads in combination with extremes of temperature and adverse moisture conditions in the subgrade produce values of stress and strain which may approach ultimate values. Less damaging combinations will produce stress levels likely to contribute in some degree to failure by cumulative deformation and cracking under repeated traffic loading.

The early life of flexible pavements is characterised by rapid but limited permanent deformation. The normal result of this 'compaction' phase is to improve the stiffness and strength of the road to an extent which depends on the type of construction used. Pavements containing cemented materials are however exceptional as cracking may be initiated or developed during this phase. In long-lasting pavements, subsequent to the compaction phase, only a small proportion of traffic stresses contribute appreciably to final failure by deformation or cracking. However in virtually all such pavements the development of distress is a continuing if exceedingly slow and seasonal process.

As the degree of under-design increases the proportion of damaging stresses and strains increases and the development of distress is correspondingly more rapid until, in the case of inadequately designed or constructed roads the deterioration of the pavement associated with the initial compaction stage continues until the failure condition is reached. Visible evidence of failure is the development of rutting in the wheel-paths followed or accompanied by cracking and break-up of the road surface.

Thus the development of distress in a road leading ultimately to failure can be considered as a continuation of the development of irreversible strain in the road after a period of initial compaction, due to the repeated loading of one or more elements of the road above critical values of stress or strain. It is reasonable to expect, therefore, that the degree of distress developed in any road would be conditioned by the magnitude of the repeated transient strains generated by the traffic, and that measurement of these strains would show correlation with the subsequent performance of the road. Further, since transient strains are approximately proportional to wheel-load, a similar correlation would be expected with the strains associated with any selected wheel-load.

Various methods, such as plate loading tests, have been used to measure the total transient deflection of roads. Deflection can be most realistically and rapidly measured in conjunction with a wheel-load moving at creep speed, by the use of the Deflection Beam, a simple mechanical device first developed by Benkelman in America. Its development and method of operation in the United Kingdom have been described in an earlier Report[1]. Briefly the equipment consists of a slender pivoted beam which can be accommodated between the rear twin tyres of a loaded truck as shown in Plate 1. The maximum deflection is observed using a dial gauge as the wheel rolls past the tip of a beam. A wheel-load of 3175 kg (7,000 lb) applied by dual tyres of standardised design, spacing and inflation pressure, is used for testing.

Preliminary measurements with the Deflection Beam were concentrated on experimental roads having crushed stone bases of various thicknesses. These showed that areas where failure was developing gave greater deflections than other areas where the pavement was in sound condition. In this way it was possible to define a maximum value of deflection associated with areas of pavement, sound at the time the measurements were made. It was also demonstrated that, of the areas sound at the time the measurements were made, those having greater deflections subsequently showed signs of distress before the areas associated with smaller deflections suggesting that measurements made on roads in sound condition could be used to predict their future performance. The promise of this early work resulted in a programme of research designed to standardize the deflection test technique and to investigate systematically the relation between deflection and road performance.

3. DEFLECTION HISTORY OF PAVEMENTS

The deflection history of a pavement relates the deflection to the life of the road (in terms of the total amount of commercial traffic carried) and the condition, as determined principally by the permanent deformation and degree of cracking present in the wheel-paths.

3.1 Deflection measurements

The first deflection measurements are preferably made before the road is opened to traffic. Thereafter measurements are made twice-yearly in the early life followed in general by annual measurements in the later stages. Observations at 6-12 m intervals along the length of the road have been usual on experimental roads, but a wider spacing has been used on normal in-service roads.

Early measurements showed that it was difficult to obtain reliable measurements in hot summer conditions when the temperature within the road surfacing appreciably exceeded 30°C. They also demonstrated the importance of the temperature of the bituminous elements of a road in determining their stiffness and hence the measured deflection of the pavement as a whole. Research into the relationship between deflection and surfacing temperature, normally taken at a depth of 40 mm (1½ in), has enabled measurements taken on the road at different temperatures to be corrected to a standard temperature of 20°C and errors inherent in the procedure are minimised by restricting deflection studies to periods when the surfacing temperature is between 10° and 30°C.[2]

Spring, when temperatures most suitable for deflection measurements coincide with high water-table and therefore weak subgrade conditions, was selected as the preferred period for deflection studies. Where measurements are made twice-yearly they are carried out in spring and autumn.

3.2 Assessment of the condition of the pavement

In establishing a deflection history the condition of the road in the immediate area of the point of deflection measurement is classified into one of three major categories; sound, critical and failed. The critical classification indicates the need for pavement strengthening before failure takes place. In early work each category was sub-divided into two further groups. The six resulting classifications were associated with a) the extent of rutting or permanent deformation under a straight edge 1.8 m long and b) the amount of cracking of a persistent nature present. Slight cracking capable of being healed under hot weather conditions was not considered significant. A description of the classification in terms of these characteristics is given in Table 1.

TABLE 1

Classification of pavement conditions

Classification		Visible Evidence
Sound	1	No Cracking. No appreciable rutting
	2	No Cracking. Rutting less than 10 mm
Critical	1	No Cracking. Rutting between 10 mm and 20 mm
	2	Cracking confined to single crack in wheel path. Rutting less than 20 mm.
Failed	1	Cracking extending over the area of the wheel path and/or rutting greater than 20 mm.
	2	Dangerous to traffic. Disintegration of the road surface.

3.3 Traffic

It is generally accepted that structural damage to pavements is caused by heavy commercial traffic and public service vehicles. Traffic for design purposes should therefore be defined in terms of the number and magnitude of wheel-loads of this type using the road. However the scarcity of information on wheel-load spectra dictated in the early stages of this research programme that the cumulative total of commercial vehicles (sum in both directions) should be used to define traffic. This information can be deduced from the Ministry of Transport census data. In the later work, carried out at Alconbury Hill, results from an automatic weighbridge were available and traffic has also been expressed in terms of equivalent 8,200 kg (18,000 lb) standard axles, using the A.A.S.H.O. equivalence factors.[3]

3.4 Deflection criteria deduced from deflection history

Deflection histories are plotted with the deflection as vertical axis and cumulative commercial traffic as horizontal axis. The condition of the pavement in the immediate area of the deflection measurement is indicated by the use of an appropriate symbol for the point defining the deflection value. In this way it is possible to draw a curve through the points to separate the sound from the critical and failed conditions as indicated in Fig. 1. Such a curve has been termed a deflection criterion curve for the particular type of pavement examined. The curve is asymptotic to a limiting lower deflection value, when plotted with a linear traffic scale. If it could be assumed that the deflection of a pavement remained constant with time the deflection criterion curve, shown on Fig. 1, could be used to estimate the probable life, from a deflection measurement, and the limiting lower deflection value should be obtained on a pavement intended to have a very long life.

A change of deflection may however occur and under these conditions any estimate made in this way can only be approximate. Fig. 2 illustrates hypothetically the nature of changes which are possible in deflection studies. Two sets of measurements are shown made at different times in three areas of a pavement constructed to the same nominal specification. At area (1) the road was already failing when the first measurement was made, and more serious failure was apparent at the time of the second measurement; this is reflected in an increased deflection. Area (2) was sound initially, but was failing when the second series of tests was made. The decrease in deflection indicates that compaction of the pavement structure under traffic caused the deformation resulting in failure. In area (3) no failure has been observed but the increase in deflection suggests some deterioration within the pavement or its foundation, likely to lead to early failure.

On this evidence the straight-line deflection criterion curve shown can be drawn through the two critical deflection values revealed by the measurements. The assumption of a constant deflection level equal to the first measurement would for area (2) lead to a shorter estimated life than that actually observed, whilst for area (3) the reverse would be the case. Clearly two or more measurements at a point made after different intervals of time can be used in conjunction with a criterion curve to obtain increased accuracy in estimating life-expectancy.

It follows that the deflection criterion curve so far considered is 'immediate' in the sense that it is based on deflections obtained at the time critical conditions are developing in the pavement. It follows too that in using this criterion curve to estimate the future life of a pavement from a deflection value, the variation of deflection with life (in terms of traffic carried) must be taken into account. This matter is discussed in more detail later in this Report.

TABLE 2

Details of early full-scale pavement design experiments used for deflection studies

Site	Date of Construction	Commercial traffic in 1959 (V.P.D.)	Types of base	Types of surfacing	Soil Conditions
Boroughbridge Yorkshire (A1)	1950	2840	Crushed stone Tarmacadam	Rolled asphalt Bitumen macadam	Silty sand
	1955		Crushed stone Sand cement	Rolled asphalt Bitumen macadam	Sand
Cambridge Glos (A38)	1954	2590	Crushed stone Gravel Tarmacadam	Rolled asphalt	Medium and Heavy clay
Sapcote Junction, Leics. (A46)	1951	1690	Crushed stone Tarmacadam	Rolled asphalt	Medium clay
Loughton By-Pass, Bucks (A5)	1955	3970	Crushed stone	Rolled asphalt Bitumen macadam	Sandy and Heavy Clay

(6.)

4. THE EARLY TEST PROGRAMME

A programme of testing with the Deflection Beam, with the object of developing deflection criteria, was begun in 1956 on the full-scale road experiments then under observation by the Road Research Laboratory. Table 2 gives details of the experiments which were tested. The programme was interrupted in 1957 by the concentration of effort on the construction of the Alconbury Hill experiment, but continued thereafter. The major experiments of the group, those at Cambridge, Gloucestershire (A38) and north of Boroughbridge, Yorkshire (A1), were tested sufficiently regularly to develop their deflection histories while the remaining sites at Loughton (A5) and Sapcote Junction (A46) were examined only occasionally. These roads provided examples of a variety of types of pavement construction incorporating both asphalt and open-textured bitumen macadam surfacings laid on bases of crushed stone, open-textured macadam and sand-cement, but the deflection readings associated with the traffic compaction phase were missing as the roads were all open to traffic before the test programme began. Some of the results have already been reported elsewhere[4][5].

The main conclusions are discussed below in terms of base materials used.

4.1 Granular bases

At Cambridge a sequence of measurements over a number of years was made on sections with bases of crushed limestone, gravel and limestone scalpings constructed on a clay subgrade.[4] All tests were on the final rolled asphalt wearing course the laying of which had been delayed until several months after the opening of the road to traffic. Conforming to the expected pattern of behaviour, greater deflections were measured on the thinner sections, which also deteriorated to a critical condition before the thicker and stiffer ones. The first signs of damage always appeared in the near-side wheel path where the deflections were also generally higher than in the offside wheel path. The deflection history of the nearside wheel path of one group of three sections is shown in Fig. 3. Measurements were made on two transverse lines in each section (lines 1 and 2). There was little change in deflection values with time or traffic on the thicker sections having longer lives but more variation on the thinner section. A deflection criteria curve could be drawn from the results from these sections which, with only one exception, fitted the results from all other experimental sections at this site having granular bases reasonably well.

At Sapcote Junction sections identical with those described in Fig. 3, but on a stronger foundation, gave considerably lower deflections and performed much better. After a two-way cumulative flow of over 12 million commercial vehicles the thinnest section is still sound with deflections of between 25 and 67 mm x 10-2 (.010 and .027 in). A slight increase in deflection with age is detectable. The superior behaviour is explained by the C.B.R. value under these sections of 14 per cent (compared to 4 per cent at Cambridge) and also to the observable fact that the bend on which this experiment was constructed has the effect of spreading transversely, and therefore reducing the damaging power of the normal intense traffice loading in the wheel paths.

As all sections have remained sound the results can only indicate limits above which critical deflection values can be assumed to lie. Without making any allowance for the reduced damaging power caused by the bend it is interesting to note that the deflections at this site plotted in Fig. 4 lie beneath the criterion curve established at Cambridge.

At Boroughbridge measurements were made on sections with dry-stone bases under open-textured bitumen macadam and rolled asphalt wearing courses. Sections reconstructed less than a year before the first measurements were made, in addition to relatively old experimental construction, were tested. Additional data on the early behaviour of the latter sections were obtained by measurements taken in the lightly trafficked overtaking lane. While the older sections were founded on a silty-sand subgrade having a CBR value of 8 to 10 per cent the newer construction was built on a stronger imported sand fill with a minimum CBR of 20 per cent.

The results have been reported[5] and the deflection criteria derived are reproduced on Fig. 4. They indicate a criterion curve lying below that obtained for sections of similar type with a rolled asphalt wearing course at Cambridge. Differences in the type of subgrade at the two sites and the fact that at Boroughbridge, unlike Cambridge, the final surfacing was applied before the road was opened to traffic may be responsible for the different curves. Manual weighbridge censuses[6][7] at the two sites did however indicate a preponderance of heavier wheel-loads in the traffic at Boroughbridge so that any comparison based on a concept of damaging power instead of the cumulative total of commercial vehicles would bring the curves into closer agreement.

No appreciable changes in deflection with time were observed in a limited number of measurements which were made at any one point at Boroughbridge.

One set of measurements was carried out at Loughton By-Pass, which before the opening of the southern part of M1 in 1959 carried over 4,000 commercial vehicles per day, indicated a critical deflection of about 0.5 mm (0.020 in) after a two-way cumulative total of 5.5 x 10^6 commercial vehicles. These measurements are shown on Fig. 4. The sections tested, built on a heavy clay subgrade, had a two course rolled asphalt surfacing (90 mm, 3½ in), crushed stone base (300 mm, 12 in) and a sub-base of hoggin (130 mm, 5 in). This critical deflection agrees with that observed at Boroughbridge (which probably had a similarly damaging traffic spectrum) but is below that for the Cambridge site which had a less damaging traffic spectrum.

4.2 Tarmacadam bases

Sections with a 50 mm single-size tarmacadam base at Cambridge, Sapcote and Boroughbridge gave smaller deflections than were recorded on unbound bases of equal thickness reflecting the greater stiffness of the tar-coated materials. Little significant change of deflection with time was noted except on the two thinner bases (200 and 300 mm thick) at Sapcote whose deflection histories departed from the general pattern. After several years traffic deflections increased rapidly on these sections until they were substantially greater than those of the corresponding uncoated bases. An investigation of the section with 200 mm base after its failure showed that the base was completely stripped and stripping is now occurring in the thicker section.

Under an open textured bitumen macadam wearing course, tar-coated bases at Boroughbridge gave much higher deflections and poorer performance than identical bases under a rolled asphalt wearing course. These differences in both deflection and performance were too great to be explained by the stiffnesses of the two types of wearing course. The base and base course material under the relatively permeable macadam was found subsequently to be completely stripped and must have been so when the first deflections were made (in this case 6 years after the road was opened to traffic) as no further increases in deflections were observed.

The above results show how the relation between deflection and the subsequent behaviour of pavements whose deflection increases rapidly at some stage due to deterioration of the base complicates the problem of predicting performance from deflection measurements. They demonstrate that 'immediate' criteria are by themselves of limited value where unexpected changes in the road materials occur. However the experiments also indicate the value of establishing typical deflections for pavements of a given type and thickness on a given sub-grade which would enable marked deviation from normal values due to deterioration, such as stripping, which are not obvious from inspection of the road surface, to be detected and the pavement strengthened or reconstructed as appropriate.

4.3 Sand cement bases

Of the two sections at Boroughbridge of this type (added to the original experiment at a later stage), that with a rolled asphalt wearing course gave values of deflection 9 months after opening to traffic as little as 0.025 mm (0.001 in) but these increased considerably in the next two years. The development of rutting suggested that cracking of the base occurred at this time but the section remained basically sound, while under a bitumen macadam wearing course deflections of a broadly similar value led to failure of the pavement. This indicates that the behaviour of cement bound materials after they have begun to crack may be strongly influenced by the nature of the surfacing used on them.

These limited results indicated a complex relation between deflection and performance for this type of base, investigated in more detail in the later Alconbury Hill experiment.

5. THE ALCONBURY HILL EXPERIMENT

This major full-scale road experiment in flexible and concrete construction was built during the spring and summer of 1957 and opened to traffic at the beginning of November of that year. The aims and objectives of the experiment, have been described elsewhere.[8] In addition to providing valuable data on pavement design the flexible sections whose layout is given in Fig. 5 enabled the scope of the deflection programme to be widened to include modern road base materials such as rolled asphalt, lean concrete and 'wet-mix' slag not previously studied and also provided the first opportunity to study deflection histories of pavements from the time of their construction.

The thirty-three experimental flexible sections contain a wide range of types and thicknesses of base and surfacing materials laid in a range of total pavement thicknesses. Their performance can therefore be considered to be reasonably representative of flexible pavements generally and has enabled the general relevance of the definitions of pavement condition given in Table 1, originally derived mainly from the behaviour pavements with crushed stone bases, to be studied.

5.1 The deflection programme

5.1.1 Layout and sequence of measurements

A set of deflection measurements was made before the completed road was opened to traffic and then in each succeeding spring. Subsequent to 1963 measurements were also taken in the autumn. The positions chosen for testing were situated in the nearside and offside wheel-paths of the slow lane on the three lines of levelling studs used for observing changes in the transverse profile of the road. Two further points in each wheel-path, equally spaced between these three lines, were also used giving a total of ten points of measurement per section.

5.1.2 Temperature correction

The temperature susceptibility of a road, i.e. the degree to which the deflection changes with temperature, depends on the proportion of its stiffness which derives from road materials whose stiffness varies with temperature, (see Section 3.1). The experimental pavements at this site represent a wide range of temperature susceptibility. At one extreme is that of the pavement with an uncracked lean concrete base whose stiffness is virtually independent of temperature, and at the other, a pavement with a bituminous base where the stiffness of both surfacing and base are affected by temperature. Ideally a set of measurements should include observations taken at different temperatures but at substantially the same stage in the life of the road to enable a standard deflection at 20°C to be estimated from the deflection-temperature relationship thus established. These relations would also have to be re-examined at various stages in the deterioration of the pavement.

It did not prove possible to carry out the very extensive programme of testing such an approach required. However during the life of the experimental road measurements at selected points were made in the majority of the sections to establish the general trend of these relations.

As the experiment matured it was noted that the plot of deflection versus temperature for all measurements taken during the life of the pavement provided a valuable indicator of the operative deflection-temperature relations. The simplest relation was derived from pavements whose structural stiffness at constant temperature did not change appreciably with time. An example is given in Fig. 6 with the date when each measurement was made marked alongside the points. Although some scatter about a mean line is apparent, representing both real differences in deflection and experimental error, little overall change in time is discernible and all results can be corrected to the standard temperature by following the mean slope through the points to the 20°C intercept.

Fig. 7 shows the type of record obtained when real changes in deflection did occur. The points can still be grouped along lines corresponding to quasi-equilibrium conditions during the overall gradual weakening process represented by the upward progression of deflections on the graph. These lines however enable the points grouped along them to be corrected for temperature and, while some results showed a greater degree of scatter than that of Fig. 7, diagrams of this type enabled corrections to be made with sufficient accuracy, given the ultimate use to be made of the results.

5.1.3 The damaging power of traffic

Results obtained from dynamic weighbridges installed at Alconbury Hill and at the sites of the later pavement design experiments constructed for the Road Research Laboratory indicate considerable differences in the spectra of wheel-loads in the traffic at the various sites and this is also true of the traffic passing in the two directions at the same site. That such differences are significant in terms of the different damaging power they represent is demonstrated by appreciable differences in performance observed at the same site between the slow traffic lanes carrying traffic in two directions despite uniformity of subgrade conditions and an equal number of vehicles passing in each direction.

The A.A.S.H.O. Road Test indicated that heavy wheel-loads caused much more damage to roads than light ones and related the damaging power of a wheel to approximately the fourth power of its load[3]. Although the results in terms of the actual damage caused to pavements are not applicable to British conditions because of the different environmental factors the A.A.S.H.O. conclusions relating to the relative damage caused by various wheel-loads can be used with greater confidence to express the damaging power of different traffic spectra.

In the recent revision of Road Note No. 29[9] A.A.S.H.O. equivalence factors have been used to express the damaging power of a spectrum of axle loads in terms of an equivalent number of "standard" axle loads of 8,200 kg (18,000 lb) and the total damaging potential of the traffic over a period of time as a cumulative total of such standard axle loads.

As data from a dynamic weighbridge have been collected at Alconbury Hill for the greater part of the life of the experiment the deflection histories relating to the sections have been expressed in terms of the cumulative number of standard axles carried as well as the cumulative flow of commercial vehicles on the carriageway.

The A.A.S.H.O. factors are in fact slightly affected by the pavement structure as well as the wheel-load itself. They are related to the Structural Number of the pavement, this being a function of surfacing, base and sub-base thickness. Those used in Road Note No. 29 represent a mean of figures derived for fairly substantial flexible and rigid pavements (Structural Number of 5 and a slab thickness of 225 mm (9 in) respectively). The Alconbury Hill experiment contains flexible pavements having Structural Numbers between 2.5 to 6.0 with the majority towards the upper end of this range. A check on the damaging power of the wheel-load spectrum which used the road on pavements of varying strengths has therefore been made. Table 3 shows the damaging power expressed in terms of Standard Axles per 100 commercial axles for a range of pavement strengths and the percentage difference from the values adopted in Road Note No. 29. No significant variation would arise except in the case of exceptionally thin under-designed pavements. Pavements of this type laid at Alconbury Hill failed so rapidly that close observation in terms of standard axles carried was not possible.

TABLE 3

Variation of damaging power on pavements of different construction

Structural Number *	Standard axles per 100 commercial axles	Percentage difference from the Road Note No. 29 Standard
Road Note 29	22.88	-
2	26.20	+ 14.5
3	25.15	+ 9.9
4	23.04	+ 0.7
5	22.78	- 0.4
6	23.29	+ 1.8

* A low structural number indicates a very thin pavement

5.1.4 Assessment of the condition of the road

The system described in 3.2 was adopted in developing the deflection histories. Information on rutting was obtained from measurements under a 1.8 m straight edge and from the optical levelling records made in connection with performance studies. Visual inspections regularly carried out during the first few years also provided background information and helped to pin-point the time when persistent cracking first developed.

Analysis of these data also showed that after the initial period of compaction the relation between rut depth and the cumulative total of standard axles responsible for its development was basically linear up to and generally beyond the value of 10 mm taken as defining a critical pavement condition. Typical records for sections with different types of base are given in Fig. 8. This linearity enabled the value of critical life to be determined by extrapolation for those points in a section which were still classified as being in sound condition at the time of its reconstruction or at the termination of the observations when the experiment was overlaid in 1969.

The rut depths at which appreciable cracking first developed were also studied. Fig. 9 indicates that for surfacings 100 mm thick, the great majority of the ruts (measured under a 1.8 m straight edge) were between 7.5 and 12.5 mm (0.3 and 0.5 in) deep when cracking occurred, with a mean close to the 10 mm value defining the onset of critical conditions in Table 1. As appreciable surface cracking would generally signify structural weakening, the initial choice of a rut depth of 10 mm to define the onset of the critical condition is thus supported.

An overlay to strengthen an existing road should be applied before major deterioration of the original pavement has taken place, and while it still substantially retains its load-spreading ability. However on economic grounds the overlay should not be applied too early. It is important therefore to examine the relation between the critical and failure lives, as defined in Table 1, that emerges from an examination of actual road pavements. The deflection histories discussed in this Report enable this to be done for all the areas of pavement which actually failed, but which also remained serviceable for sufficient time to enable reasonably accurate estimates of the two lives to be made. It was established that for most of the pavements the ratio of critical to failed life was about 0.75 expressed in terms of 'standard' axles carried. For a pavement designed for a total life of 20 years this represents a critical life of about 16 years taking into account probable traffic growth over the life span. Experience shows that an overlay designed on the basis of a deflection survey is not normally implemented until about one year after the actual survey. On the other hand such a survey would normally be carried out before the critical condition had developed over the majority of the pavement. Bearing these factors in mind the difference of four years between the critical and failure lives should be sufficiently long for action to be taken before serious structural damage has occurred.

On under-designed roads failing after less than about 10 years the rate of deterioration is greater and the difference between critical and failure life appears to be about 2 years. Speedy action would need to be taken on the results of any deflection survey.

For some of the areas of pavement examined during these studies the ratio between critical and failure lives was as low as 0.3. At these points it must be assumed that the surface cracking observed was only superficial and that it was mistaken for more deep-seated deterioration. In making the analysis the critical lives of these areas have been arbitrarily lengthened to about 0.75 of the failure life.

5.2 Results

In considering the deflection histories in each group of sections having a common type of base, shown plotted in Figs 13 to 21, it is necessary to appreciate certain factors likely to affect the deflection behaviour of the experimental sections and their performance under traffic. These relate to changes in the moisture conditions of the subgrade and the compaction of road materials.

5.2.1 Early changes in subgrade moisture and strength

The formation at the site was prepared by removing about 300 mm (1 ft) of soil cover and then shaping it to the saw tooth profile required to give the varying thickness of sub-base along each section, Fig. 5. Drying out of the soil by the grass cover had taken place in the dry spring weather before the formation was prepared and sealed. This was reflected in high values of in-situ C.B.R. tests made on the prepared formation immediately after sealing. These gave a mean value of 7 per cent for the whole site. Moisture profiles indicated in particular lower moisture content at the thin ends of the tapered experimental sections where the thickness of the original soil removed was a minimum. Prediction of the C.B.R. values likely when moisture equilibrium was achieved indicated values lying almost wholly between 3 and 6 per cent with a mean for the site of 4.1 per cent. These values have since been substantially verified by investigations on sections which failed in the first few years[10]. For example the first failure investigation carried out in May 1958 (6 months after construction) on sections 62 and 63 showed that the predicted reduction in soil strength had already taken place. However the movement in water-table levels recorded in bore-holes in the verge at the site, two examples of the results obtained being given in Fig. 10, suggested that the wetting-up process had not necessarily been complete by that date at all points along the road. It was therefore expected that the first set of deflection measurements after the road was opened to traffic, made in April 1958 would show increases over the initial values measured in the previous October and that further smaller increases might be recorded in the following spring at some points.

5.2.2 Subgrade conditions after the initial equilibriation period

As was anticipated all water-table profiles showed that deflection measurements made in the early spring coincided with reasonably consistent conditions of high water-table. The profiles also indicated the marked influence of the prolonged hot summer of 1959, and towards its end supplementary deflection measurements on four sections were carried out to investigate the effect on pavement strengths.

The unusually severe winter of early 1963, when freezing conditions penetrating to a depth of 450 mm (18 in) below the road surface were common, was expected to cause weakening of the experimental pavements during the following thaw period when abnormally high moisture contents existed in both subgrade and granular road materials. Additional deflection measurements on several sections were therefore made in March 1963 as soon as possible after the thaw.

Within any section the greatest deflection would be expected at the thin end of its tapering construction with values steadily decreasing with increasing pavement thickness. This was usually the case but that it was not always so reflects variations in thickness and quality of the pavement materials, and in particular, variation in subgrade strength. The latter is attributable to both minor variation in soil type and to the varying height of formation level above the water table along the length of any tapering section. The most extreme departure from the expected pattern of deflections was observed in section 49 where the deflections after moisture equilibrium was attained at the thin end of the section were marginally less than at the thick end, the corresponding CBR values being 7.7 and 1.7 per cent. While this represents an abnormal range of strengths at this site, variation between 3 and 6 per cent is common and is sufficient to distort detailed relations between deflection and total thickness that would exist under completely uniform sub-grade conditions. For certain sections where the pavement was removed before reconstruction a detailed penetrometer survey of the sub-grade was made. An example is given in Fig. 11.

5.2.3 Compaction effects

The sand sub-base used at Alconbury Hill was difficult to compact when unconfined. Although additional compaction occurred when the bases were laid over the sand, it is known that some compaction under traffic occurred during the early life of the road. The sand may also have had some detrimental influence on the compaction of the bases and particularly the wet mix base material. The effect of compaction within the pavement on deflection levels has been discussed in Section 2. It reduced initial deflections.

Failure investigations on eight sections which failed in the first three years showed that the mean dry density of their sand sub-bases had increased by 0.115 Mg/m^3 (7 lb/cu ft) from a value of 1.57 Mg/m^3 (98.1 lb/cu ft) at the time of construction and that this increase paralleled the increases in density observed in sections 62 and 63 when excavated in May 1958 indicating that sub-base compaction was already complete by that date. Similar large increases were recorded in the slag 'wet-mix'. Analysis of data obtained from in-situ testing of the sub-base at the time of its construction gives the strongly defined relation between CBR value and dry density of Fig. 12. This indicates that the above density change would bring about an increase in the CBR from 20 to 55 per cent. This increase in stiffness due to compaction was confirmed by increases of about 300 per cent in the dynamic moduli of the sand measured by save-propagation methods and up to 200 per cent in the slag[11]. Although a considerable part of these changes, particularly in the sand, would in fact have taken place during the construction of the upper layers of the road further wave-propagation measurements made in October 1957 on several of the completed sections showed that further increases of up to 100 per cent in the dynamic moduli of the slag bases must have taken place after the road was opened to traffic. Considerable decreases in measured deflections would be expected as a result of compaction. These decreases would counter the increases due to wetting-up of the sub-grade and in some cases might exceed them to bring about an overall reduction. The thick end of section 38, Fig. 14, showed the greatest reduction in deflection. It is significant that the formation under this section was the first to be shaped and sealed on the site (April 1957) and that a considerable part of the wetting-up could therefore have been completed before the initial deflection measurements in October. This section, built with a 225 mm thick slag base, contains the greatest thickness of compaction material in the experiment and therefore also had the greatest potential for deflection reduction.

The overall pattern of early deflection behaviour at this site confirms that initial deflection values are not a very reliable criterion of a road's future performance under traffic, if major changes in subgrade strength or stiffening of the road materials due to compaction occur in the road's early life.

5.2.4 Deflection histories of sections - general

The deflection histories presented in Figs 13 to 21 are for both nearside and offside wheel-paths of the experimental sections. As critical conditions developed first in the nearside wheel-path the behaviour there effectively controlled the performance of the sections and relationships between deflection and performance have therefore only been examined in detail for this wheel-path. The offside results, however, serve to check on possible anomalies, either in the experimentation or in the representativeness of the construction in the nearside wheel-path. All deflections presented are converted to a temperature of 20°C and are referred to as 'standard' deflections.

Inspection of the deflection histories indicated that, after the early changes in deflection already considered, subsequent changes on pavements which remained in sound condition were generally less rapid. While the deflections of some of the stronger sections remained substantially constant for a number of years, in a few cases until the end of the twelve and a half year life of the experiment, the great majority showed a slow increase with time mostly subsequent to the winter of 1963. The immediate effect of this winter was to cause large irreversible increases in the deflection of many of the surviving sections and was often associated with the developments of critical conditions in them. It is likely that this damage and the pattern of slow increases in deflection on the longer lasting sections is at least in part a consequence of the high water-table condition at this site which also served to accentuate the damaging effect of the severe winter.

Any road with a design life of longer than about ten years is likely to be subject to one winter of abnormal severity, though perhaps not one as severe as that of 1963. It is therefore logical to accept the effects of this winter on deflection levels and pavement performance at Alconbury Hill as being reasonably representative of those at a poorly drained site, at least for the stronger sections where their design life under the heavy traffic using the road is ten years or more. The thinner sections at Alconbury Hill underwent trafficking at a greatly accelerated rate compared to that to which they would be subject in achieving a design life of more than ten years under the traffic for which their structure would be appropriate. These would be expected to and did fail before the winter of 1963 and their performance was therefore not affected by that winter, while pavements of intermediate strength effectively subject to a lesser degree of acceleration, which in the absence of the severe winter would have reached a critical condition in, say, 1964 or 5 may have in fact done so in the spring of 1963.

In view of the possibility of the existence of a discontinuity in road performance and deflection behaviour in 1963, critical lives of individual points in the experimental sections have been analysed in terms of mean values of spring deflections prior to 1963 or to mean values up to the time critical conditions develop, if this was before that date. These deflection values are subsequently referred to as standard early life deflections. The spring readings in 1958 were included unless considerable increases in deflection between them and the following spring took place. These mean values are also those potentially of the greatest value to the road engineer in predicting pavement performance and an analysis using them should also indicate whether the abnormal winter had any major effect on deflection performance relations.

5.2.5 Rolled asphalt bases

This group of sections, Fig, 13, was founded on perhaps the most uniform length of sub-grade on the site. CBR values at the time of construction of 4 to 10 per cent compared to predicted equilibrium values of 4 to 4.5 per cent. The latter were substantially confirmed by penetrometer measurements made at the bottom of core-holes near lines 1, 3 and 5 on all sections of this group in 1969 and 1970. However under section 50, in which the rolled asphalt surfacing was laid directly on the sand sub-base, lower values close to 3 per cent were measured. As was expected the greatest increases in deflection which occurred immediately after the road was opened to traffic took place at the thin ends of the sections.

The influence of the hot summer of 1959 can be observed from a comparison of sections 51 and 52. No change in standard deflections took place during the summer on the stronger section while section 51 showed considerable increases including one of 50 per cent at its thinnest end. These increases had however disappeared by the following spring. As Fig. 10 shows that although water table fell by about 1 m (3 ft) during this summer, the effect of this was clearly not sufficient to prevent an overall temporary weakening, particularly of the thinner pavements in this series.

Section 52, the stiffest section of this group on which extra measurements were made in March 1963, showed increases in deflection due to the abnormally wet conditions during the thaw, the greatest increases being at the thin end of the section. The deflections had returned to 1962 values by the time of the normal spring measurements at the end of April. However on weaker sections considerable irreversible increases were recorded over this period.

There is little evidence of structural deterioration of the base and surfacing material during the life of the experiment. Although wave propagation measurements made in 1960[11] indicated a slight weakening at the bottom of the base material no evidence of this was obtained from the physical condition of cores taken in 1969 or from their densities measured by a radioactive technique.[12] One core did give an exceptionally low value of density throughout the depth of the base and this presumably represented a poor state of initial compaction not completely rectified by traffic action rather than deterioration with time. The core densities did however confirm the lower shear moduli, measured by wave propagation[13], in the lower 75 mm (3 in) lift of the two course 150 mm (6 in) bases caused by difficulties encountered in fully compacting the asphalt directly on the sand sub-base. These lower moduli were also measured in the 75 mm (3 in) base of section 51, laid in one lift.

Standard early life deflection values for all points have been plotted against their critical lives in terms of cumulative standard axles in Fig. 22. Where the road at a point was still in sound condition at the end of the experiment its critical life was determined by extrapolation of the linear relation between rut depth and standard axles noted in section 5.1.4. Where extrapolation was not possible the last measurement on the point has been plotted in such a way as to indicate a life for the point which must be longer than that indicated by the position of the plotted point.

The results indicate a very strongly defined relation between early life deflection and performance of the form

$$\text{Life} \propto \frac{1}{(\text{Deflection})^3}$$

The results from section 50 suggest that a deflection of about 1.2 mm (0.050 in) is the maximum consistent with satisfactory performance even under very light traffic.

Differences of sub-base thickness are indicated on Fig. 22 by reference to the "line" number corresponding to the measurement which can be referred back to the key included on Fig. 13. While sub-base thickness affects both the critical lives and their deflection values the distribution of plotted points indicated that separate criterion curves related to sub-base thickness are not required.

Also shown on Fig. 22 is a minimum criterion curve (shown dotted) above which virtually all points lie. The difference in deflection of .04 - .06 mm between the two curves represents the difference between all points on a road and only 50 per cent of them being sound at a given time. Expressed in terms of critical lives the second of these conditions is reached after a life about 30 per cent greater than that for the first condition. The relation between mean and minimum criteria will play an important part in the statistical analysis of the large amounts of data generated by deflection surveys on roads particularly where the Deflectograph is used.

5.2.6 Wet-mix slag bases

Sections with wet-mix slag bases were located in two groups, the first designed to examine the effects of base thickness and the second the effects of surfacing type and thickness on pavement performance. The deflection histories are also presented in Figs. 14 and 15 corresponding to these two groups, with the history of section 60 whose structure is common to both, repeated on both diagrams. (This presentation is followed for all the remaining types of base in the experiment.)

Estimated equilibrium CBR values were near the mean for the site, with the exception of section 39, whose predicted value of 2.8 per cent was in fact the lowest at the site. This value was subsequently verified by failure investigation and penetrometer testing which also indicated considerable variation along the length of section 40 and higher than average values (about 5 per cent CBR) for sections 59 and 60. Table 3 gives values of dynamic shear moduli derived from wave propagation measurements[11] made on some of the sections with wet-mix bases before opening to traffic and also subsequently.

TABLE 3

Dynamic shear moduli (wave propagation) of wet-mix slag bases

Section number	Dynamic modulus MN/m^2 (lbf/in^2)		
	Oct 1957	June or Nov 1958	May 1960
38	69 (10,000)	-	96 (14,000)
39	41.5 (6,000)	41.5 (6,000) June	-
40	48 (7,000)	48 (7,000) June	-
59	55 (8,000)	138 (20,000) Nov.	138 (20,000)
60	-	-	117 (17,000)

It is significant that in section 39, built on a very weak sub-grade and in section 40, where only a thin base was used on a sub-grade which was also very weak at the thick end of the section, values of base moduli did not increase under the influence of trafficking. The severity of the stress conditions imposed on the bases of these two sections was presumably such as to preclude their stiffening by compaction and cementitious hardening which took place on other sections. That no significant compaction occurred was confirmed by density measurements made in 1960 on section 39. These pieces of evidence are reflected in the deflection behaviour. The deflection changes which took place between opening the road and the following spring are summarized in Table 4.

TABLE 4

Percentage change in deflections on wet-mix slag bases during the first six months of trafficking

Section No.	Change in base stiffness	Percentage change in deflection		
		Line 1*	Line 3	Line 5
38	Increase	- 33.5	- 16	- 3.5
39	None	+23	+ 36	+ 30
40	None	+14	+ 1.5	+ 29.5
60	Increase	- 30	- 13	- 21

* See Fig. 14 for significance of line numbers.

Sections 39 and 40 showed increases in deflection after the road was opened probably brought about mainly by wetting up of the sub-grade, whereas in sections where stiffening of the base was detected the reduced, or little changed, deflections indicated that the effects of wetting-up were neutralized or reversed by the stiffening of the base material.

As the elastic moduli of granular materials are stress dependent, the weaker bases in the presence of a weak sub-grade develop even lower moduli than in a more normal situation and this is particularly reflected in the high deflections and short lives derived from section 39 when compared to those of its nominal replicate section 60 built on a stronger sub-grade and which had a base which developed increased stiffness with age.

That the stiffening of the slag by cementing action continued for some time after the spring of 1958 when compaction effects were largely complete is indicated by the deflection history of the thick end (line 1) of section 38 whose spring deflections continued to decrease for two or three years after the road was opened.

The hot summer of 1959 brought about increases in standard deflection in the thin end of section 60. These returned to normal values in the following spring. Considerable increases in deflection were also measured on sections 38 and 60 at the time of the spring thaw in 1963. At the weakest points in both sections the increased deflection levels were not appreciably reduced subsequently and critical conditions developed within a year, while at the stronger points pre-thaw levels of deflection were generally re-established in this period.

The deflection performance results for wet-mix slag bases under a 100 mm (4 in) rolled asphalt surfacing are given in Fig. 23. Results for section 50 (zero thickness of base) are also included. The mean criterion curve through the points is similar in form to that derived for sections with asphalt bases. For critical lives greater than one million standard axles the curves are in fact identical. No influence of sub-base thickness or sub-grade strength on the criterion curve was detectable, and the plotted points are not distinguished in the manner used for the asphalt bases.

It is important to realise that similar criterion curves for two different forms of construction do not imply equivalent performance of the base materials. A greater thickness of one base may be required to achieve equivalence of deflection, and therefore performance.

Fig. 24 gives the results derived from the performance of wet-mix bases under different thicknesses and types of surfacing.

Reduction of the thickness of asphalt surfacing to 70 mm (2¾ in) Section 61 gives a criterion curve of the same form for a limited range of lives, coinciding with that derived from the sections with a 100 mm (4 in) asphalt surfacing. Similar agreement cannot be confirmed for thinner surfacings as much of section 62 built with the 40 mm (1½ in) surfacing had failed and all had been replaced before definable values of equilibrium spring deflection could be established. As the first signs of failure in this section were the cracking of its surfacing before the development of serious rutting which characterized the behaviour of the other sections, a different deflection criterion may well be appropriate. Its results are however plotted for information.

The four points derived from section 59 built with a 100 mm (4 in) open textured bituminous macadam surfacing lie grouped around the mean curve of Fig. 24, 100 mm rolled asphalt surfacing. However the result for the longest lasting point was derived by extrapolation from rut depth data and it is possible that this pavement would fail by attrition of its weak surfacing before the predicted critical life was reached.

5.2.7 Lean concrete bases

The deflection histories for this group of sections are given in Figs 16 and 17.

Predicted equilibrium CBR values under the three sections with 100 mm asphalt surfacings designed to investigate the influence of base thickness on performance (44, 45 and 46) were appreciably greater than that for the site as a whole. Estimated equilibrium values of 5.5, 6.5 and 5.0 per cent were later confirmed by penetrometer measurements as being 5.7, 6.4 and 6.0 per cent. These values represent slight increases compared to initial values and they also conceal larger changes representing both wetting up and drying out at individual points.

The remaining sections of the group (67-70) were founded on substantially weaker sub-grades with estimated equilibrium CBR values again confirmed by subsequent measurements on three of the four sections.

High deflection values and the results of wave propagation tests demonstrated that the base material in the 75 mm (3 in) base of section 46 was broken up, probably by construction traffic, before the road was opened. Deflections on all other sections of the group with the exception of section 69, were less than 0.25 mm (0.010 in). These low values reflected the great stiffness of this type of base. The latter was shown by wave-propagation measurements to have been in a virtually uncracked condition with a Youngs Modulus of between 24×10^6 and 38×10^6 kN/m^2 (3.5×10^6 and 5.5×10^6 lbf/in^2).

It can be shown by multi-layer elastic analysis that under an uncracked concrete base of 150 mm (6 in) or more, stiffening of the sub-base in the early life of the road would not cause any appreciable changes in deflection values. As changes in the sub-grade strength following the opening of the road are known to have been small at amny points it follows that, in the absence of major deterioration of the surfacing, changes of deflection would reflect the developments of cracking in the base. Further wave propagation measurements from which a measure of the degree of cracking in the base can be deduced were made over line 3 on Sections 44, 45, 67 and 68 in 1959 and 1960 and on lines 1, 3 and 5 of sections 44 and 45 in 1968 when these two sections were the only ones in the group still largely classified in terms of their surface condition as being sound. This enabled the relation between changes in deflection and the development of cracking in the base to be followed. Wave propagation tests enabled the condition of a base to be classified as being either virtually uncracked (with perhaps an occasional lateral shrinkage crack), partially cracked, or extensively cracked. Results of the study are given in Fig. 25.

Most points show increases in deflection between 1957 and 1960 and at all points which did so the development of cracking was also detected by the latter date. The onset of cracking was therefore detected by deflection changes at least as sensitively as by wave propagation techniques. Only one point in section 44 showed no change in deflection over the three year period. It survived until 1968 with its base described as virtually sound and with only a small increase in deflection between 1960 and 1968. The results show that different levels of deflection can be ascribed to different conditions of the base and also suggest that the critical deflection level separating bases in sound condition from those in which cracking is developing (shown dotted on Fig. 25) tends to decrease with time and traffic as would be expected. As the deflections are showing a tendency to increase even below this level it is possible that cracking may be beginning at lower deflection levels than indicated by wave propagation measurements.

After the opening of the road to traffic deflections on the initially uncracked sections tended to increase, with the greatest increases taking place at the thin ends of sections, where cracking was most likely to develop first. It is noticeable that once deflections exceeded the 0.25 mm level at which cracking would be developing according to Fig. 25, results in succeeding years became more erratic, presumably due to the reaction of the developing crack pattern in the base on the measured deflections.

Deflection levels indicated that at the majority of points of measurement in the thickest section of the group (44) cracking was developing in the base by 1962 and that this was generally true of the other sections in 1959. However the sections with a 100 mm (4 in) surfacing of either asphalt or bituminous macadam were classified as being sound for several years afterwards and, in the case of those founded on the stronger sub-grades were still so rated at the end of the experiment in 1969. On the other hand under the thin asphalt surfacing of section 70 cracking of the base was quickly followed by rupture of the surfacing and total failure of the section. The section of intermediate surfacing thickness (69) also had to be replaced within two years of general cracking of the base developing. This was however in a partially cracked condition initially.

Establishment of mean deflection levels was impossible for some points whose deflections rapidly increased in line with the deterioration of the pavement to a critical condition. Significant mean values were also difficult to define where deflections on sections with cracked bases showed great variation from year to year.

The deflection-performance plots for all sections with lean concrete bases are given in Fig. 26. The results from sections with 100 mm (4 in) rolled ashalt surfacings on 150 mm (6 in) or 225 mm (9 in) bases lie grouped about a line of similar slope to that derived for both wet-mix slag and asphalt bases, but considerably below it. Within this group the points derived from section 68 gave higher deflections and shorter critical lives than those from its nominal replicate (section 45). The differences reflect the stronger sub-grade conditions under the latter section.

Two points lie well below the mean line. Point B is the only one remaining substantially uncracked at the end of the experiment. As other points in the section performed satisfactorily when in a cracked condition for at least 11 years, representing traffic equivalent to 6 x 10^6 Standard Axles, the likely minimum critical life of this point is 13 x 10^6 Standard Axles. At point A cracking began to develop between 1964 and 1968 and by the same argument its minimum life is therefore about 11.5 x 10^6 Standard Axles. Both minimum figures are confirmed by extrapolation of the relationship between rut depth and traffic. The deflection criterion curve for critical lives in excess of 10^7 Standard Axles is therefore a line drawn either through these points as shown or to their right. The implication of a line of the gradient shown between 10 and 14 million standard axles is that however great the thickness of base and sub-base used no life greater than about 20 x 10^6 Standard Axles would be possible with this form of construction under a 100 mm (4 in) asphalt surfacing. While theoretically this appears to be unlikely when considering the behaviour of a small area of road in which the base is continuous, lateral cracks due to shrinkage and thermal stresses occur in the base of roads of this type. They tend to be spaced at 5 to 8 metre intervals and the surface deterioration which slowly develops in the vicinity of these cracks ultimately constitutes a maintenance problem. The cracks themselves do not significantly increase the deflection level while granular interlock is maintained. Their presence can be said to define the critical life of this type of pavement under heavy traffic conditions, if no failure occurs previously from another cause. In practice for lives of more than 10^7 Standard Axles Road Note 29 recommends the use of an additional upper base layer of bituminous material. While not appreciably affecting the magnitude of the deflection or of the traffic stresses generated in the lean concrete base its presence will moderate the intensity of thermal stresses in the base and also delay or prevent the onset of reflection cracking in the asphalt surfacing over cracks. The inclusion of this upper base layer in the construction at points A and B would therefore lengthen their critical lives considerably without altering their deflection values. For very heavy traffic the dotted extension of the criterion curve beyond 10^7 Standard Axles is then probably more appropriate.

The lives derived from points on section 67 with a 100 mm (4 in) bitumen macadam surfacing were similar to those from the adjacent section 68 with rolled asphalt surfacing of similar thickness. Both were founded on sub-grades of similar strength. The slightly lower average deflection measured on the section 68 presumably reflect its stiffer surfacing, which did not however noticeably improve the performance.

The base of the thinnest section in the group (46), being extensively cracked before the road was opened, gave deflections whose magnitude was typical of those of a section with a thin granular base. Its performance was also typical of this type of construction and the points derived from its behaviour lie on the criterion curve for wet-mix slag bases.

No meaningful performance data could be derived from the results of section 70 and only two points from section 69. These are plotted for information.

Additional observations made on section 44 in March 1963 did not show any significant effect of that severe winter. The normal spring measurements made before and after the 1963 frost on sections 67 and 68 which were more extensively cracked and were sited on the weaker sub-grades were however appreciably higher but generally recovered subsequently.

5.2.8 **Soil cement bases**

Deflection histories for this group of sections are given in Figs 18 and 19. The predicted equilibrium CBR values for these sections were generally close to the mean value of 4 per cent for the site although where testing was subsequently carried out during failure investigations some variations were noted. Section 49 showed the marked decrease in CBR value with increasing depth of construction typical of several sections at this site. Sections 65 and 66 also proved to have weaker sub-grades than predicted with mean values of rather less than 3 per cent. CBR values at the time of construction were typically in the range of from 4 to 10 per cent.

When tested 15 days after being laid the bases gave values of dynamic moduli of about 3.1×10^6 kN/m^2 (4.5×10^5 lb/in^2) except for section 47 where the better compacted density resulted in a higher modulus of 4.1 kN/m^2 (6.0×10^5 lb/in^2). Wave propagation measurements made before the road was opened to traffic indicated that the bases of all sections tested except the thickest and strongest of the group (47) already showed signs of cracking and that the cracking in the section with the thinnest base (49) was extensive. Deflections on this section were greater than those observed on the untrafficked wet-mix base of equal thickness (40), thus confirming extensive fragmentation of its base material. The high initial deflections on section 66 also indicated a base in badly cracked condition. Further cracking in all sections was detected by wave propagation in the period between the road being opened to traffic and the succeeding autumn. During this period compaction of the sub-base and wetting up of the sub-grade was also taking place. From this combination of factors increases in deflection resulted at most points, the major exceptions being sections 49 and 66 where the compaction of the sub-base beneath bases already fragmented when the road was opened was the dominant factor. This led to considerable reductions in deflection from very high initial values. Thereafter critical conditions developed fairly rapidly with little change in deflection and therefore presumably with little further degradation of the base material. Failure investigations made when the sections were later reconstructed demonstrated that the bases were badly cracked. Untypically, critical conditions first developed at the thick end of section 47 and the approximately equal deflections measured at all points along its length indicated a weaker condition of the base towards the thick end, probably brought about by inadequate support from a weak sub-grade.

It is noticeable from Fig. 18 that the thicker soil cement bases in their disintegrated condition gave consistent deflection values from year to year in contrast to the results obtained on lean concrete in a badly cracked condition. This probably represents differences in the way the two materials broke up, the former into small interlocking lumps and the latter into larger pieces. This was confirmed by a subsequent failure investigation.

Extra measurements were made in September 1959 to examine the effect of that abnormally hot summer. Only the offside wheel-path of section 48 was tested where moderate increases in deflection took place at the thin end of the section. By the following spring the deflections were however reduced to values equal or below that of the preceding one.

On the deflection-performance plot for sections with 100 mm (4 in) rolled asphalt surfacings given in Fig. 27 the results for the thicker bases lie grouped about a criterion curve identical with that derived for the much stronger lean concrete bases. A further similarity is to be seen in the behaviour of the 75 mm (3 in) thick bases of the two materials. Both bases were completely broken up before the road was opened and their results also conform more closely to the criterion curve for the granular wet-mix slag bases. The performance of the stronger lean concrete material was however much better in the fragmented state.

The rapidity with which critical conditions developed in two of the four remaining sections of the group made it impossible to establish with accuracy either their deflections or critical lives. As happend on the wet-mix bases the section with the thinnest 40 mm (1½ in) surfacing deteriorated initially by cracking and crazing of its wearing course and again a different criterion may be appropriate.

The deflection/performance plots for the remainder of this group of sections are given in Fig. 28. The very high initial deflections measured on Section 66 constructed with a 100 mm bitumen macadam surfacing made accurate determfination of both deflections and critical lives difficult. However the results approach the mean curve for granular bases under 100 mm surfacings and lie astride the extrapolated curve for soil-cement bases under 100 mm asphalt surfacings.

Changes in the thickness of asphalt surfacing in the range 40 to 100 mm did not greatly change the measured deflection; on all three sections (63, 64 and 65) the minimum deflection was about 40 x 10^{-2} mm (0.016 in). This indicated that the surfacing was contributing relatively little to the stiffness of the stiffer pavements of this type. The presence of the thicker surfacings did however greatly increase the critical lives as Fig. 18 shows, and the criterion curves are therefore dependent on the surfacing thickness.

The results from the Section of intermediate surfacing thickness (64) also merge with those of the other sections at deflection levels where the base must be disintegrated. Only the section with the thinnest surfacing (63) which, as in the case of the corresponding section with a wet-mix base (62), deteriorated rapidly by cracking and crazing of the surfacing, gave a rather different criterion with a disintegrated base.

5.2.9 Tarmacadam bases

Deflection histories for this group of sections are given in Figs 20 and 21. The sections laid to determine the effect of base thickness on performance (sections 41 - 43) were constructed with an open-textured tarmacadam on a sub-grade initially of fairly uniform strength with CBR values typically between 3.5 and 4.5 per cent. These values were generally a little greater than predicted equilibrium values.

During the construction of the experiment, the sub-base of sections 42 and 43 was flooded with water from a sudden rain storm, before surface sealing had been carried out. The sand was removed over a width of about 5 feet on the nearside of these sections and new sub-base laid with a gravel drain adjacent to the nearside edge beam. As a result of these operations differential settlement occurred between the old and new sub-base and the sections were declared non-experimental. Section 42 was patched to take out the settlement but the greater part of section 43 remained unpatched although a rut in excess of 100 mm rapidly developed in the nearside wheel-path. Subsequently the rut developed extremely slowly, at a rate which was consistent elsewhere with that of a section destined for a long life. No cracking of the surfacing took place. The level of deformation defining the critical threshold was therefore increased for this section to take into account the settlement factor which caused its abnormal early behaviour. Deflection measurements were made on all three sections.

Marked increase in deflection occurred during the first few months of the life of these sections. These were followed by slow decreases of similar magnitude over the next year consistent with increased traffic compaction in bases and sub-bases. Subsequent deflection changes were small, with the exception of some weaker points whose deflections increased irreversibly following the forst of 1963.

Wave propagation measurements suggested that some stripping of the bases of sections 41 and 43 had taken place in the first two years. However little evidence of any such action was visible when section 41 was reconstructed several years later. Only the lower part of its base at the thin end had noticeably deteriorated. It is unlikely that degradation of the tarmacadam bases had any significant effect on their performance.

The remaining sections of the group (55 - 58) utilised a medium-textured tarmacadam base. They were built on a sub-grade initially in a more variable condition than sections 41 - 43. While mean predicted equilibrium values were generally higher than for the first sub-group penetrometer measurements made when section 55 was reconstructed gave CBR values of the order of only 1.5 to 2.0 per cent as compared with initial values of between 3.5 and 7 per cent. Initial levels of deflection showed little change in the first year but large increases in the year following, particularly at the thin end of the section. Subsequent values remained high and only under the 100 mm (4 in) asphalt surfacing of section 57 did substantial reductions in deflection take place after the hot summer of 1959. The large degree of wetting up known to have taken place under section 55 would not have been completed by the spring of 1958. The accompanying strength changes up to this time were probably largely masked by increases in sub-base stiffness due to traffic compaction and only became apparent in deflection levels in the following twelve months after the sub-base was fully compacted. As this deflection change is repeated on section 56 and to a lesser extent on section 57 it is likely that similar large increases in moisture content took place under these sections also. Only on section 58 where initial CBR values were the lowest of the sub-group were increases in deflection relatively small.

No appreciable signs of stripping of the tarmacadam bases were noted when the sections were reconstructed between 1964 and 1966.

Fig. 29 gives the deflection-performance relation derived for the two sections of open-textured tarmacadam bases whose performance was not considered to be unrepresentative due to the construction difficulty already noted (sections 41 and 43). It follows closely the line already derived for both asphalt and wet-mix slag bases under 100 mm (4 in) asphalt surfacings also shown. Both deflection and performance of the thicker end of section 41 are little different to that of much of the section 43 whose base is three times the thickness of that of the former section. This cannot be explained by compensating differences in sub-grade strength under the two sections and therefore represents differences in the quality of the two pavements. It is encouraging to note however that the same criterion-curve is still applicable to both.

The results from sections with 150 mm (6 in) medium-textured tarmacadam bases shown in Fig. 30 give good agreement, irrespective of the type and thickness of surfacing, with the mean curve from the more open-textured bases up to critical lives of 2.5×10^6 Standard Axles. For longer lives the pattern is less clear. While some points conform to the mean curve two lie well below. Two other points which lie slightly below the curve remained sound at the end of the experiment. Further evidence would therefore be necessary to establish the deflection/performance relation for medium-textured bases on very heavily trafficked roads. It is however unlikely that the curve lies appreciably below that for the open-textured bases because, although the latter are probably rather less stiff than the medium-textured one and would therefore give higher deflections, any major divergence between the curves for the two types of base would imply superior performance of the open-textured material, which has not been observed in practice.

6. STRUCTURAL PARAMETERS AND THE DEFLECTION-PERFORMANCE RELATIONSHIP

Deflection has been shown by the investigations described in this Report to be related to the performance of a road and to be a measure of structural parameters known to influence road performance, such as the thickness and stiffness of the pavement layers and strength of the sub-grade. The ability to relate these pavement and sub-grade parameters directly to deflection-performance relationships of the type derived in this Report would enable deflection measurements to be used to assess how satisfactorily a particular road design had been achieved in practice. From the deflection level it would be possible to decide whether a pavement was well built and also whether the "as-built" sub-grade conditions agreed with those assumed in the design. Deflection measurements alone cannot necessarily distinguish between these two aspects but major deviation from expected deflection values would be an indication of possible future trouble. The early deflection, considered in relation to the appropriate criterion curve, will indicate the probable life before the condition becomes critical and wheter this is satisfactory in relation to the overall design life chosen.

For the reasons already discussed the Alconbury Hill experiment was not ideally suitable for the derivation of relations between deflection and structural pavement parameters. Those sections with rolled asphalt bases, whose surfacing and base could be considered to be one layer and which were built on a relatively uniform sub-grade, provided data which enabled the individual influences of pavement and sub-base thickness on deflection levels to be evaluated with confidence. Fig. 31 shows relations between standard deflection (20°C) and critical life for various thicknesses of base and sub-base deduced from the data given in Fig. 22 and a knowledge of the actual thicknesses of base and sub-base at the various points of measurement. To correct the deflections at individual points to take into account differences of CBR value from the mean value of 4.5 per cent for these sections two approaches were used:-

1. The CBR of the sub-grade was related to its effective elastic modulus E by

$$E = A.(CBR)^{2/3}$$

where E is the modulus derived from wave-propagation measurements at the time of construction and the CBR values are those obtained by in-situ testing at the same period[14]. Treating the road as a layered elastic system the appropriate sub-grade moduli were then used to correct deflections to the standard sub-grade condition.

2. The general pattern of the results obtained showed that deflections were not in complete conformity with the assumption of linear elastic behaviour. Comparison with the results obtained from other experimental sites suggested a rather greater dependence of deflection on CBR than indicated by the elastic assumption.

In deriving Fig. 31 the mean of the two approaches is used. From Fig. 31 the deflection, and therefore performance, equivalence between asphalt and sand sub-base can be derived. It varies between 8 to 1 and 2.2 to 1 depending on the combination of layer thicknesses considered. In particular the equivalence is sensitive to the ratio of pavements to sub-base thickness as shown in Fig. 32.

The concept of layer equivalence to simplify the relation between pavement performance and pavement structure was used in the analysis of the results of the A.A.S.H.O. road test[15]. Performance was related to one pavement parameter, the Structural Number (or Thickness Index) N defined as

$$N = 0.44L, + 0.14L_2 + 0.11L_3$$

where L, L_2 and L_3 are the thicknesses of asphalt, granular base and sub-base respectively. The constant equivalence between layers was imposed by the form of the mathematical model selected for the analysis. While it will be seen that the asphalt/sub-base equivalence of 4 to 1 lies within the range of values derived at Alconbury Hill it is not known whether a different type analysis would have resulted in variations similar to the Alconbury Hill values.

If the Structural Number concept is applied to the results of the sections at Alconbury Hill constructed with asphalt bases, with the asphalt coefficient of 0.44 and the sub-base coefficient varying as shown in Fig. 32, the good relation between Structural Number and critical life shown in Fig. 33 results. The curve, corrected to correspond to uniform sub-grade conditions is shown dotted. Comparison of lives derived from the A.A.S.H.O. test with those from Alconbury Hill for pavements described in terms of Structural Number is given in Fig. 34. The lives of the A.A.S.H.O. pavements are defined in terms of limiting rut depth equal to that used to define critical failure lives in British practice.

The major difference between the two experiments is one of climate. The severe winters at the A.S.S.H.O. test site, resulting in prolonged frost penetration of the sub-grade and subsequent pavement weakness during the spring thaw period, is not normal in Britain. On the thinner pavements at A.A.S.H.O. the majority of damage occurred in the thaw period while the more substantial pavements were little affected. It would therefore appear logical that agreement between lives of a given Structural Number at the two experiments should be obtained for thicker pavements designed for heavy traffic while pavements of lighter construction would give longer lives in the less severe environment of the United Kingdom.

7. COMPARISON OF THE DEFLECTION APPROACH WITH THE RECOMMENDATIONS OF ROAD NOTE NO. 29, THIRD EDITION

The information given in Fig. 31 relates the deflection of pavements with rolled asphalt bases and sand sub-bases of various thickness to the critical lives of these pavements. It was derived from deflection studies on pavements whose performance in terms of deformation behaviour was the basis of the thickness recommendations in Fig. 7 of Road Note No. 29. It is therefore of interest to compare the results of these two completely different types of performance analysis both based on the in-service behaviour of the same pavements, that is, the deflection approach and Road Note No. 29.

Road Note No. 29 recommends minimum thicknesses of pavement layers for roads with different design or failure lives. As critical life has been shown in section 5.1.4 to be about 75 per cent of the failure life considered in Road Note No. 29 minimum thicknesses required for different critical lives can therefore be deduced for direct comparison with data from Fig. 31. In Fig. 35 the combined thicknesses of asphalt surfacing and base predicted by the Deflection Method for thicknesses of sub-base equal to that required by Road Note No. 29 are compared to the pavement thickness recommendations of the Road Note. Good agreement is obtained over a wide range of critical lives particularly in the experimentally verified range shown by the solid line.

8. APPLICATION OF DEFLECTION CRITERIA TO ROAD MAINTENANCE REQUIREMENTS

Fig. 36 reproduces the relations between early life deflection values and critical lives of pavement with 100 mm rolled asphalt surfacings in combination with the five types of base used in the Alconbury Hill experiment. The curves have been shown to be substantially independent of sub-base and base thickness and also of sub-grade strength in the range of CBR values between 2.5 and 7 per cent. It is however possible that the use of different types of sub-base or construction on sub-grades either weaker or stronger than the range quoted above would affect these relationships. Data obtained from other sites are therefore being examined to check and extend the validity of the curves in Fig. 36.

Examination of the deflection histories given in Figs 13 to 21 indicate that on almost all pavements, including many of these which remained in sound condition for the full 11½ year term of the experiment, a slow increase in deflection with time and traffic took place once the compaction phase was complete. The pattern is partly obscured by variation in deflection from year to year due to changing sub-grade conditions and to inaccuracies arising from major temperature corrections, but plotting on the basis of a running three year mean brings out the general slow upward trend more clearly. The significance of a given level of deflection in predicting future performance therefore depends on the age of the road when the deflection measurement is made.

Fig. 37 shows the deflection values when critical conditions develop for the sections with rolled asphalt bases. Also shown are mean trend lines of deflection from early life values towards the critical curve.

Fig. 37 enables the unexpired life of a pavement to be estimated at any stage before critical conditions develop, from a knowledge of the deflection and of the cumulative traffic which has used the road to date. Estimation of the latter parameter can be made from Ministry of Transport census data and the wheel-load factor given in Road Note No. 29.

This figure embodies almost all the deflection performance data presently available relating to rolled asphalt bases. For the other base types used at Alconbury Hill there is also considerable other evidence for sites having different traffic and sub-grade conditions. This is being analysed to check if major sub-grade differences affect the deflection peformance relationship and it is hoped to prepare charts similar in form to Fig. 37.

Deflection surveys of roads will indicate at what time strengthening will be required and must therefore help in forward planning and making the best use of maintenance finance. The magnitude of the measured deflections, used in conjunction with the results of overlay studies now in progress, will also give a guide to the thickness of any overlay required and to the relative economics of overlaying as opposed to reconstruction where major damage has already occurred.

Numerous surveys using this approach have already been completed and rehabilitation programmes for the M1, M5 and M50 have been largely prepared in this way.

9. CONCLUSIONS

1) The early programme of research on experimental pavements already in service showed that for crushed stone construction meaningful relationships could be established between the deflection measured under standard conditions and the cumulative flow of commercial vehicles needed to bring about critical pavement conditions indicating the need for overlaying in the near future. Little change of deflection with time and traffic was observed at least until after critical conditions developed. There was some indication that the nature of the spectrum of axle-loads carried as well as the total commercial traffic affected road performance. An insufficient number of examples of critical conditions developed on the tarmacadam and sand-cement bases under observation for criteria to be developed. Cracking of the sand cement and stripping of the tarmacadam was however readily detectable as a considerable increase in the deflection level. From the detailed deflection histories developed from measurements made throughout the life of the experiment in flexible construction at Alconbury Hill several important conclusions were established.

2) Changes in moisture conditions in the sub-grade following the disturbance of the construction phase can cause considerable changes in deflection level. Compaction of granular layers of the pavement and cementing action can bring about major reductions in deflection. While compaction is normally complete within about a year cementing action and moisture equilibration in the sub-grade can take longer. Therefore, in many cases, deflections measured before the road is opened to traffic are an unreliable indicator of its future performance.

3) After the initial equilibration phase, just described, deflections changed very slowly. This enables measured deflections to be corrected to a standard equivalent deflection at a temperature of 20°C by considering all the deflection results from any point as one group for deflection/temperature analysis.

4) Changes in the stiffness of pavement layers detected by wave-propagation techniques were closely related to observed changes in deflection.

5) Relationships between pavement (combined base and surfacing) thickness, sub-base thickness and deflection were obtained for pavements with rolled asphalt bases. For the other types of base, containing greater variations in base stiffness within each group and generally founded on less uniform sub-grades convincing numerical relationships could not be established.

6) In each group of sections incorporating a particular type of base the sections with a 100 mm (4 in) rolled asphalt surfacing gave a wide range of critical lives reflecting both differences in base and sub-base thickness and variations in sub-grade strength. Well defined relationships between critical lives and mean early life deflections were obtained. For lives of greater than about one million standard axles the curves were of the form

$$\text{Life} \propto \frac{1}{(\text{Deflection})^3}$$

7) Virtually identical curves were derived for rolled asphalt, tarmacadam and wet-mix slag bases, This does not however imply equal performance as different thicknesses of different bases are required to attain a given deflection, and therefore life. Similarly sand-cement and lean concrete bases shared a common curve, although over a shorter range of experimentally verified critical curves. Again equal performance by the two types of base is not implied.

8) Sections with wet-mix soil cement and lean concrete bases under the thinnest 40 mm (1½ in) asphalt surfacings deteriorated by surface cracking before the more normal rutting in the wheel-paths had developed. Deflection criteria were difficult to establish because of the early failure and erratic deflection behaviour of these sections.

9) Soil cement and lean concrete bases gave a poorer performance at a given deflection level under the intermediate 70 mm (2¾ in) rolled asphalt surfacing than under the 100 mm surfacing, while no similar significant difference was obtained on the wet-mix bases.

10) The spring deflections measured immediately after the abnormally severe winter of 1963 were generally appreciably greater than those of the preceding years. The increase were most marked in the weaker sections surviving at that time. In these the former deflection levels were rarely re-established and the critical lives probably shortened. However the overall deflection performance relationships for the groups of sections were not noticeably distorted by the effects of this winter. As the Alconbury Hill site is characterized by an abnormally high winter water-table it is therefore reasonable to conclude that abnormally severe winters will also not affect deflection performance relationships for roads with more normal water-table conditions.

11) A performance equivalence was established between asphalt and sand sub-base which was dependent on the ratio of the layer thicknesses, unlike the fixed ratio assumed in the analysis of the results of the A.A.S.H.O. road test. The assumed value however was close to the mean value of Alconbury Hill.

Comparison of the results on the basis of performance equivalence indicated inferior performance by the weaker sections at the A.A.S.H.O. site, atributable to the more severe winter conditions there. The thicker pavements which were relatively unaffected by climatic conditions performed in a very similar manner to the British pavements.

12) The relations developed between deflection and performance and deflection and layer thicknesses for rolled asphalt bases enabled a comparison with the thickness requirements of Road Note No. 29 to be made. These showed good agreement over the experimentally verified range.

13) Prediction of the unexpired life of pavements from deflection measurements made at any time before critical conditions develop can be made from charts which take into account the changes in deflection with time and traffic. An example for pavements with rolled asphalt bases is given.

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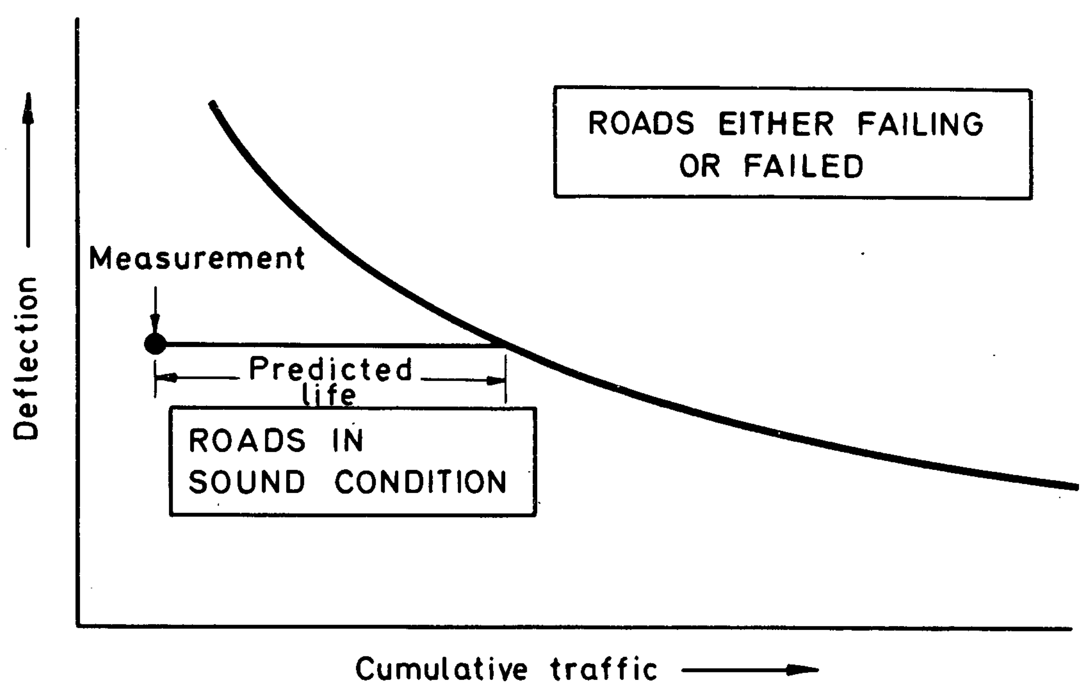

Fig. 1. THE DEFLECTION CRITERION CURVE

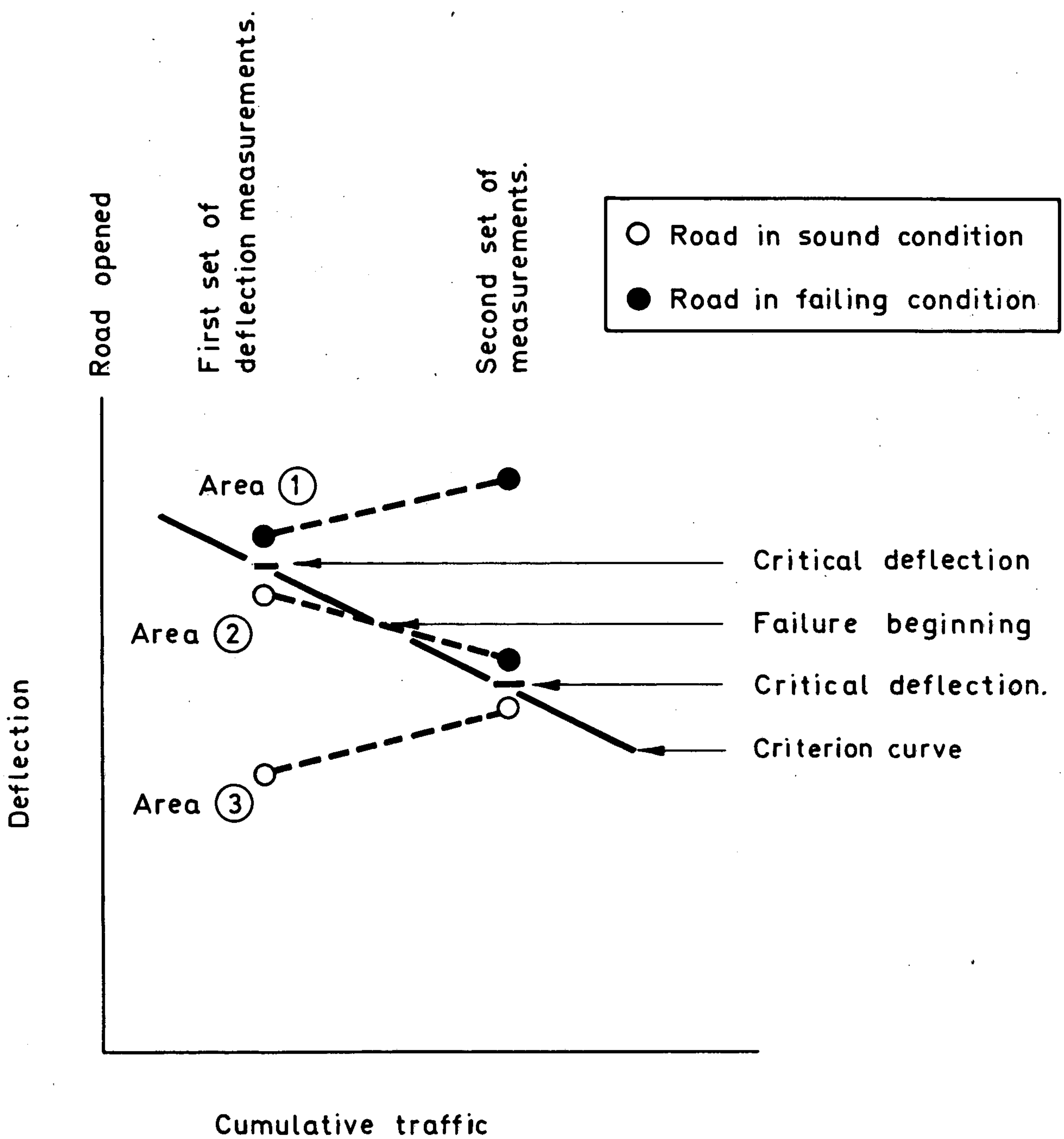

Fig. 2. THE INTER-RELATIONSHIP BETWEEN DEFLECTION, TRAFFIC & ROAD PERFORMANCE.

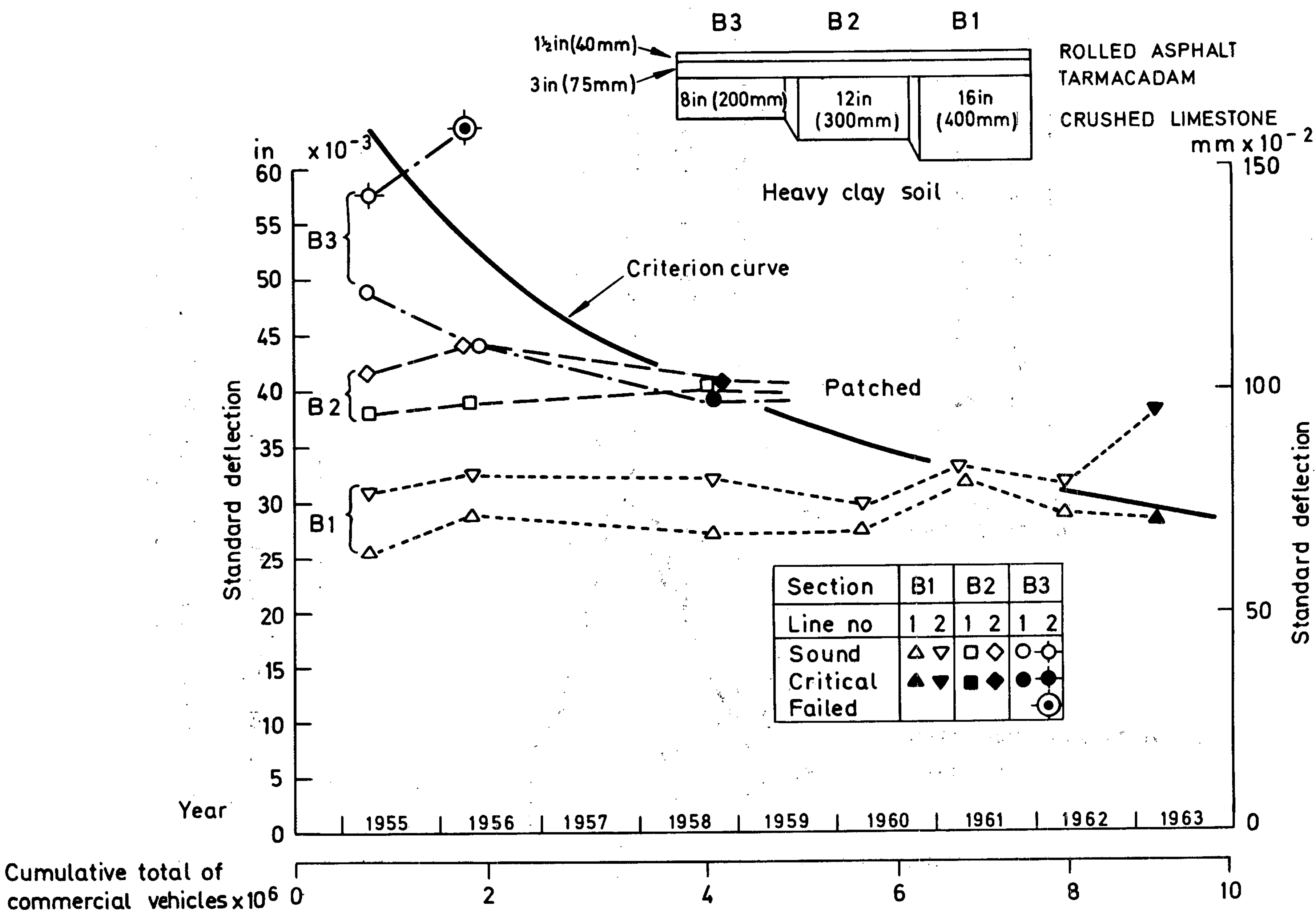

Fig 3 DEFLECTION HISTORY OF THREE EXPERIMENTAL SECTIONS AT CAMBRIDGE (GLOS) A 38

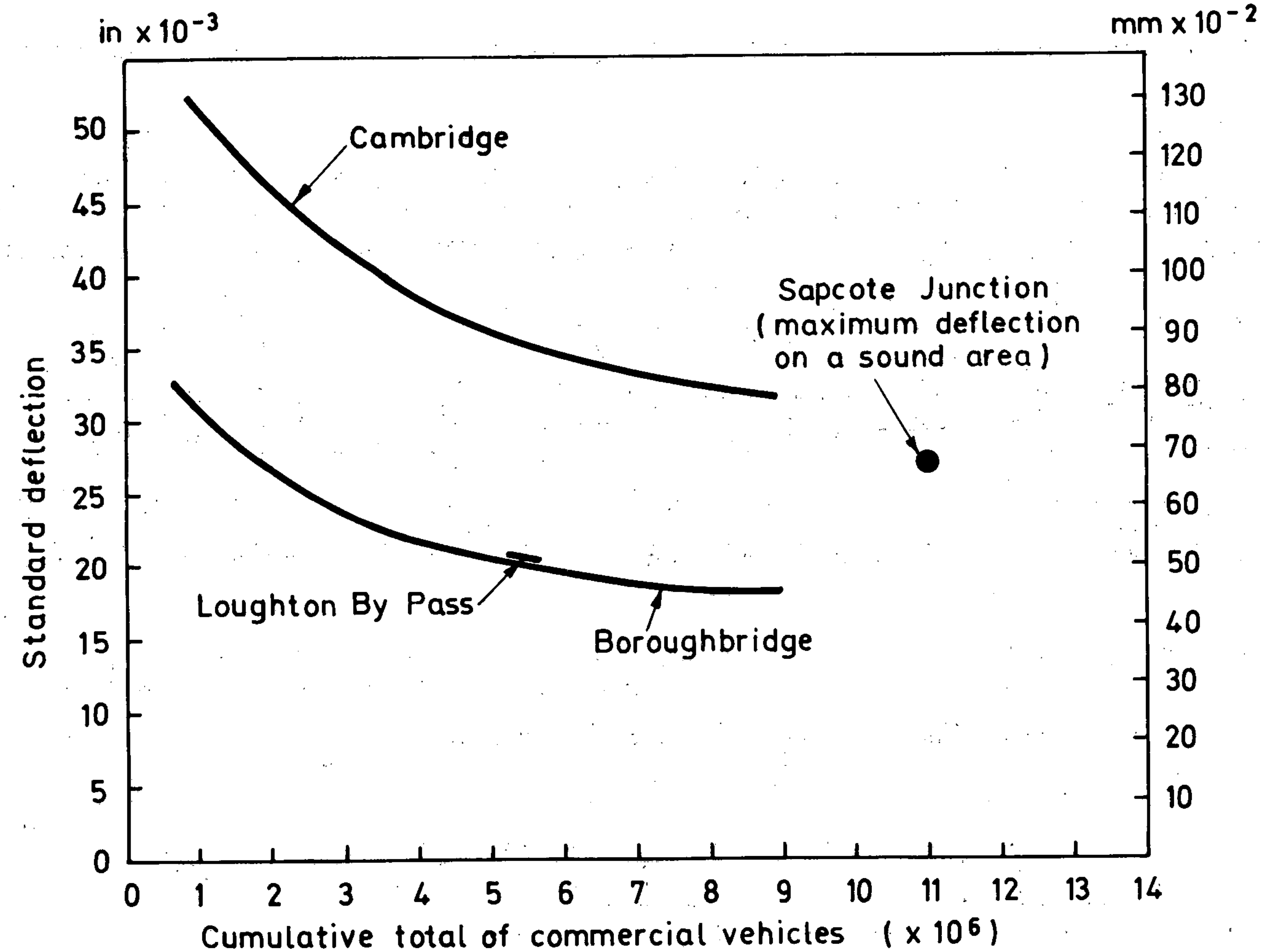

Fig. 4. COMPARISON OF DEFLECTION CRITERIA FOR CRUSHED STONE BASES ON THE BASIS OF CUMULATIVE TRAFFIC.

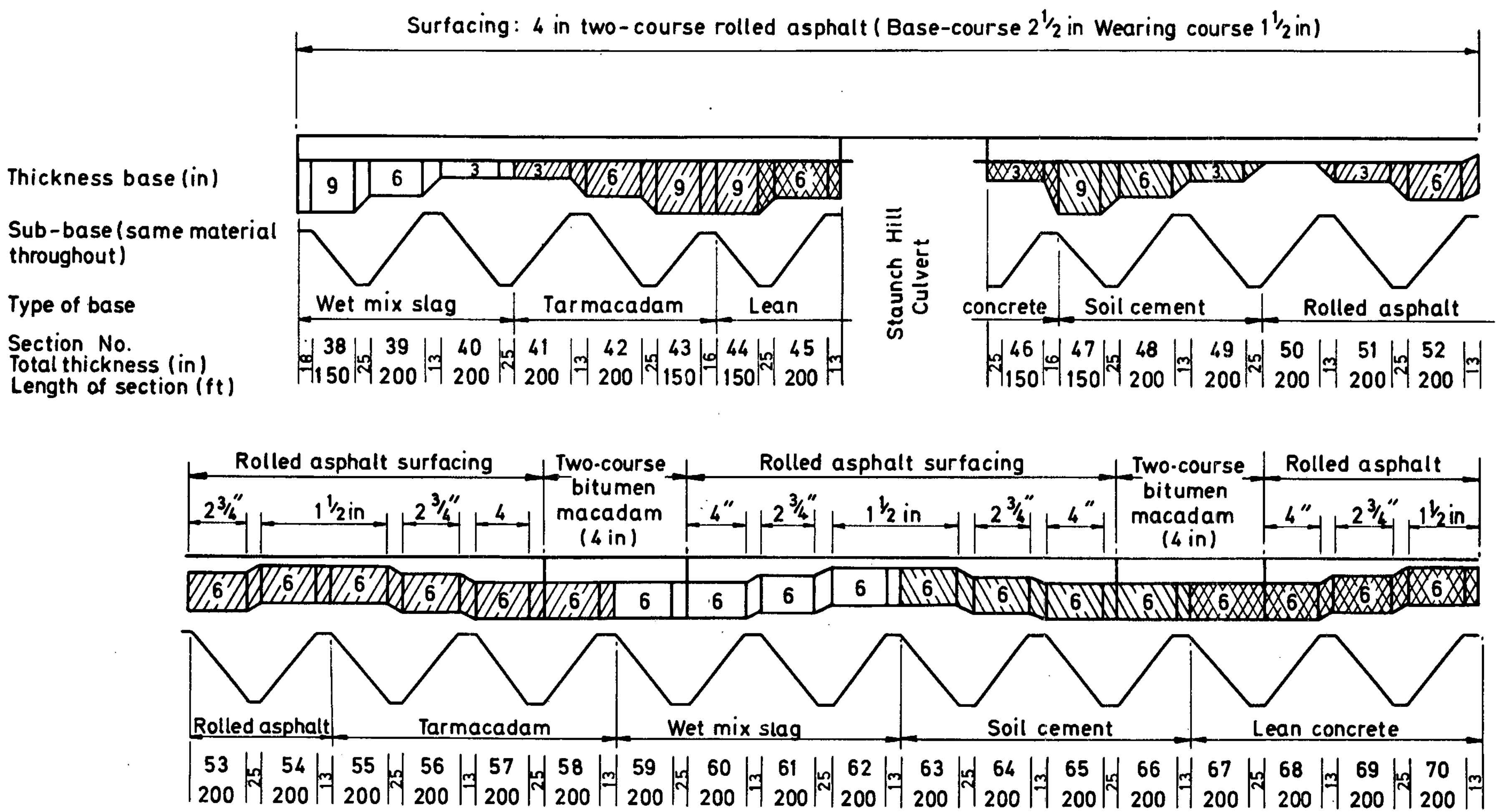

Fig.5. LAYOUT OF EXPERIMENTAL SECTIONS OF FLEXIBLE CONSTRUCTION - ALCONBURY HILL A.1.

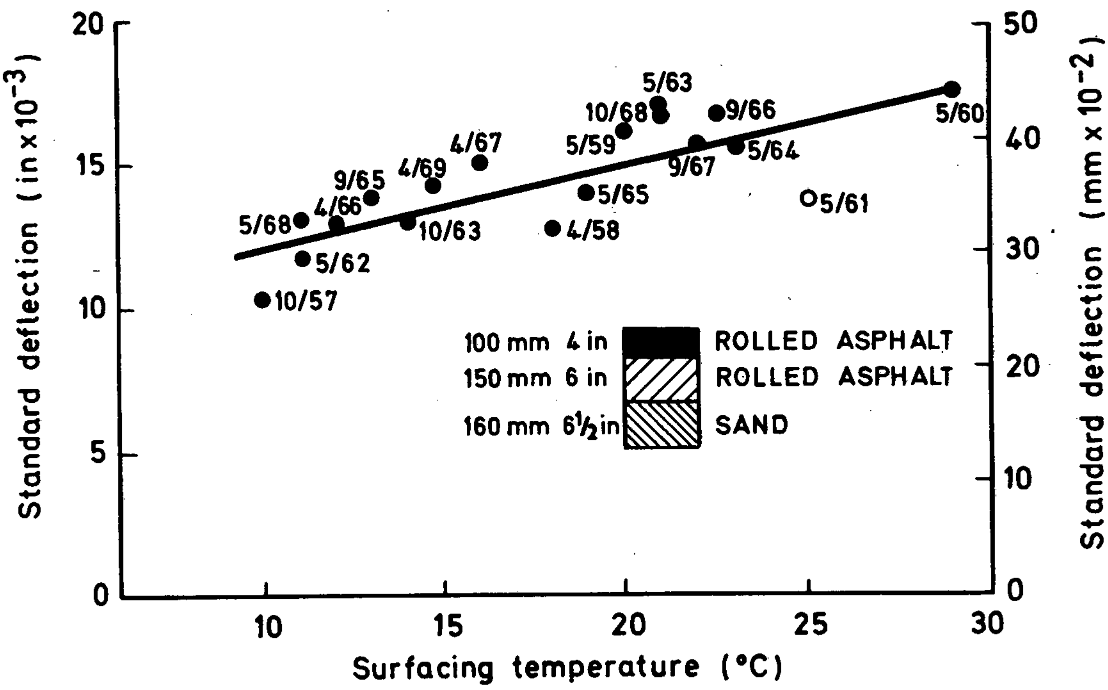

Fig.6 TEMPERATURE SUSCEPTIBILITY FOR SECTION OF VIRTUALLY CONSTANT STRUCTURAL STIFFNESS

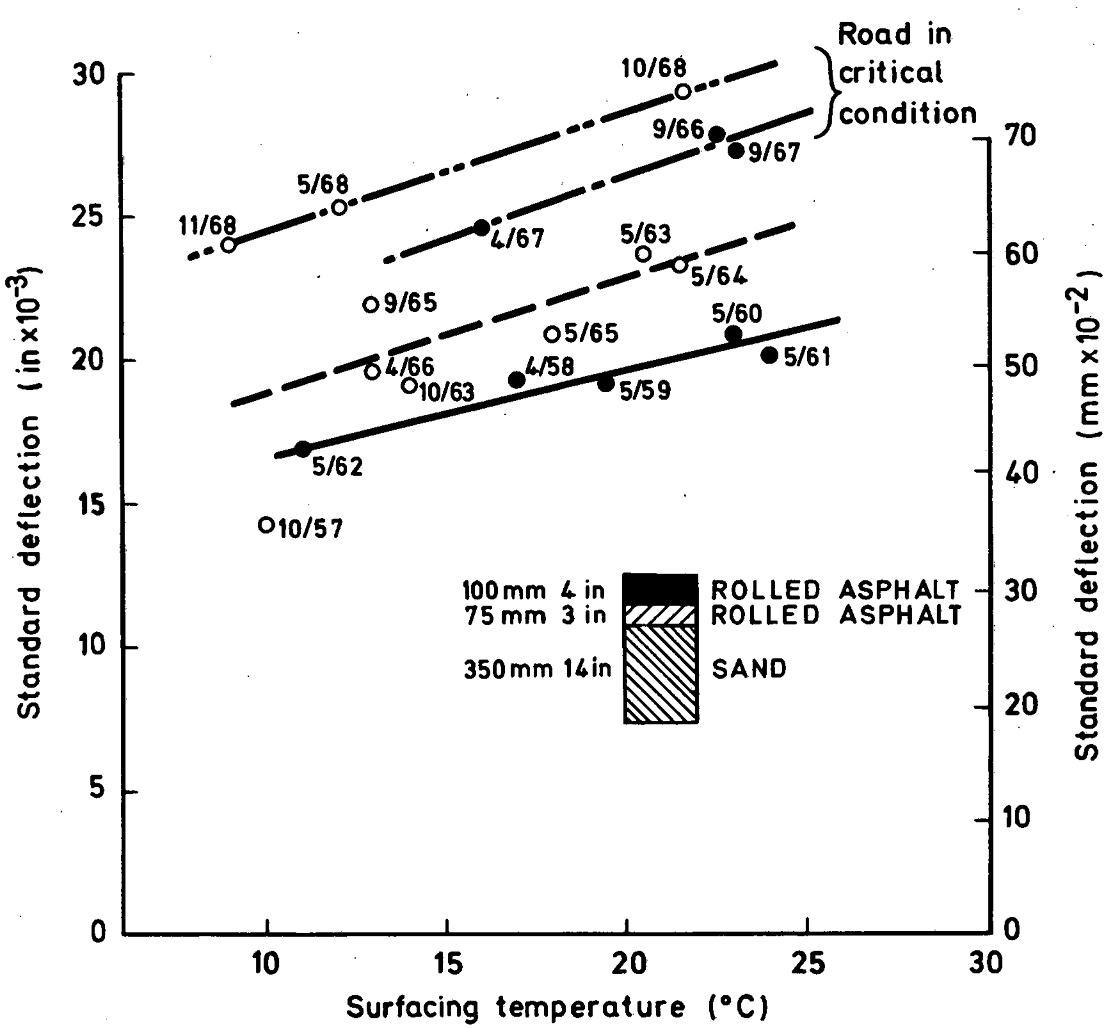

Fig. 7. TEMPERATURE SUSCEPTIBILITY FOR SECTION WITH DETERIORATING STRUCTURAL STIFFNESS

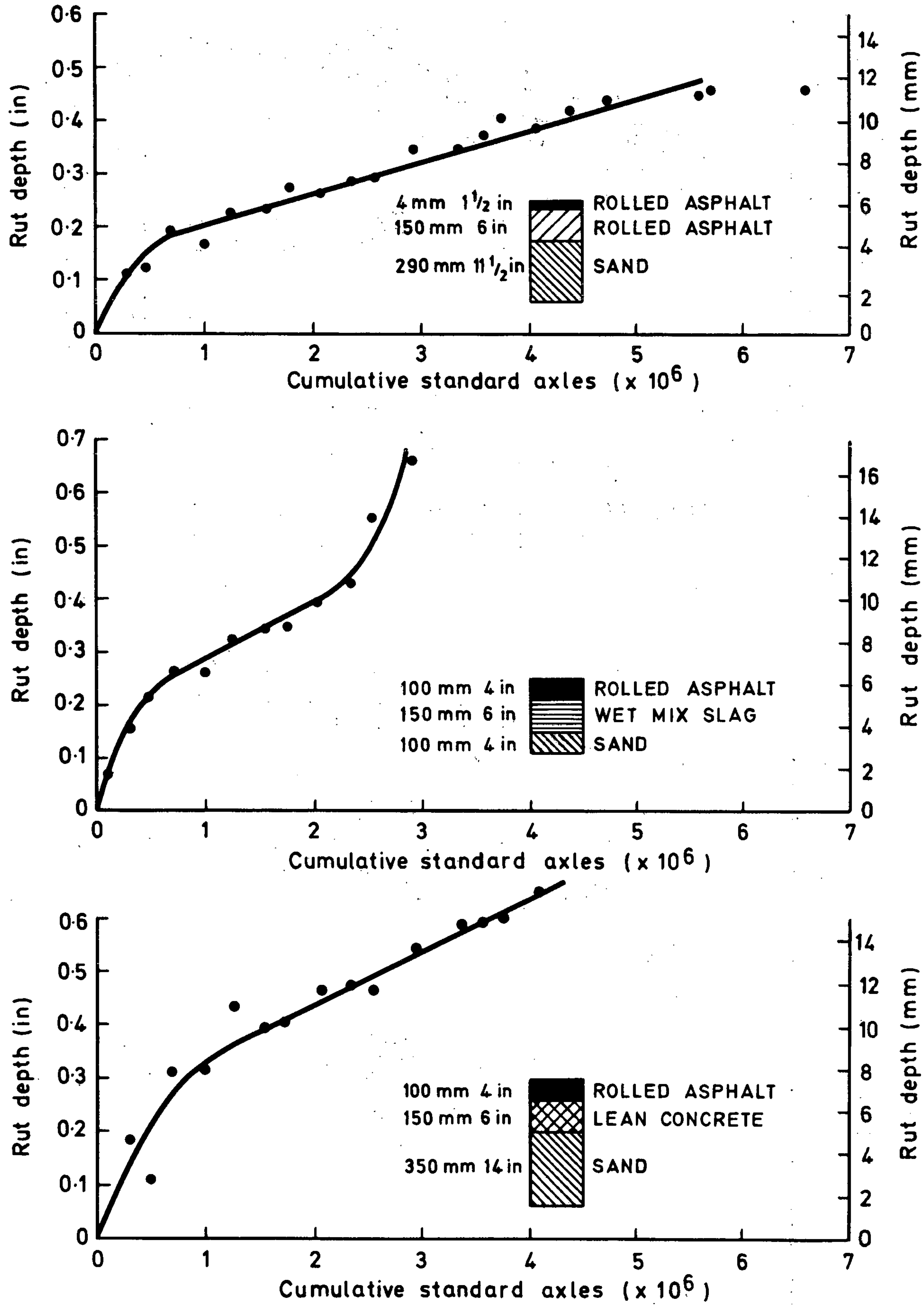

Fig.8. TYPICAL RELATIONS BETWEEN RUT DEPTH AND CUMULATIVE TOTAL OF STANDARD AXLES

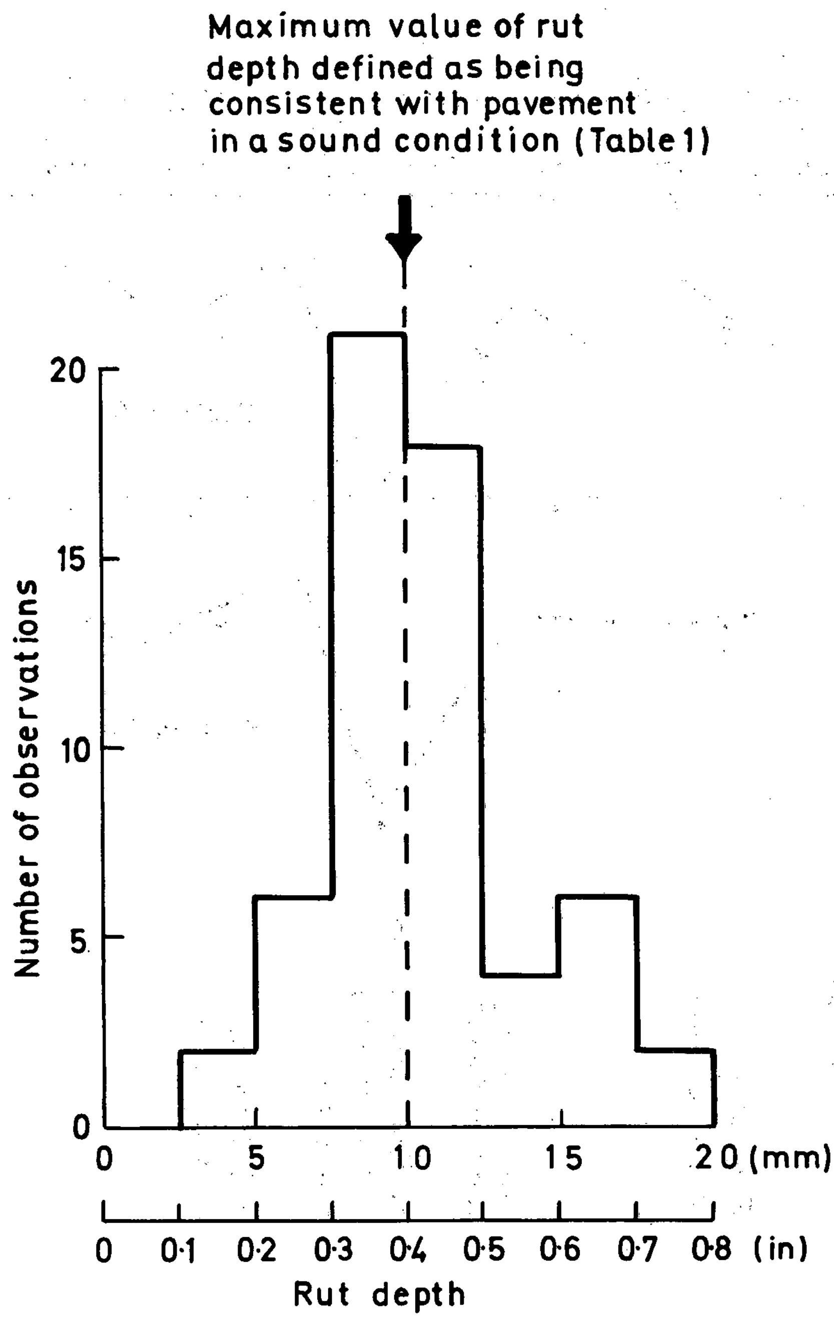

Fig.9. DISTRIBUTION OF RUT DEPTHS IN THE NEARSIDE WHEELPATH AT WHICH SIGNIFICANT CRACKING OF THE SURFACING COMMENCED

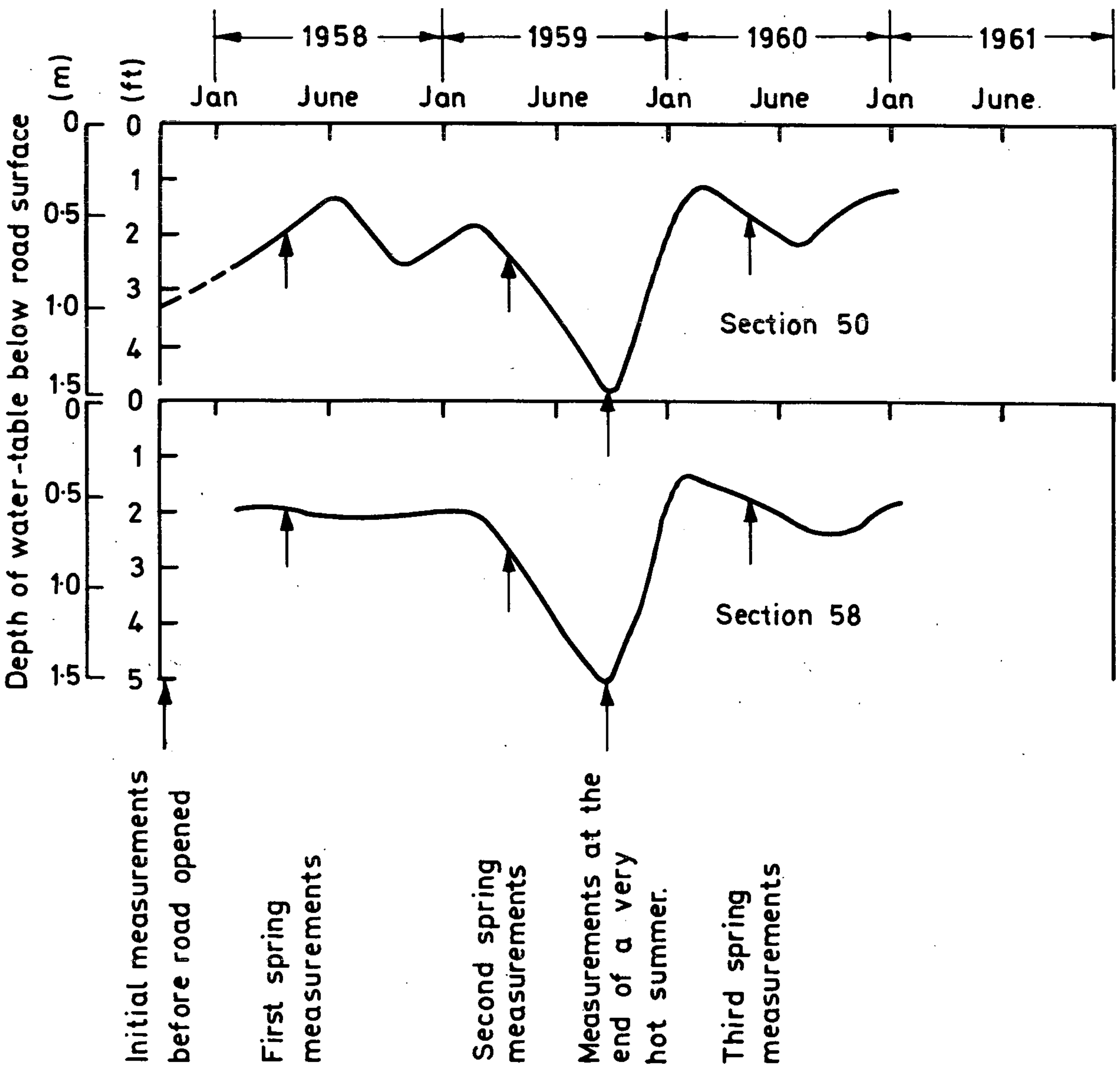

Fig. 10. TWO TYPICAL RECORDS OF VARIATION IN WATER TABLE LEVELS AT ALCONBURY HILL

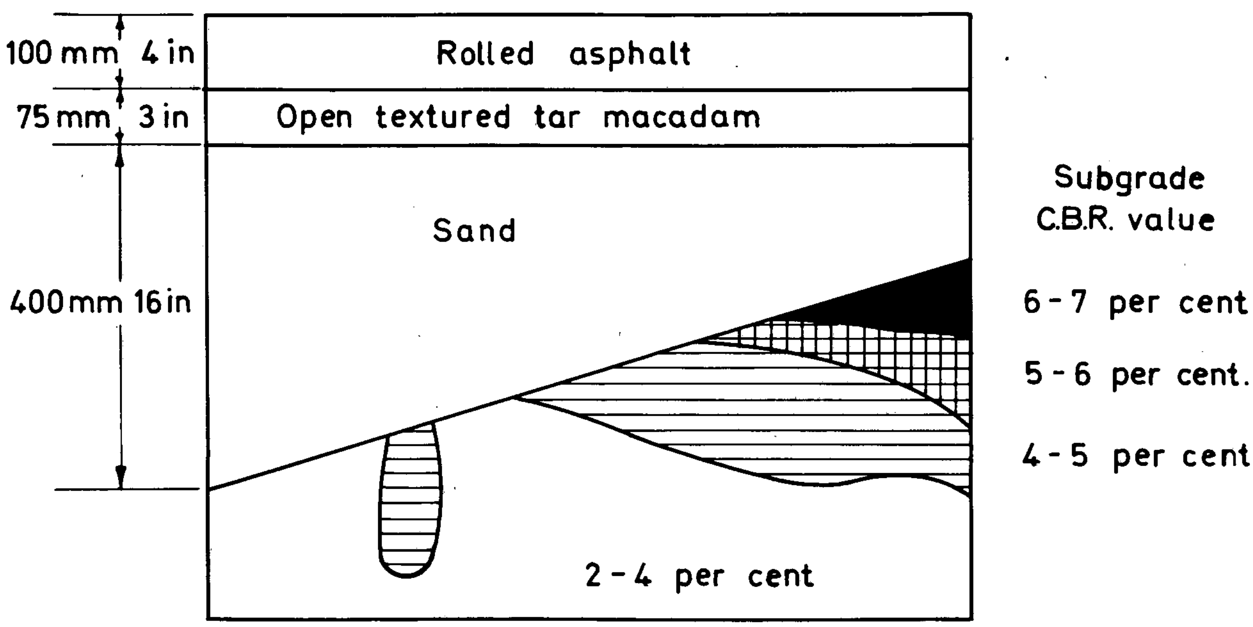

Fig. 11. VARIATION OF SUB - GRADE STRENGTH UNDER SECTION 41.

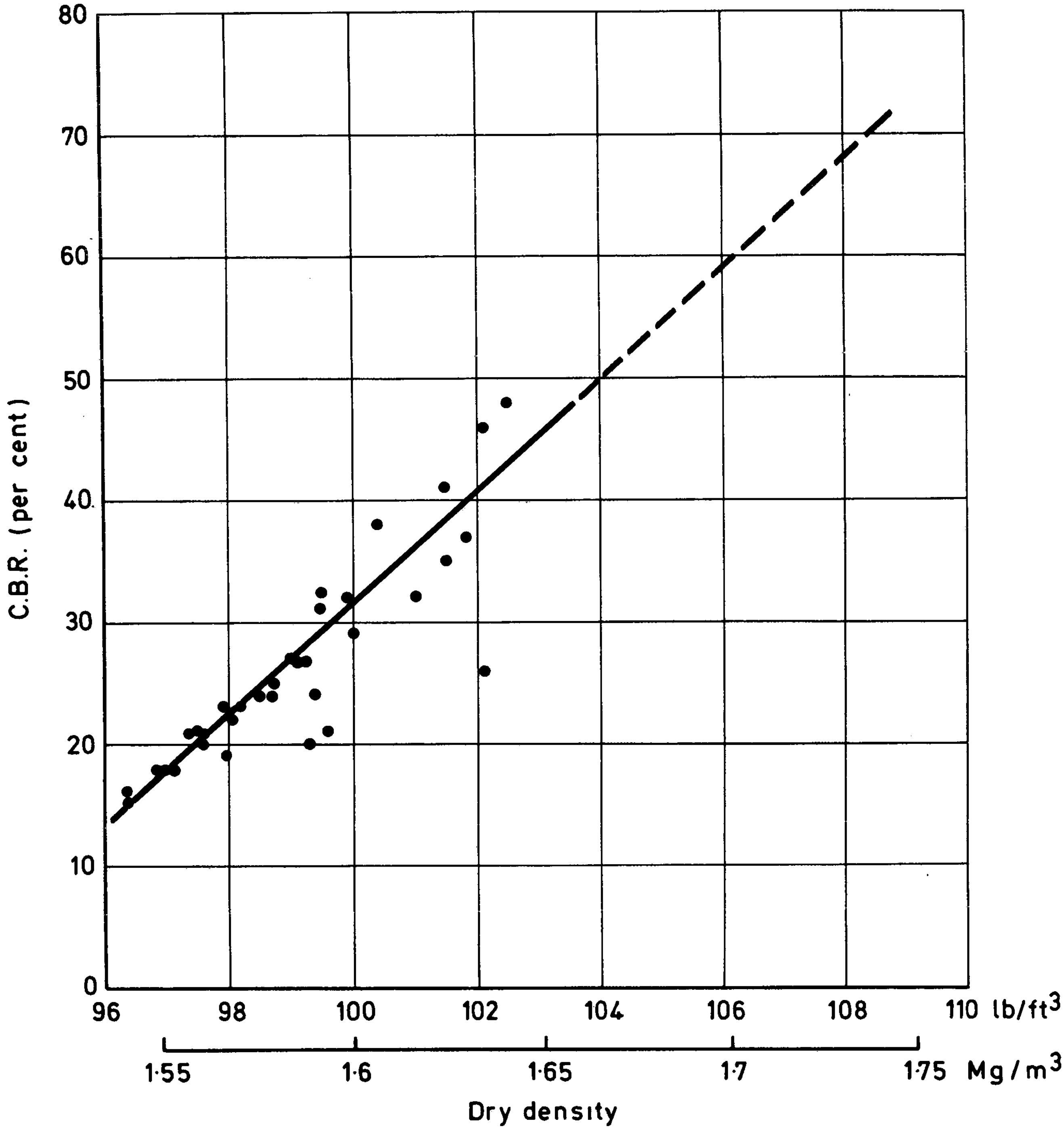

Fig. 12. RELATION BETWEEN IN-SITU C.B.R. AND DRY DENSITY OF THE SAND SUB-BASE AT ALCONBURY HILL

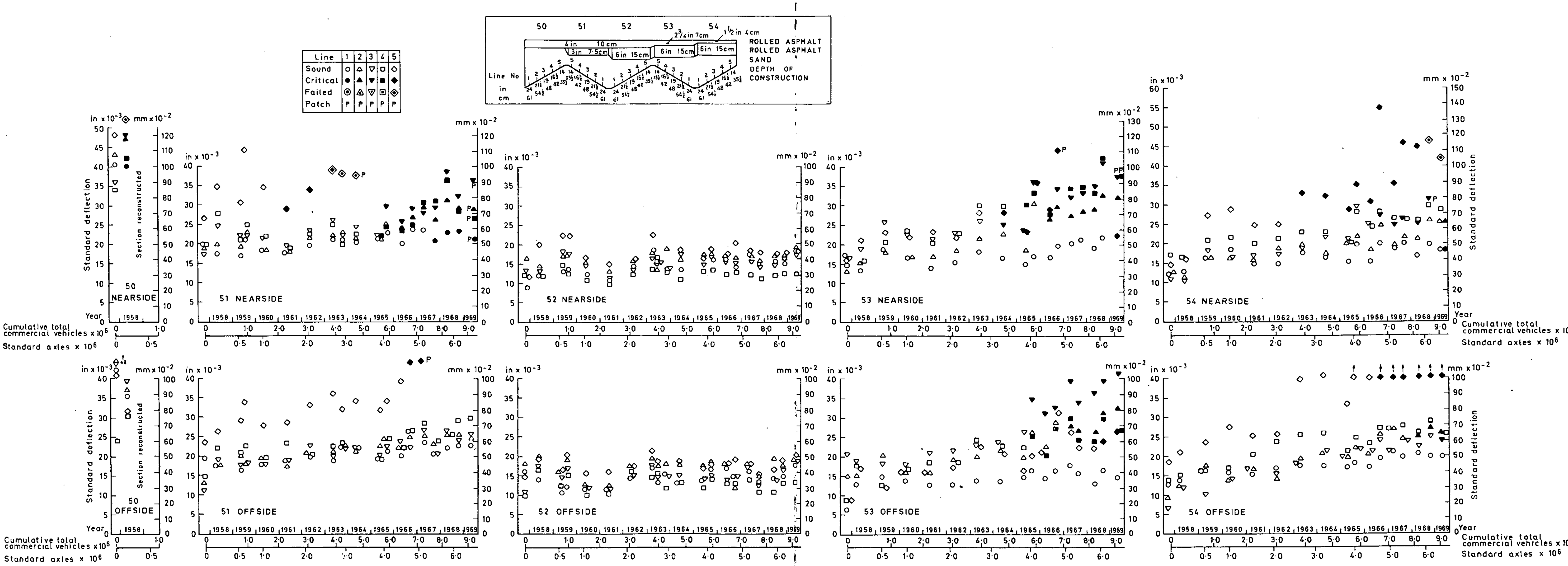

Fig. 13. DEFLECTION HISTORY OF SECTIONS WITH ROLLED ASPHALT BASES AT ALCONBURY HILL

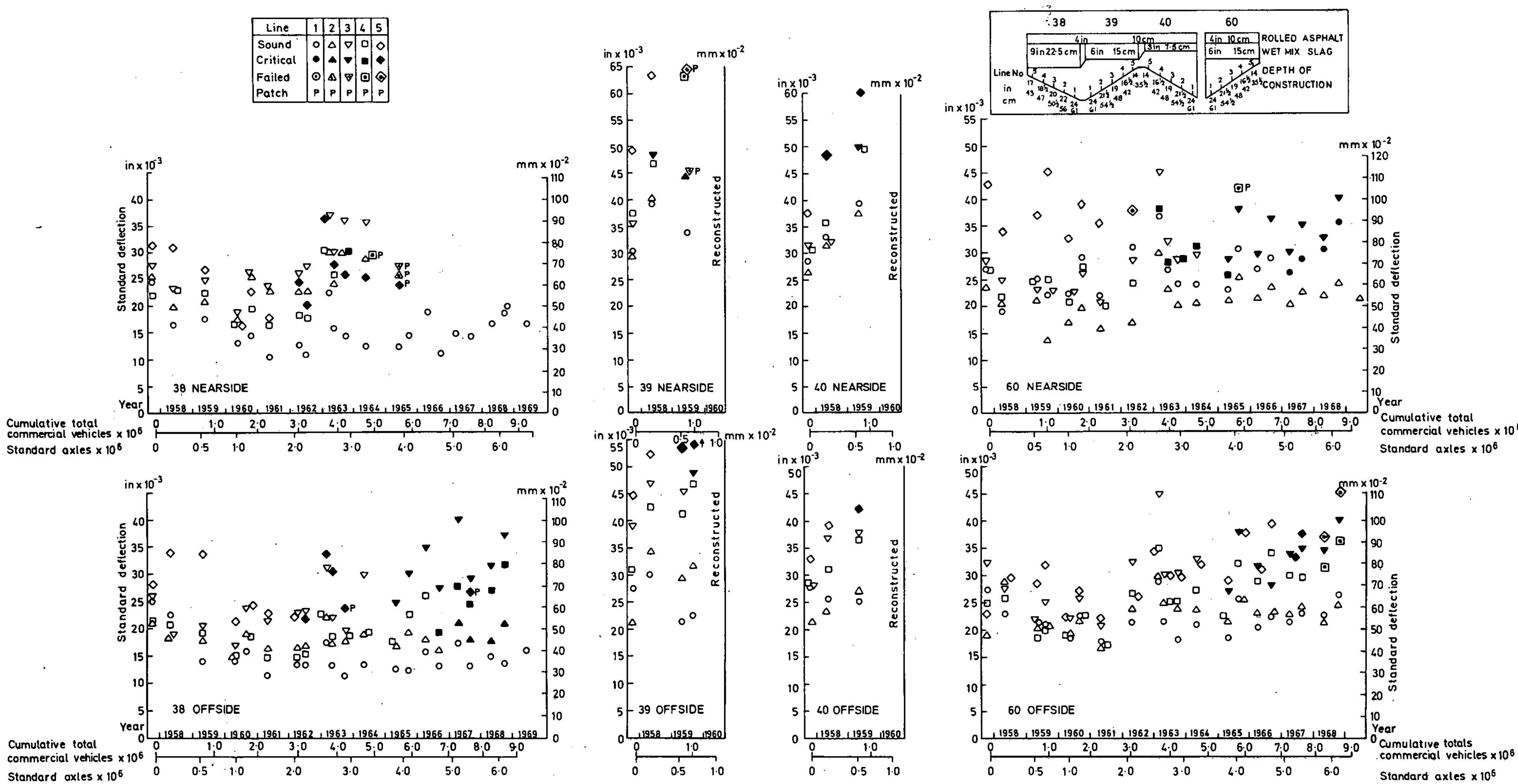

Fig.14. DEFLECTION HISTORY OF SECTIONS WITH WET-MIX SLAG BASES AT ALCONBURY HILL (100mm ASPHALT SURFACING)

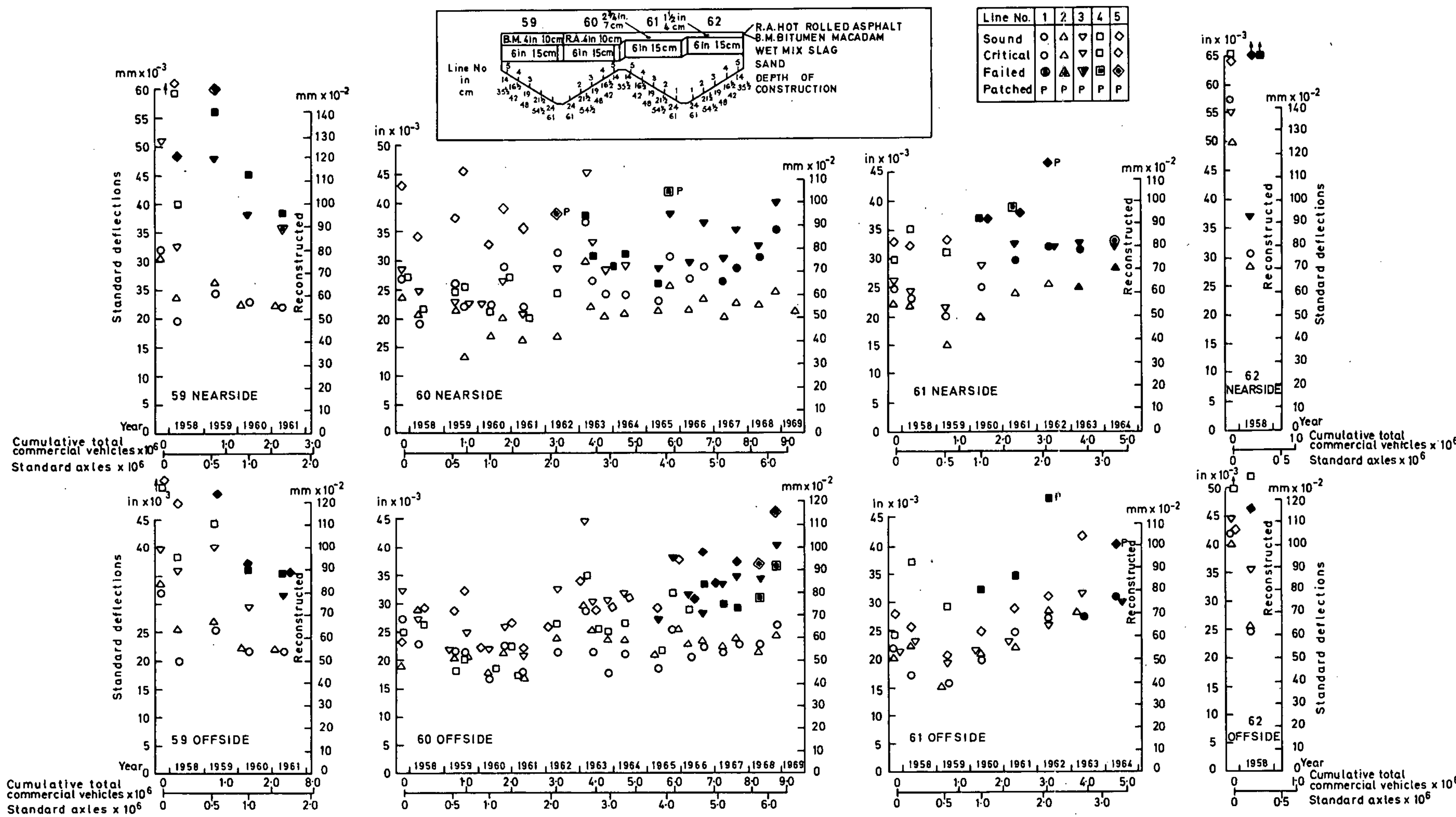

Fig. 15 DEFLECTION HISTORY OF SECTIONS WITH WET-MIX SLAG BASES AT ALCONBURY HILL (150mm BASES)

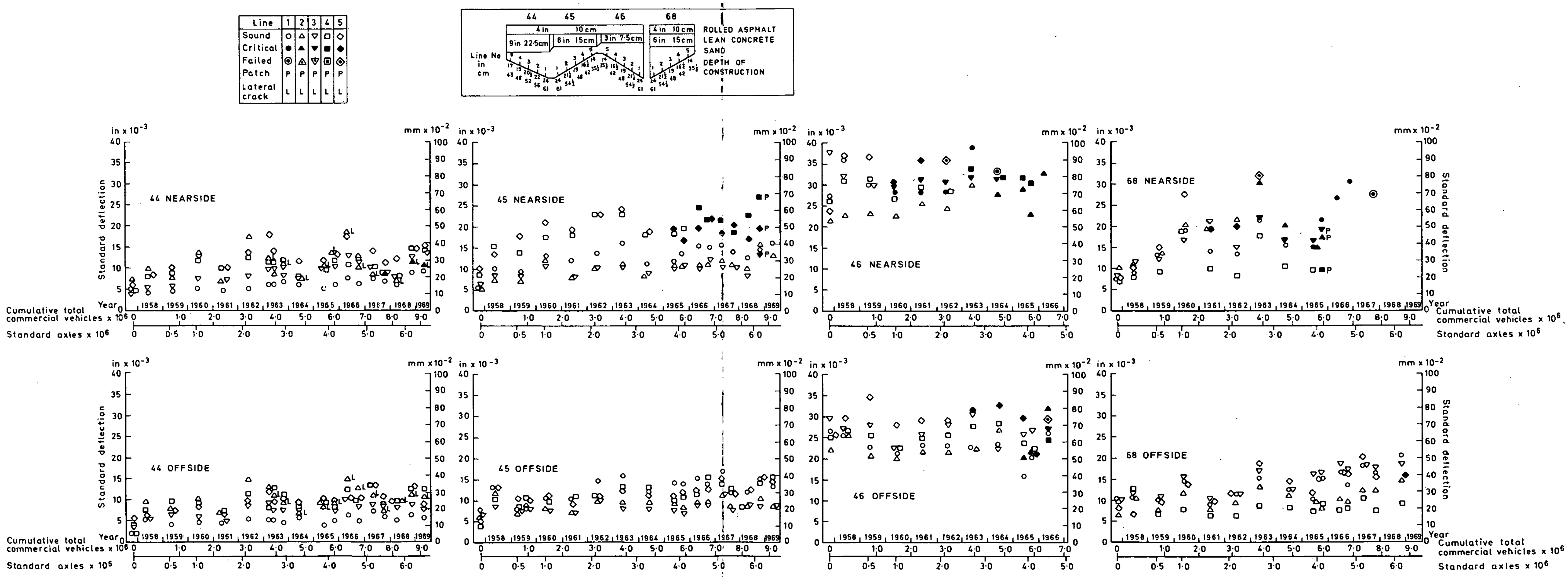

Fig. 16. DEFLECTION HISTORY OF SECTIONS WITH LEAN CONCRETE BASES AT ALCONBURY HILL (100mm ASPHALT SURFACINGS)

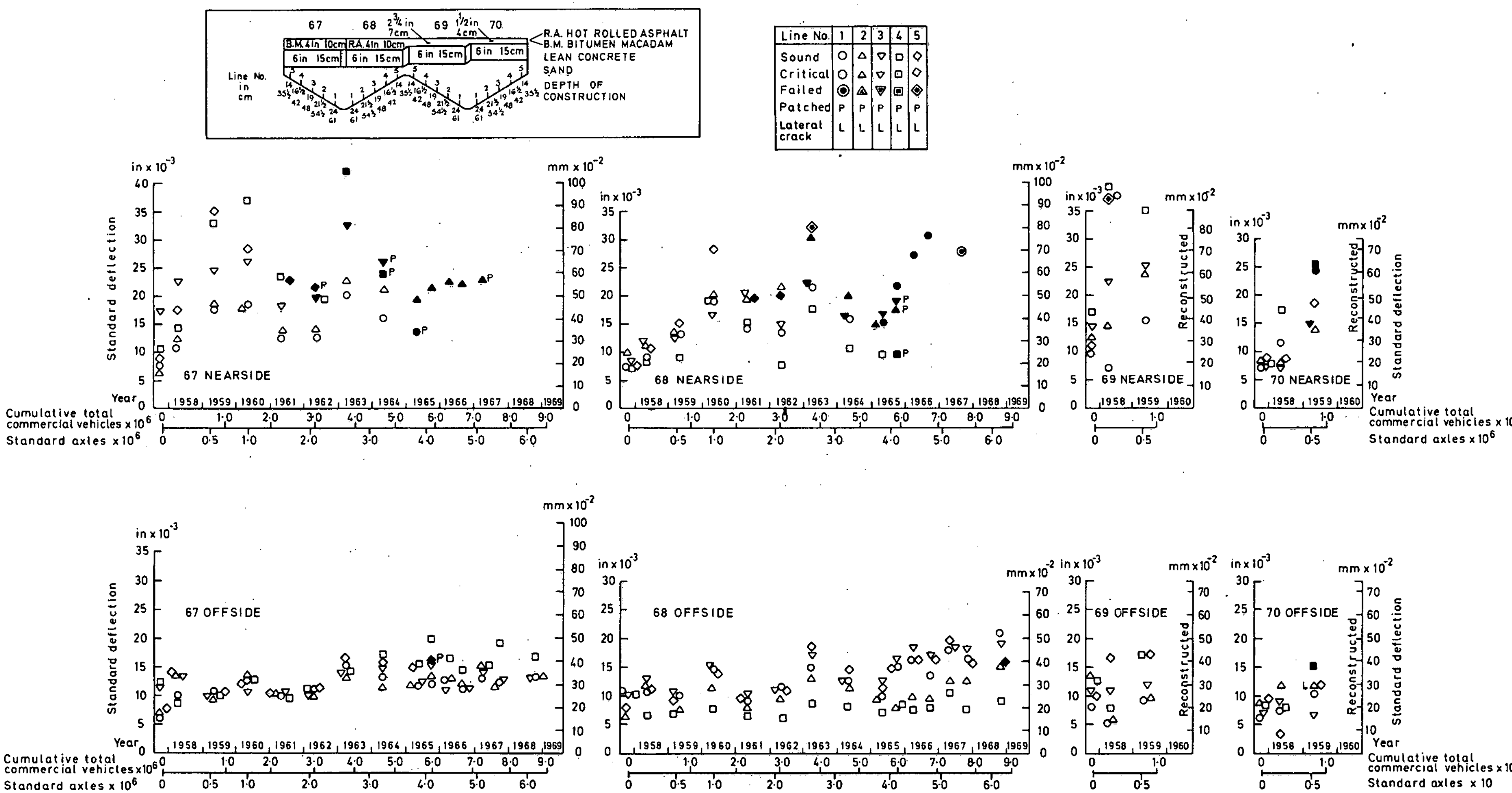

Fig. 17. DEFLECTION HISTORY OF SECTIONS WITH LEAN CONCRETE BASES AT ALCONBURY HILL (150mm BASES)

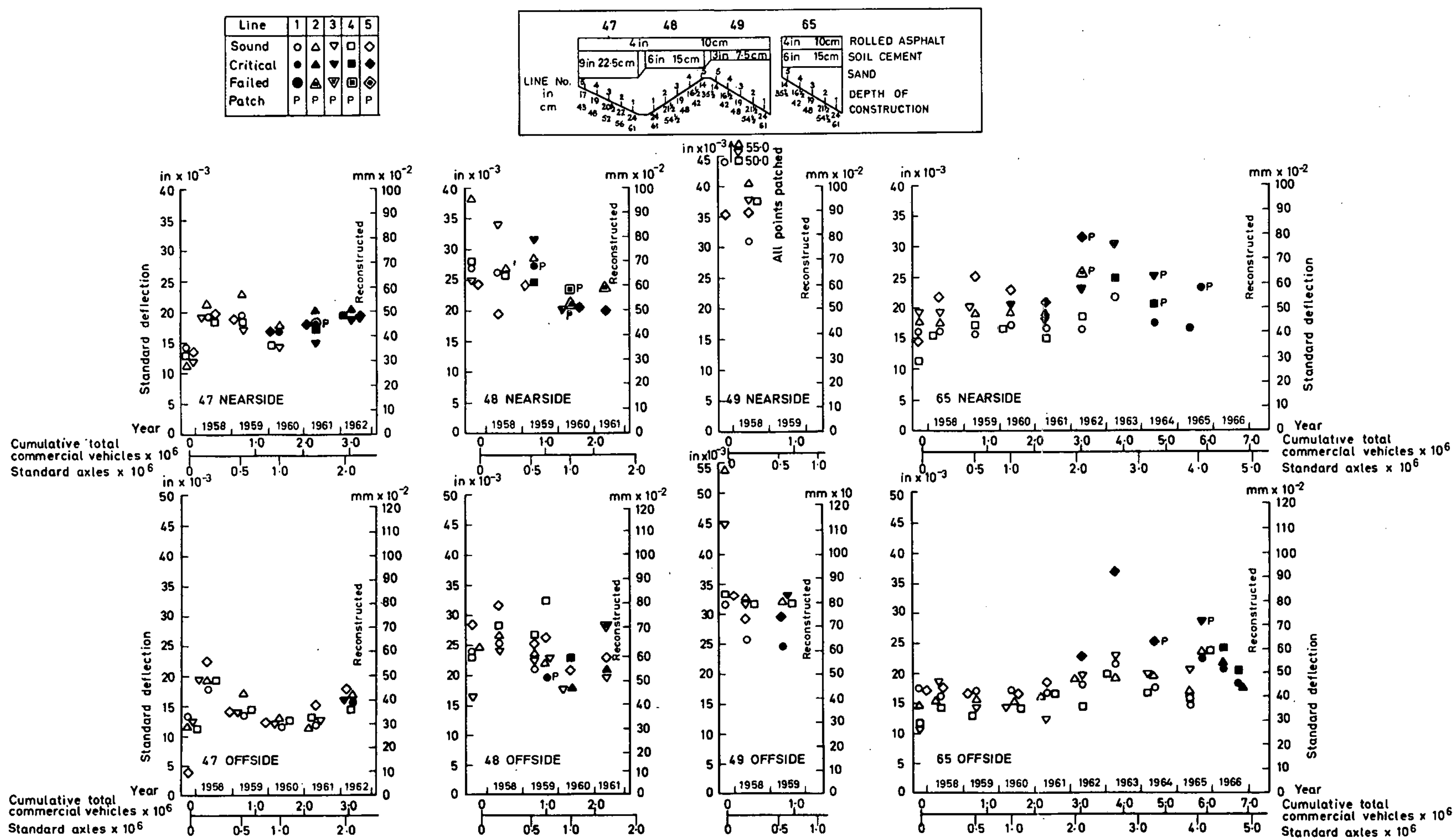

Fig.18. DEFLECTION HISTORY OF SECTIONS WITH SOIL CEMENT BASES AT ALCONBURY HILL (100mm ASPHALT SURFACINGS)

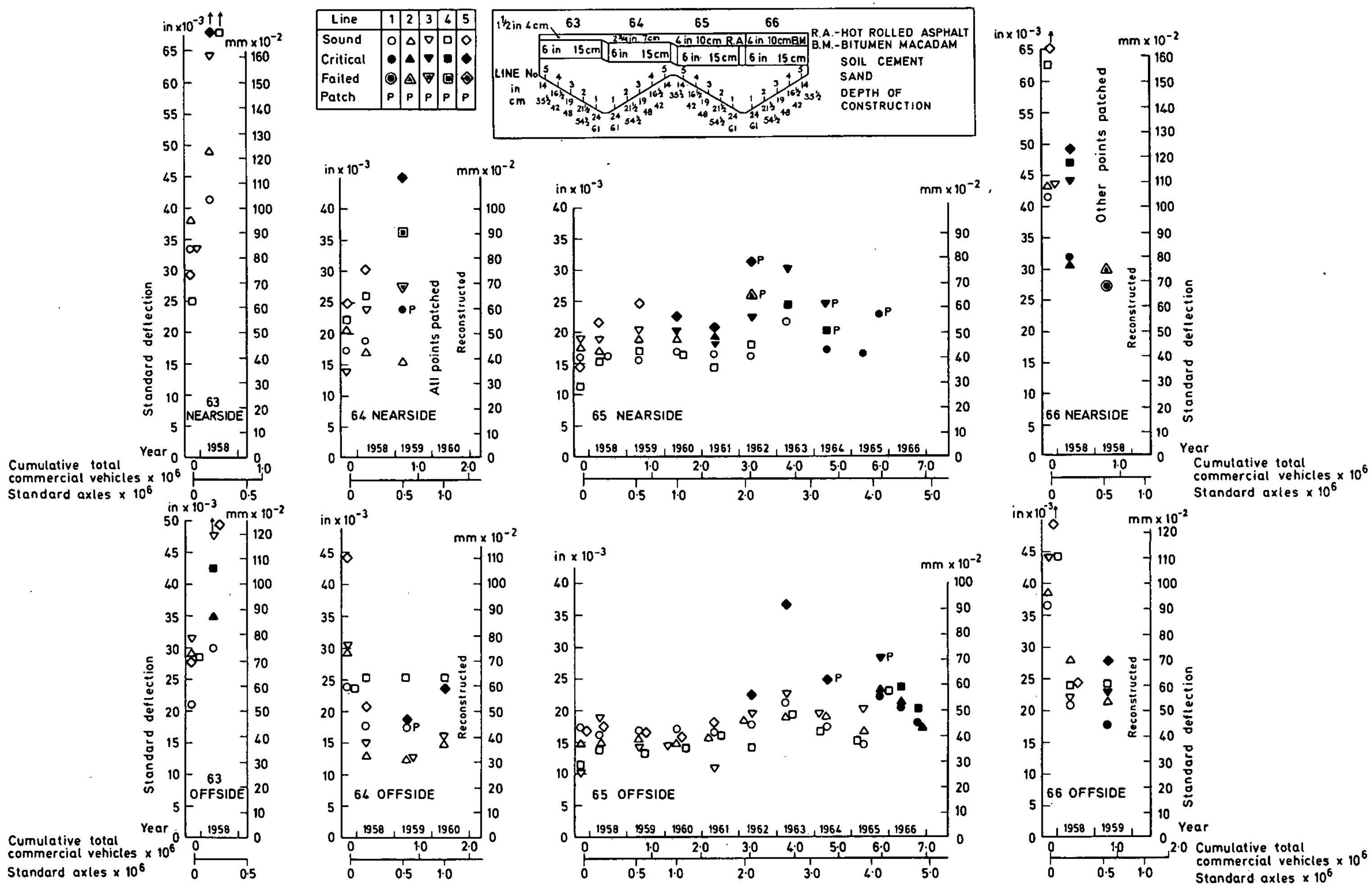

Fig. 19 DEFLECTION HISTORY OF SECTIONS WITH SOIL CEMENT BASES AT ALCONBURY HILL (150mm BASES)

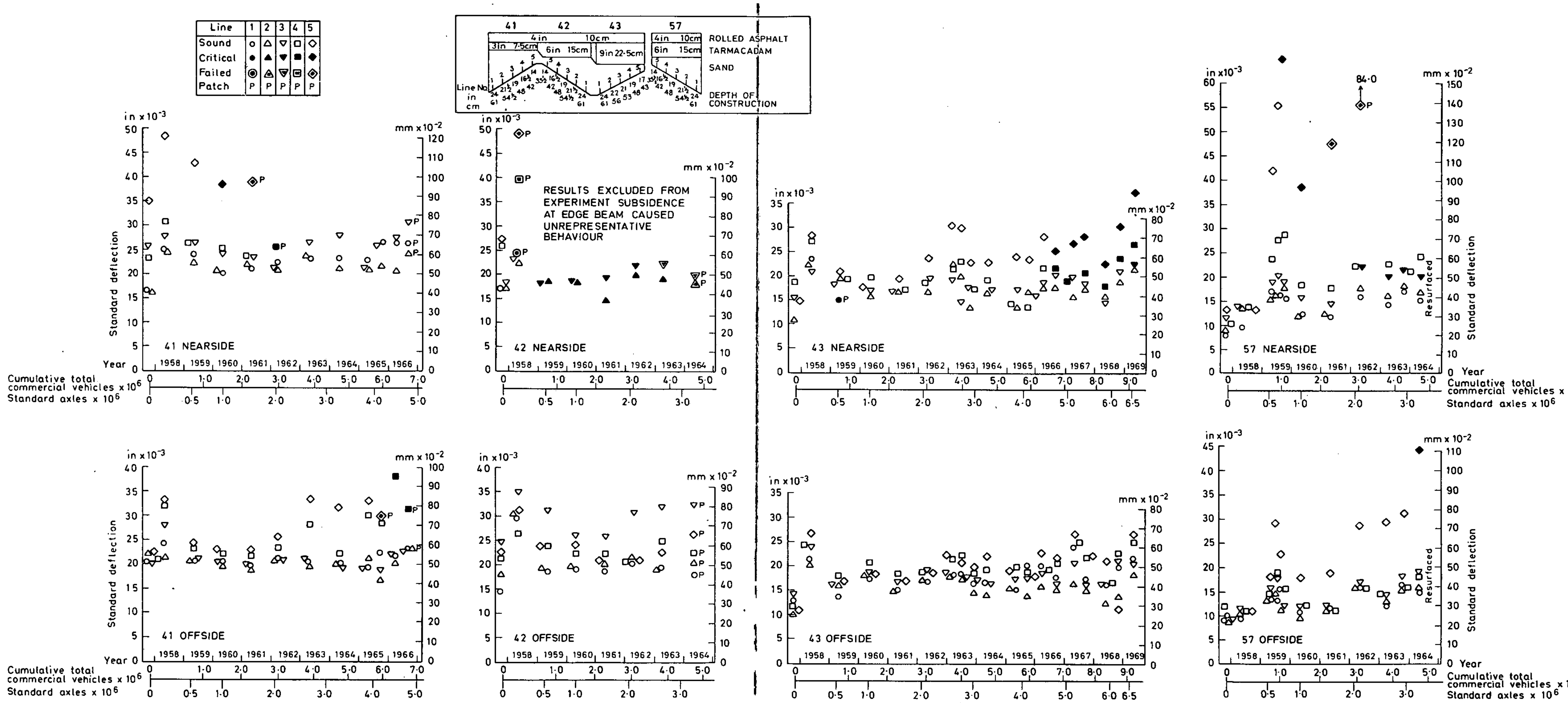

Fig. 20 DEFLECTION HISTORY OF SECTIONS WITH TARMACADAM BASES AT ALCONBURY HILL (100mm ASPHALT SURFACING)

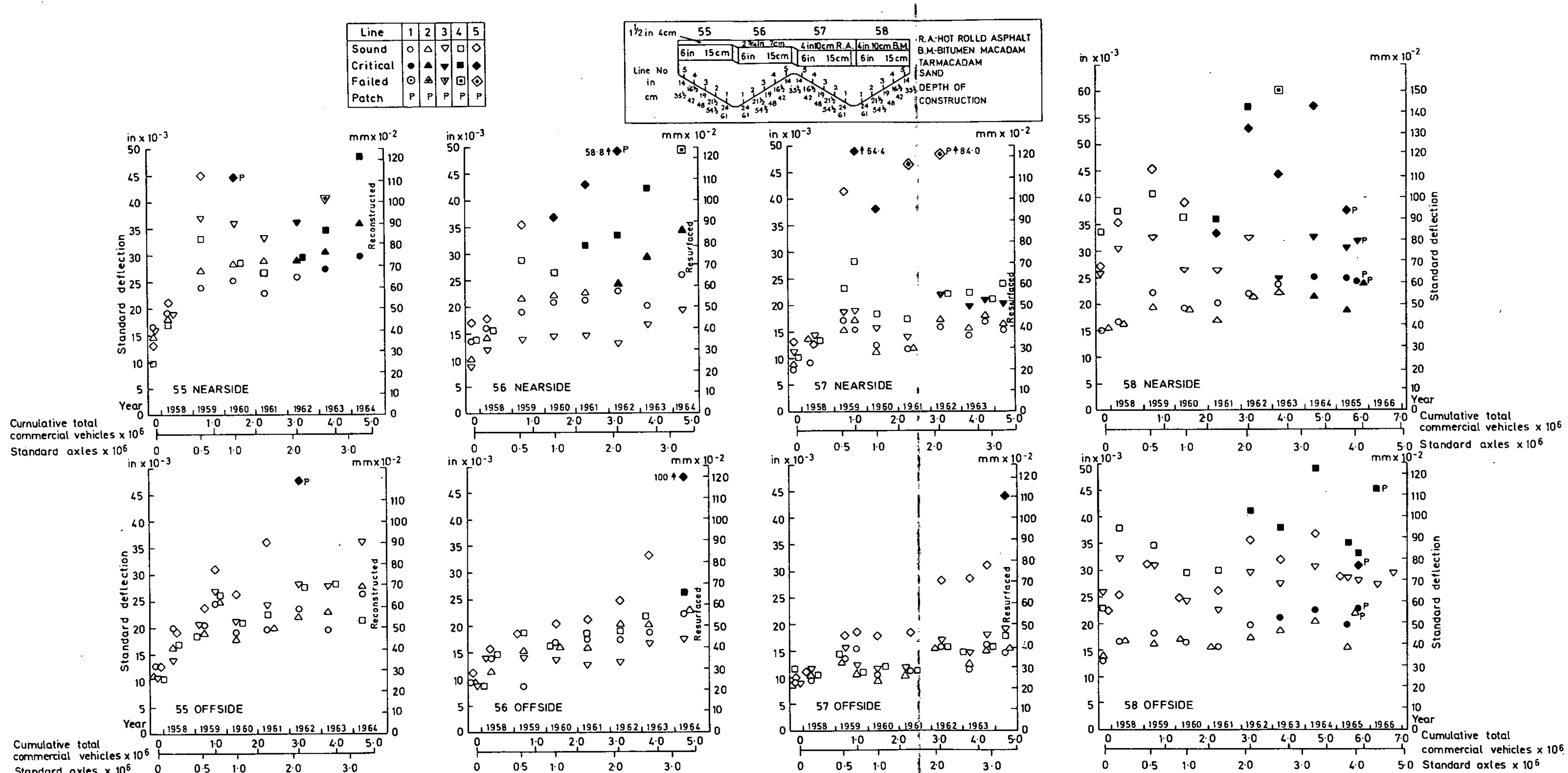

Fig. 21 DEFLECTION HISTORY OF SECTIONS WITH TARMACADAM BASES AT ALCONBURY HILL (150mm BASES)

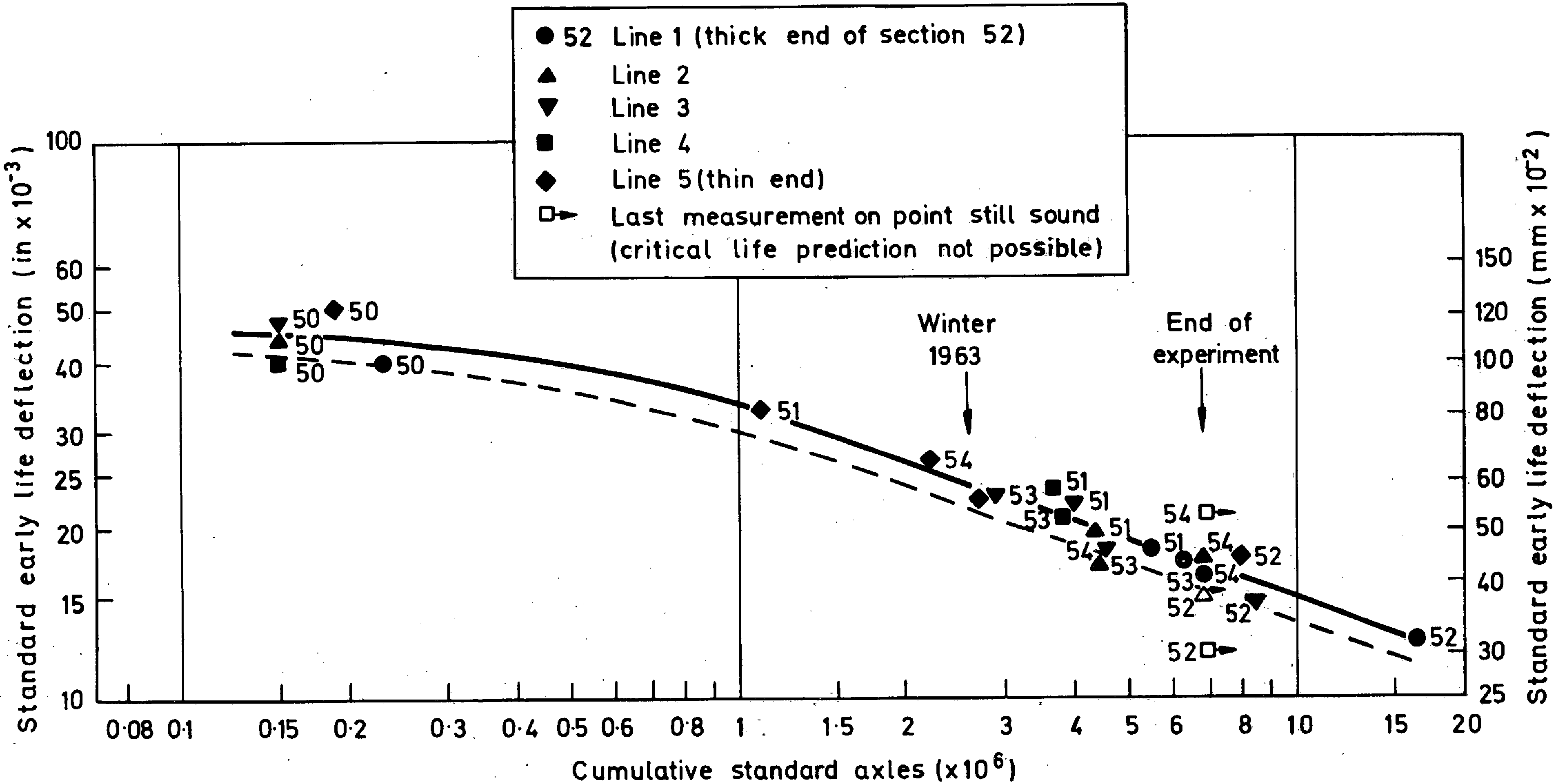

Fig. 22. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR SECTIONS WITH ROLLED ASPHALT BASES AT ALCONBURY HILL

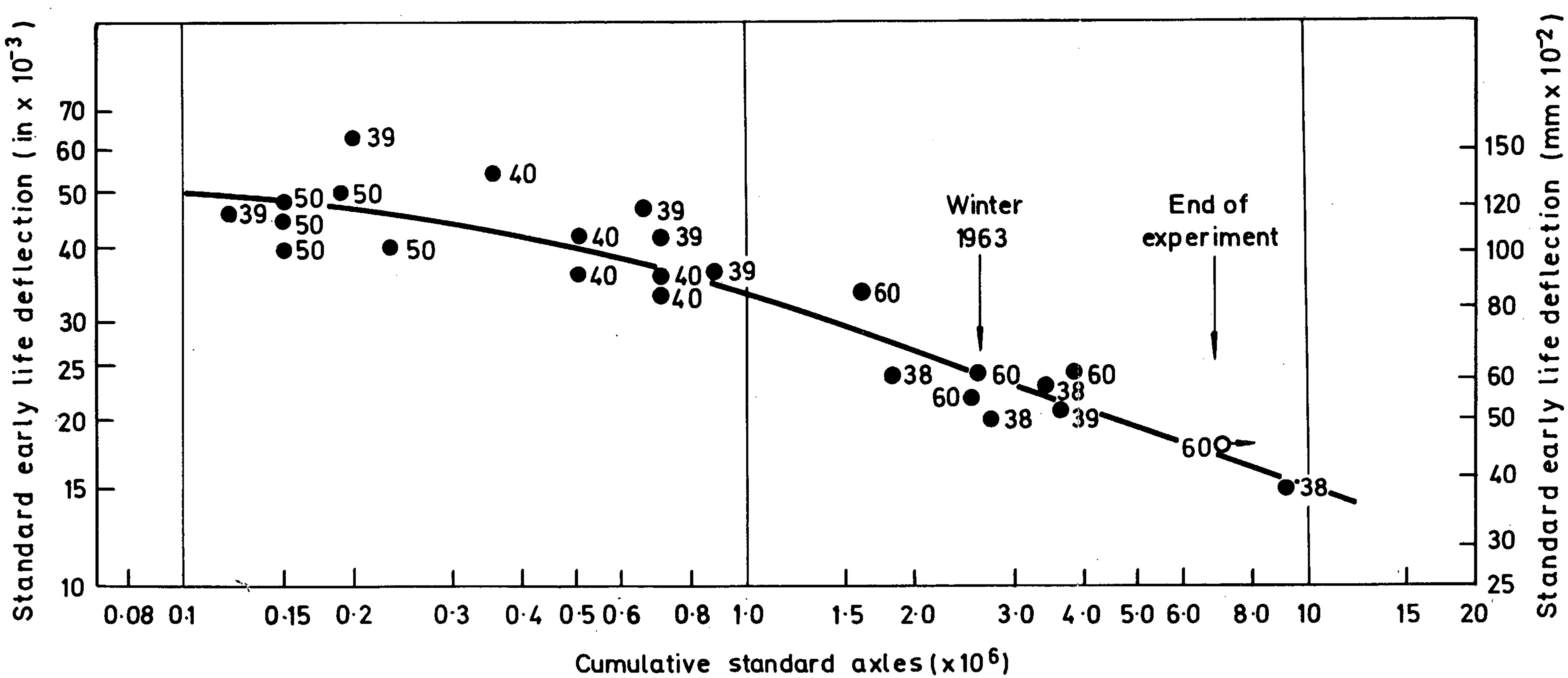

Fig. 23. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR WET-MIX SLAG BASES UNDER 100 mm ROLLED ASPHALT SURFACING AT ALCONBURY HILL

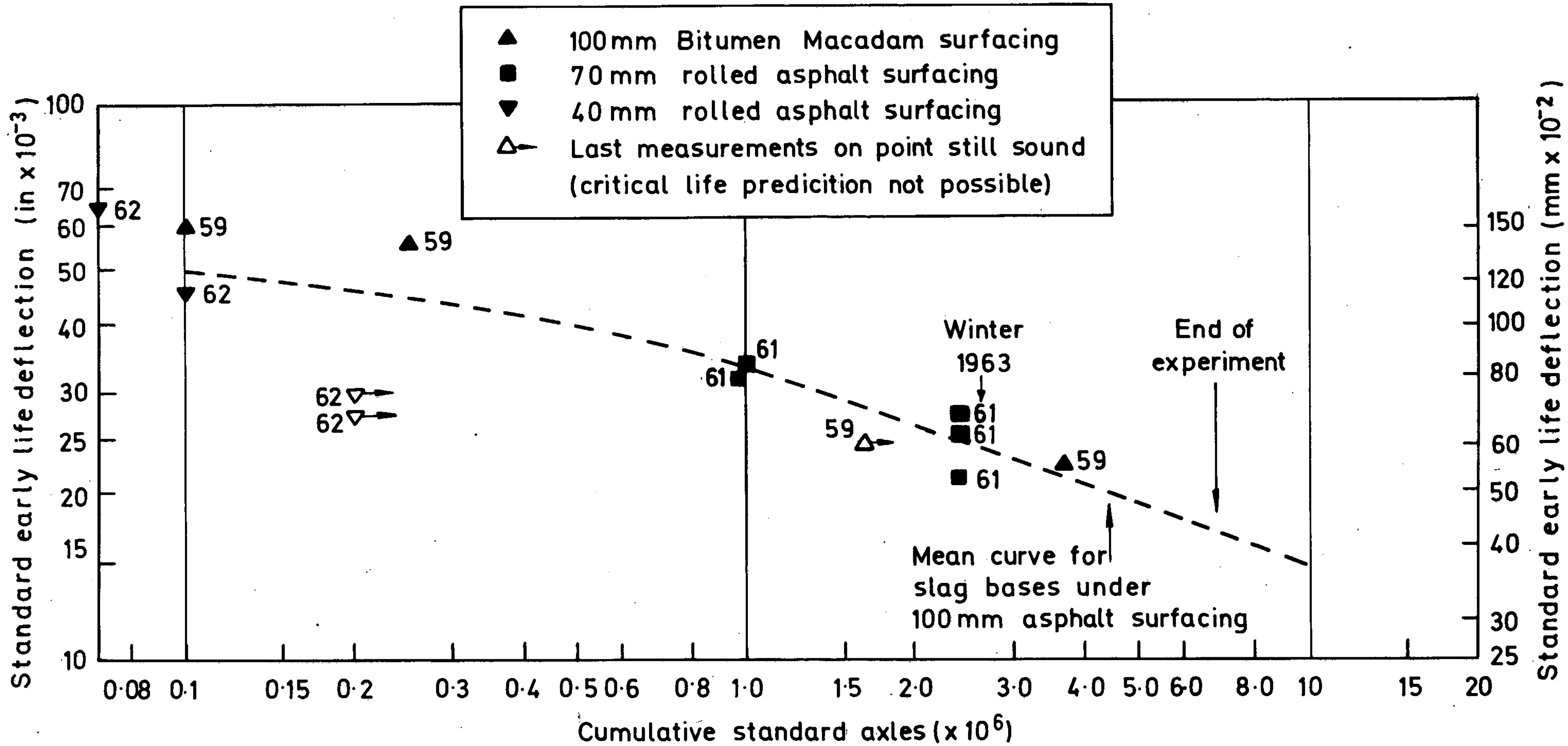

Fig. 24. RELATIONS BETWEEN DEFLECTION AND CRITICAL LIFE FOR WET-MIX SLAG BASES AT ALCONBURY HILL

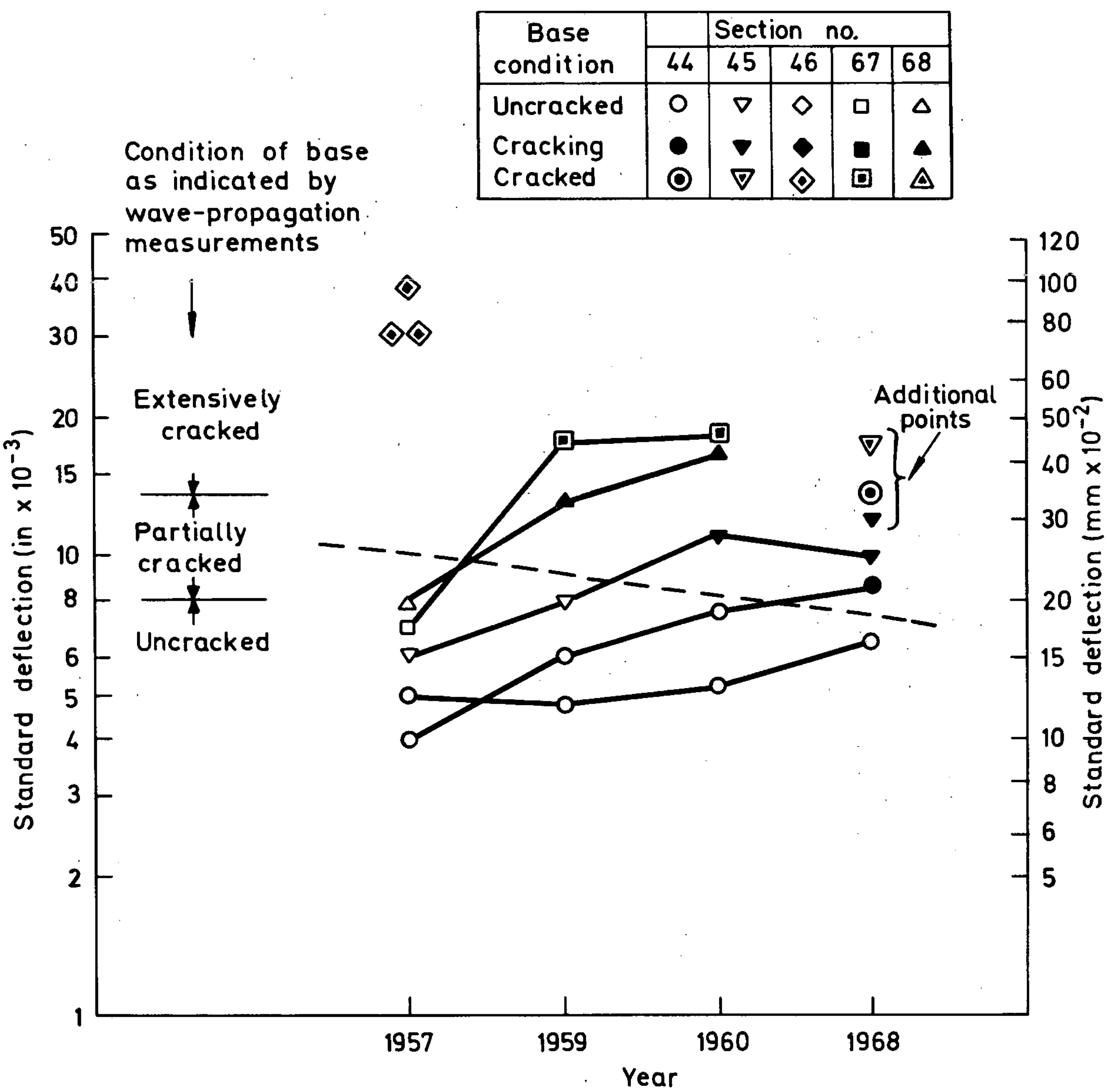

Fig. 25. RELATION BETWEEN DEFLECTION AND THE CONDITION OF THE LEAN CONCRETE BASES

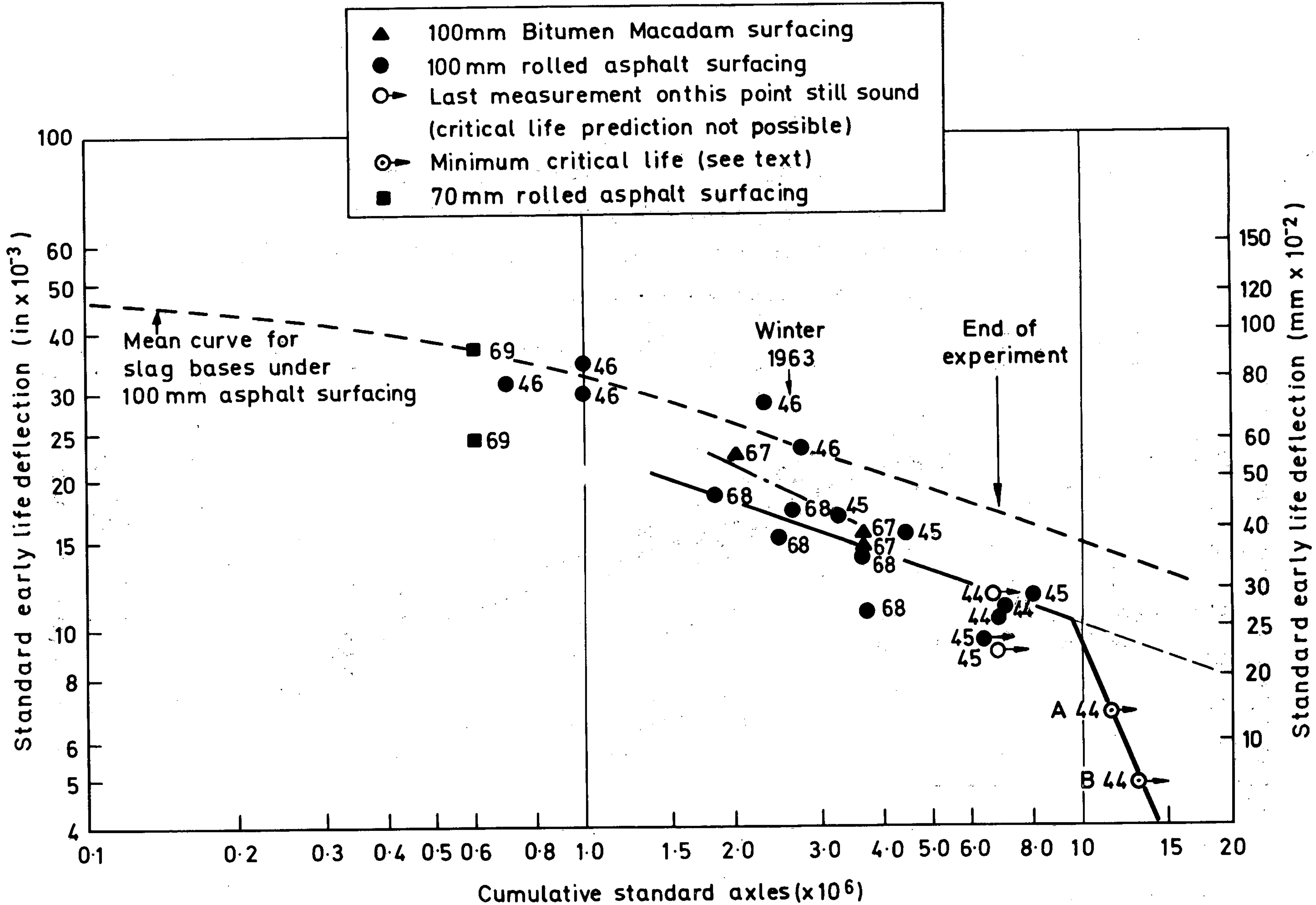

Fig. 26. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR LEAN CONCRETE BASES AT ALCONBURY HILL

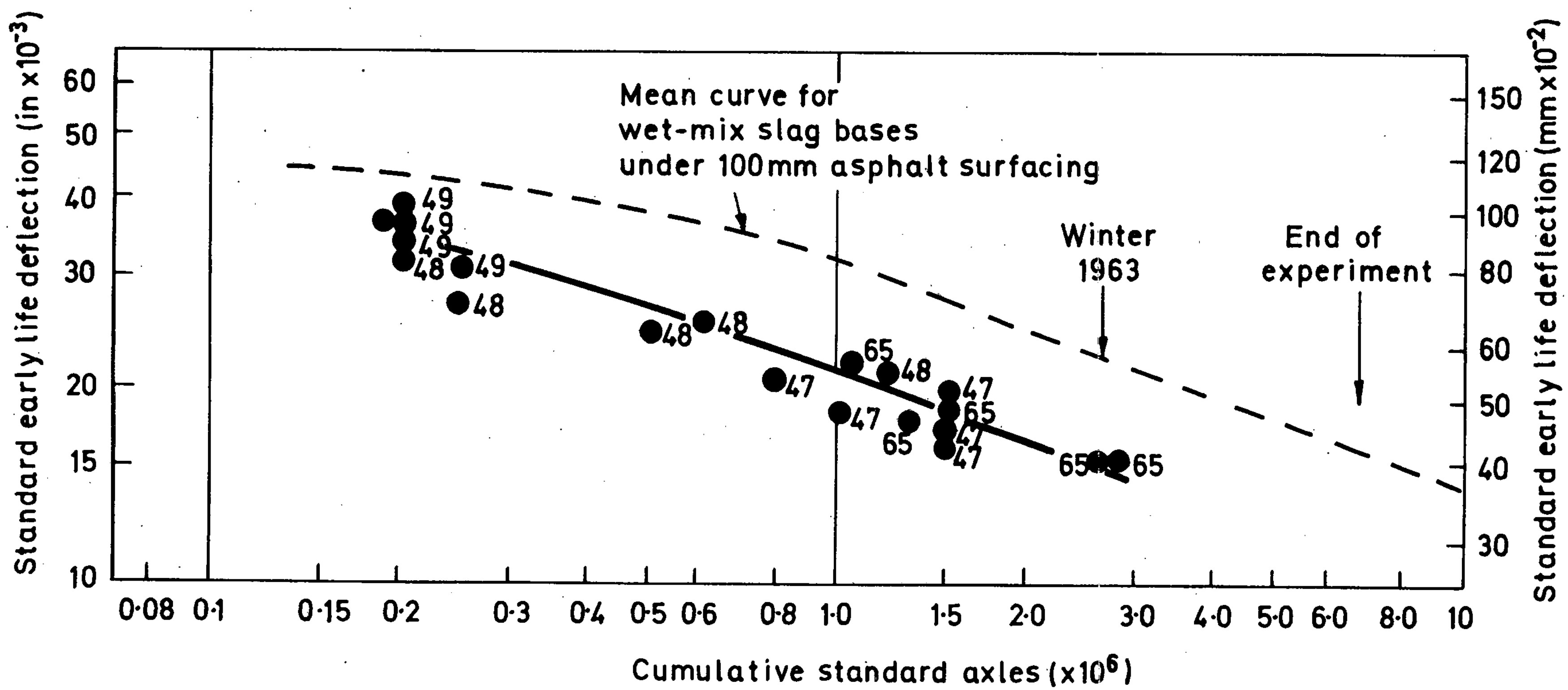

Fig. 27. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR SOIL CEMENT BASES UNDER 100mm ROLLED ASPHALT SURFACING AT ALCONBURY HILL

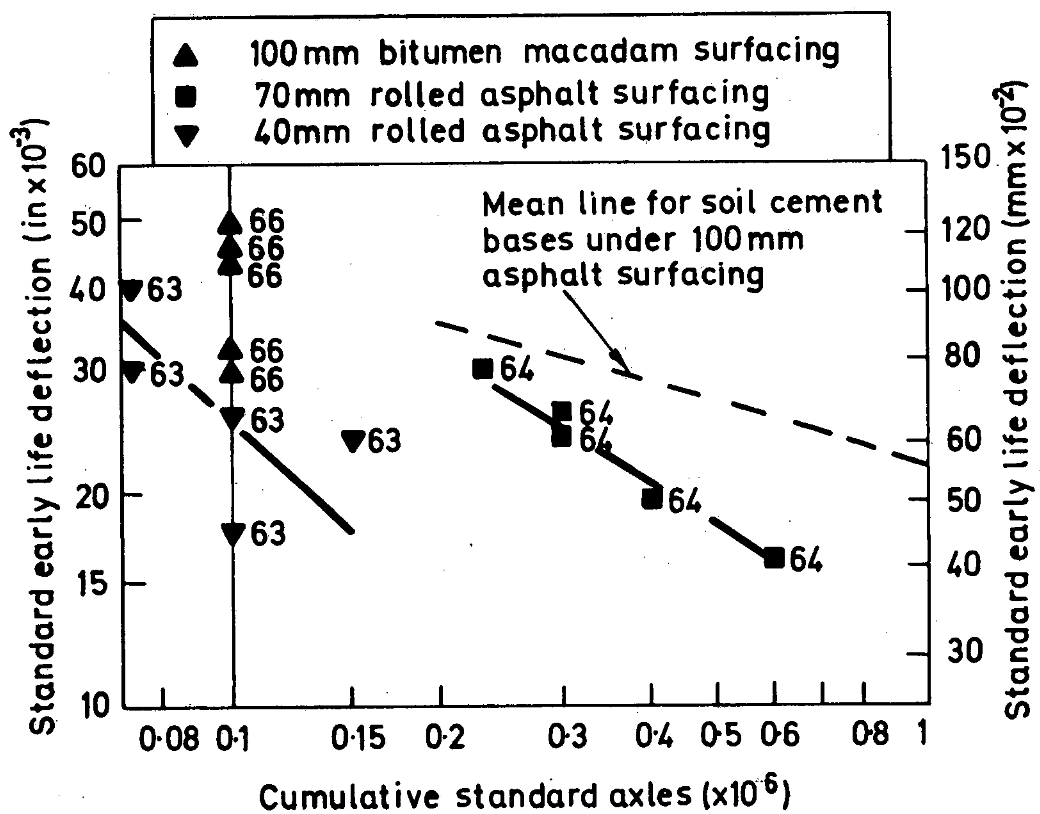

Fig. 28. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR SOIL CEMENT BASES AT ALCONBURY HILL

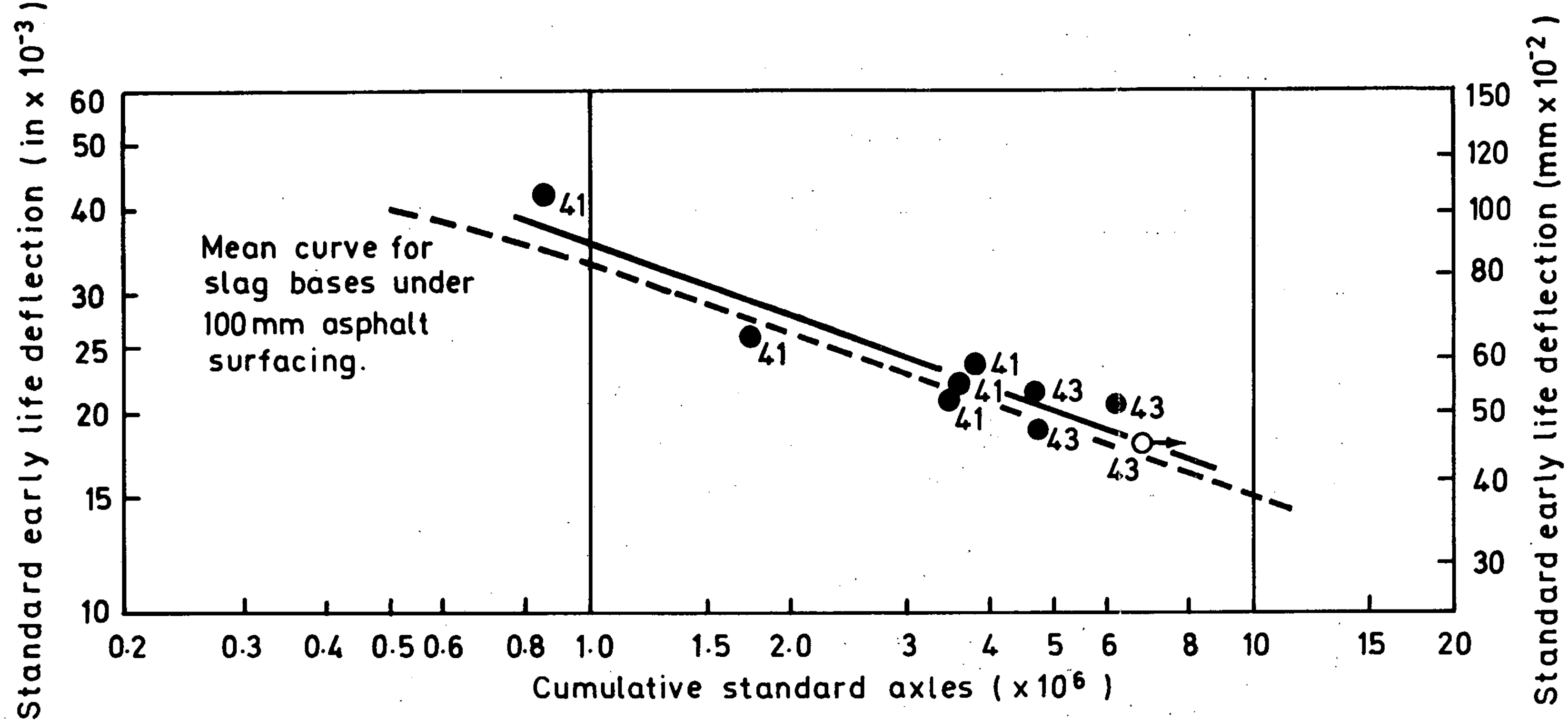

Fig. 29 RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR OPEN TEXTURED TARMACADAM BASES UNDER 100 mm. ROLLED ASPHALT SURFACINGS AT ALCONBURY HILL.

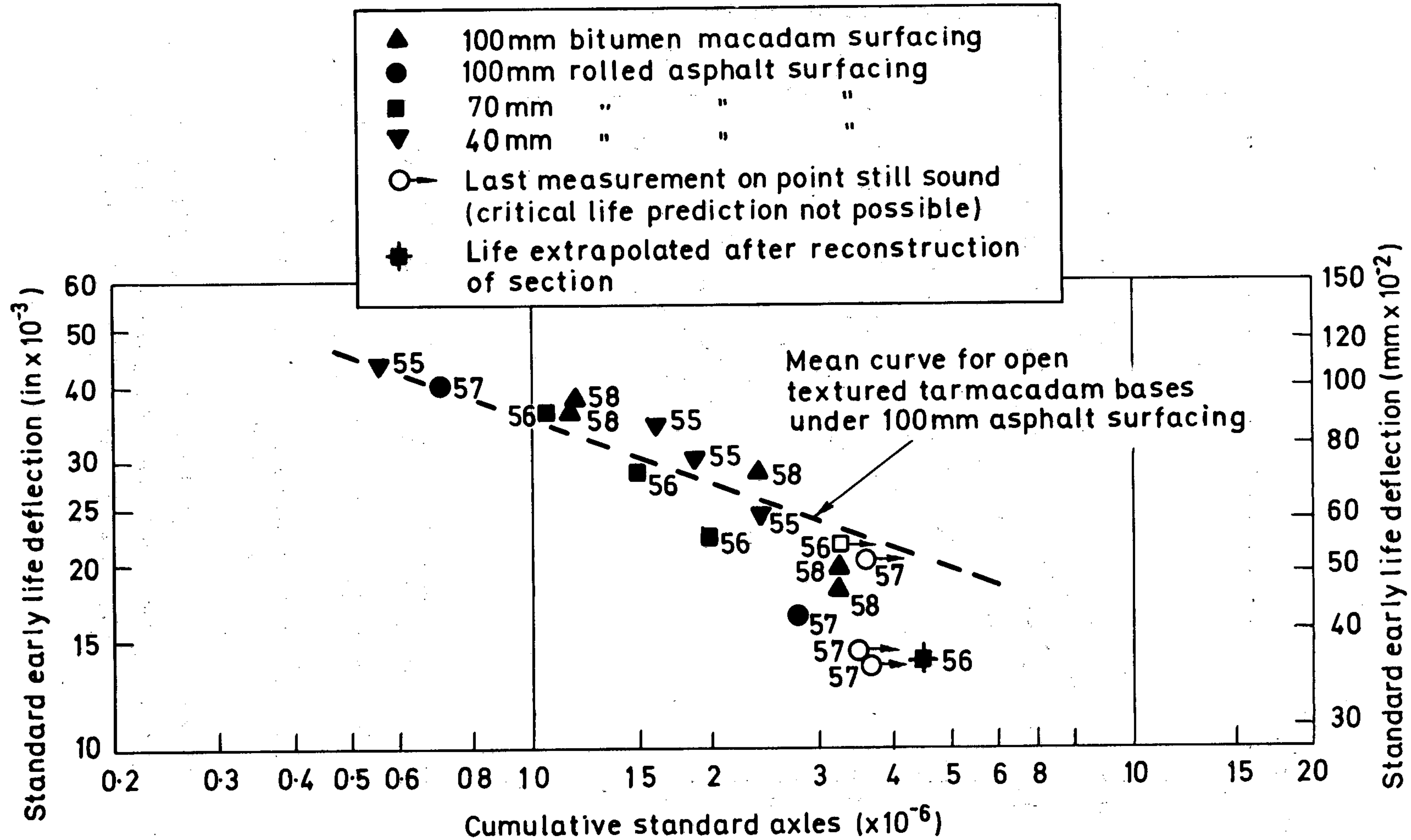

Fig. 30. RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR MEDIUM TEXTURED TARMACADAM BASES UNDER VARIOUS SURFACINGS AT ALCONBURY HILL

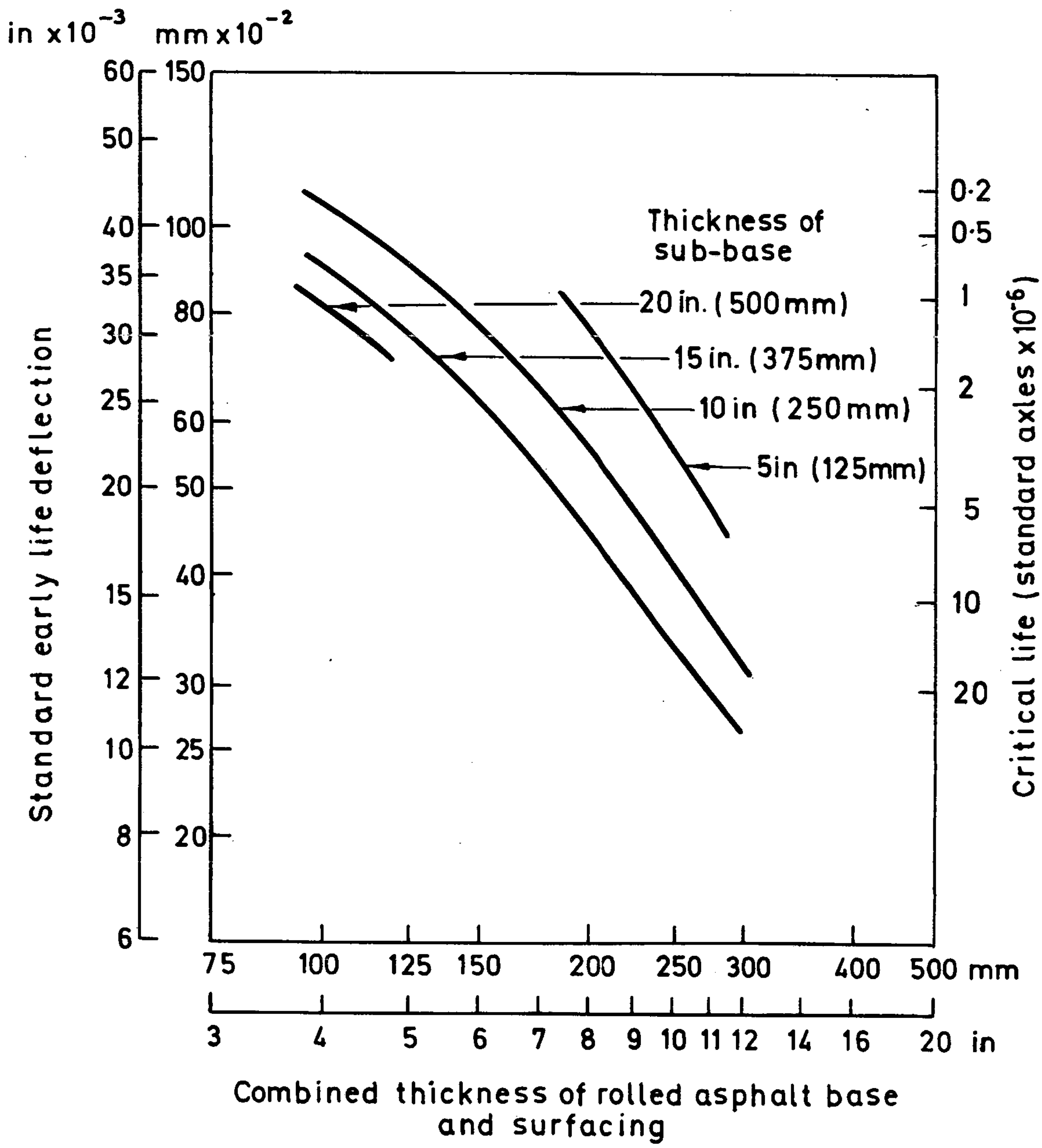

Fig.31 RELATION BETWEEN DEFLECTION, CRITICAL LIFE AND THICKNESS FOR PAVEMENTS WITH ROLLED ASPHALT BASES AT ALCONBURY HILL

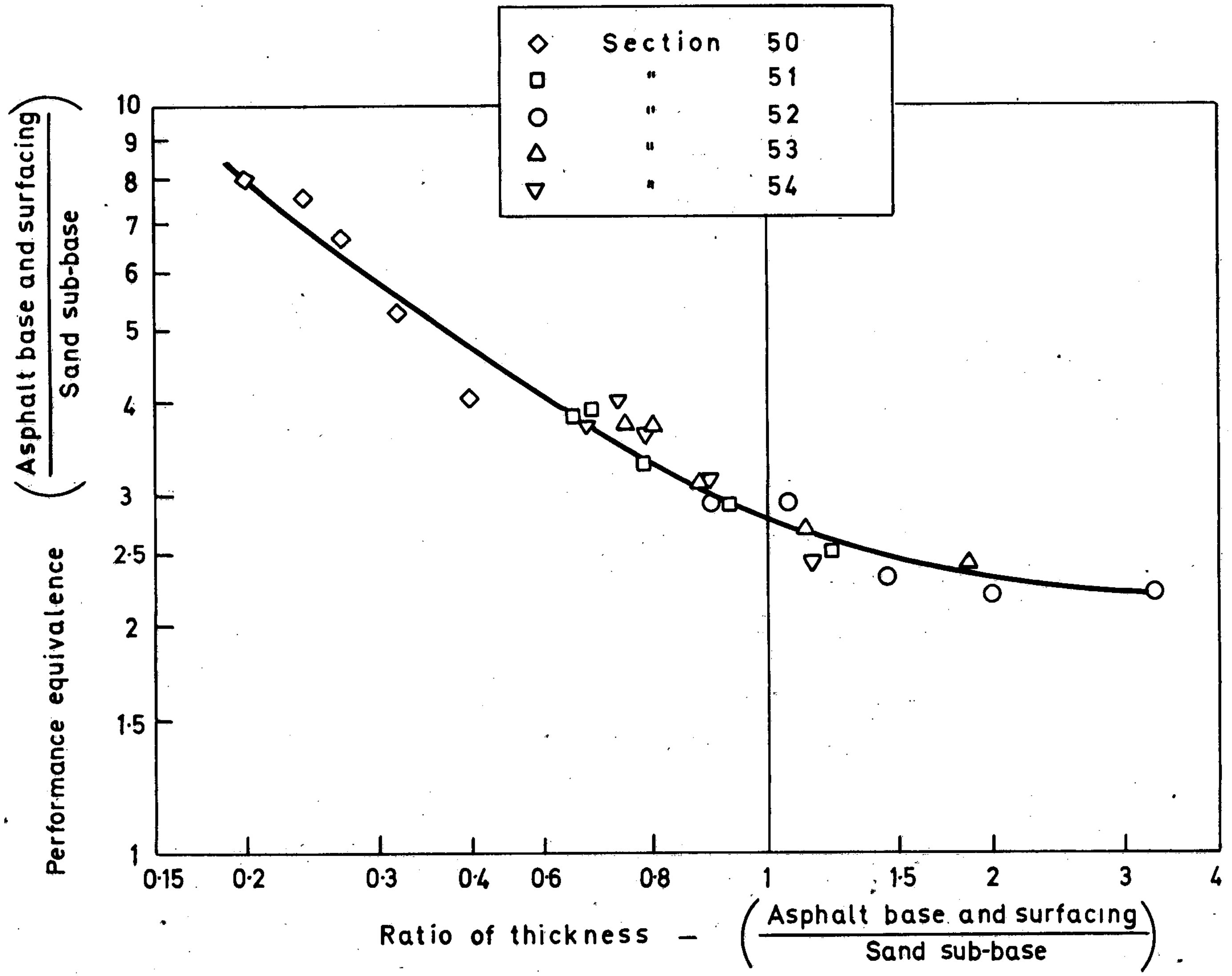

Fig.32 RELATION BETWEEN PERFORMANCE EQUIVALENCE AND THICKNESS RATIO FOR ASPHALT PAVEMENT AND SAND SUB-BASE AT ALCONBURY HILL

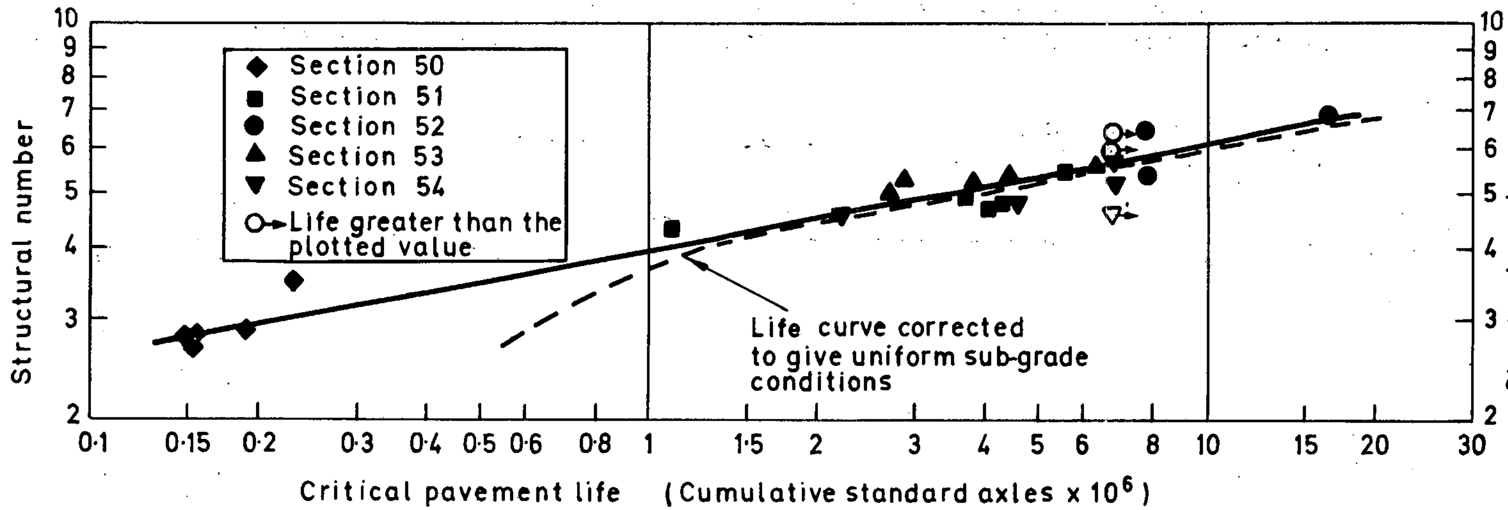

Fig.33 RELATION BETWEEN STRUCTURAL NUMBER AND CRITICAL PAVEMENT LIFE FOR SECTIONS WITH ASPHALT BASES AT ALCONBURY HILL

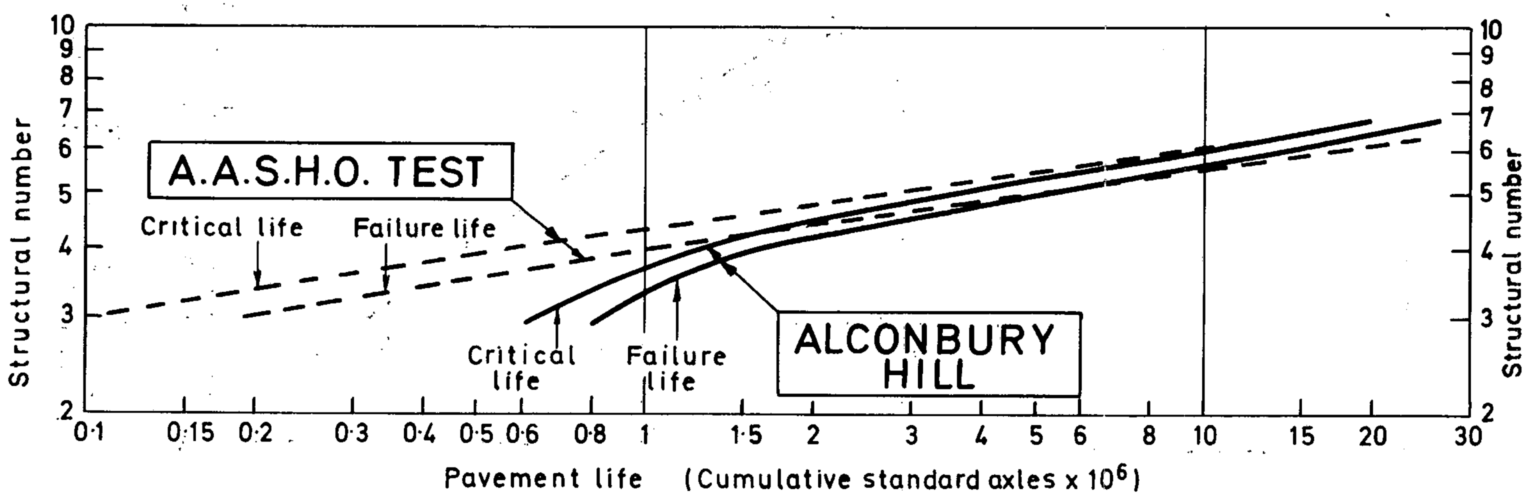

Fig.34 COMPARISON BETWEEN PAVEMENT LIVES PREDICTED BY THE DEFLECTION METHOD AND THE A.A.S.H.O. TEST

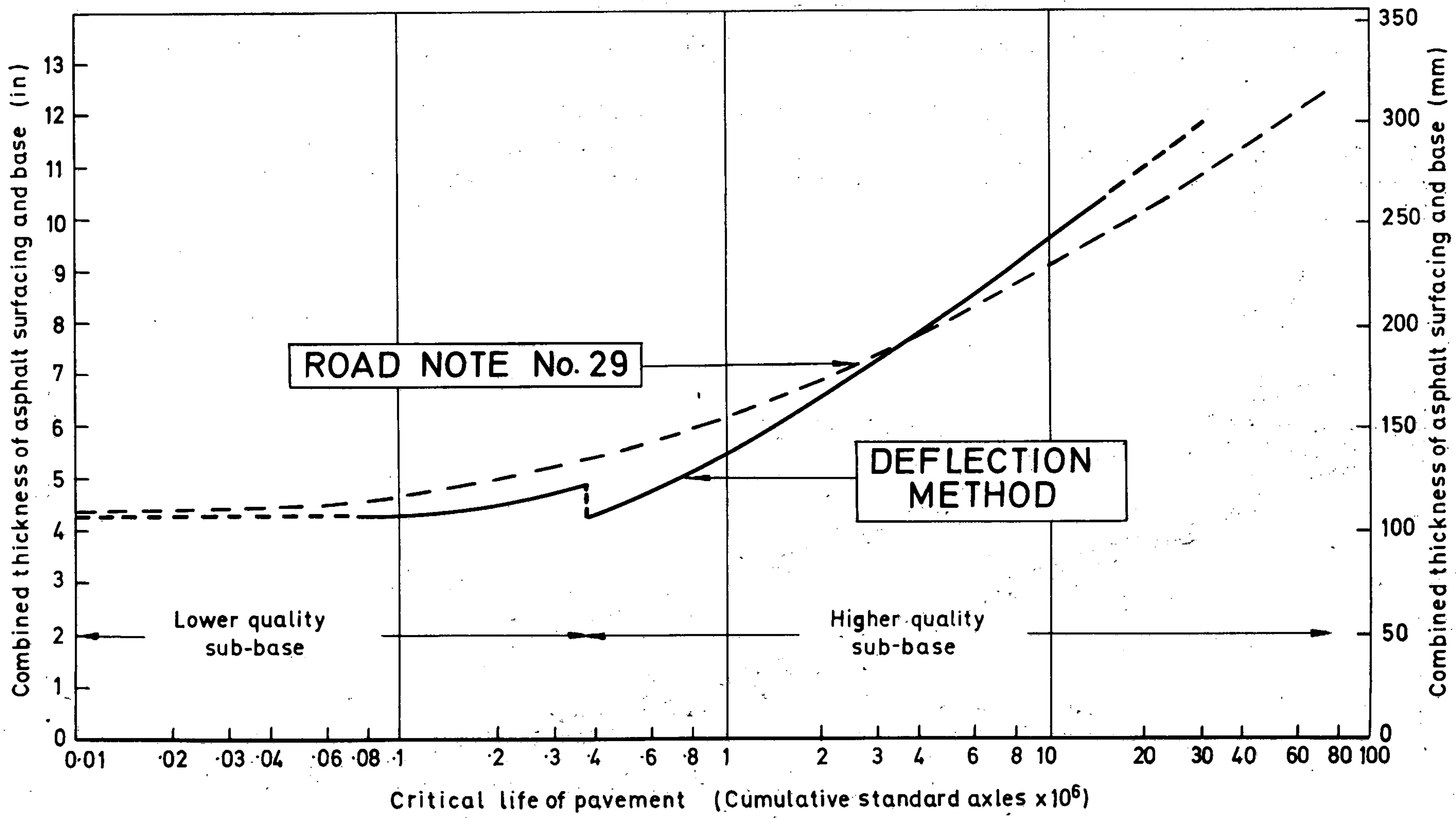

Fig.35 COMPARISON OF THICKNESSES OF ASPHALT SURFACING AND BASE PREDICTED BY THE DEFLECTION METHOD AND ROAD NOTE No.29

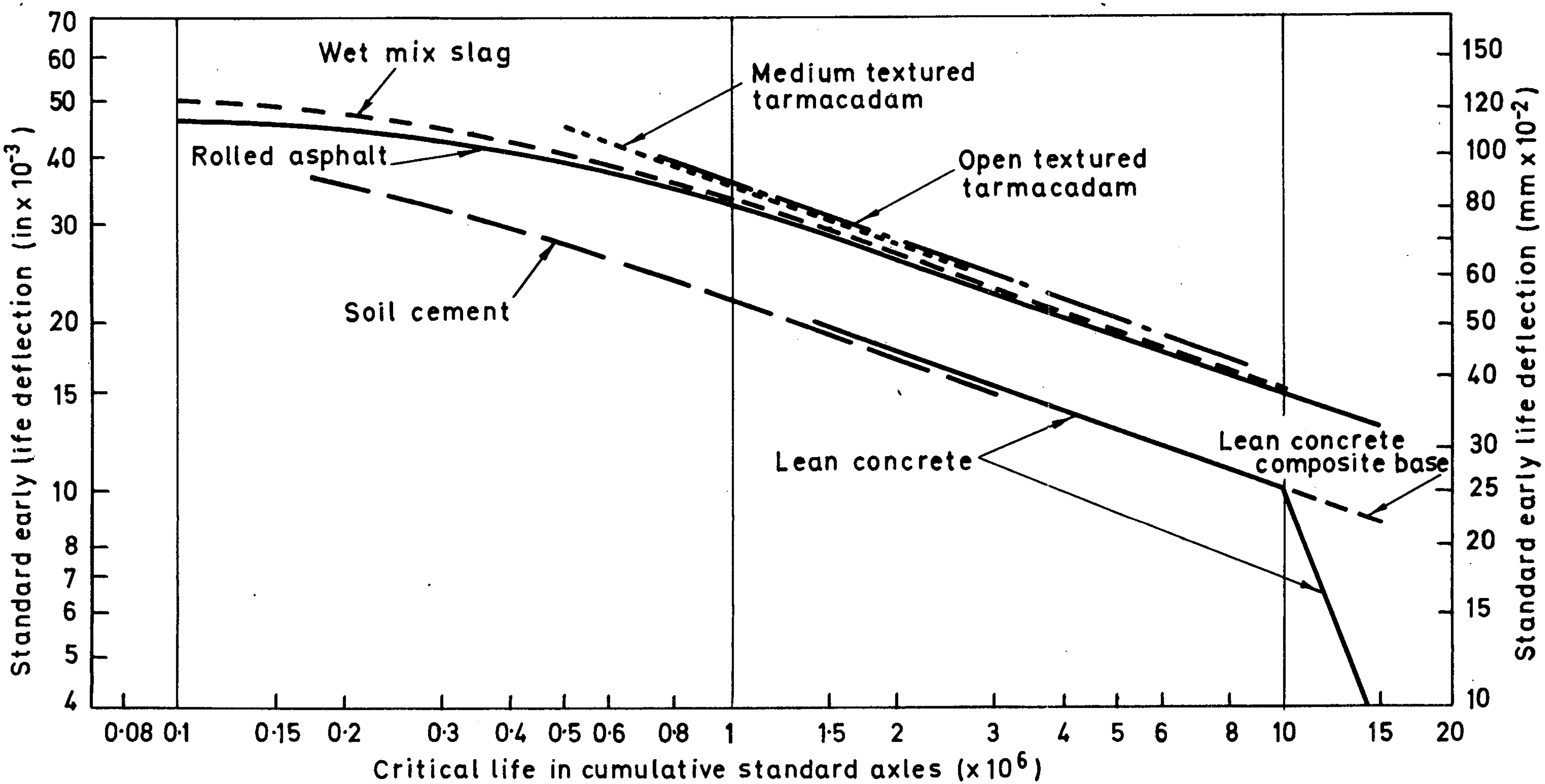

Fig.36 RELATION BETWEEN DEFLECTION AND CRITICAL LIFE FOR DIFFERENT TYPES OF BASE UNDER 100mm (4in) ROLLED ASPHALT SURFACING

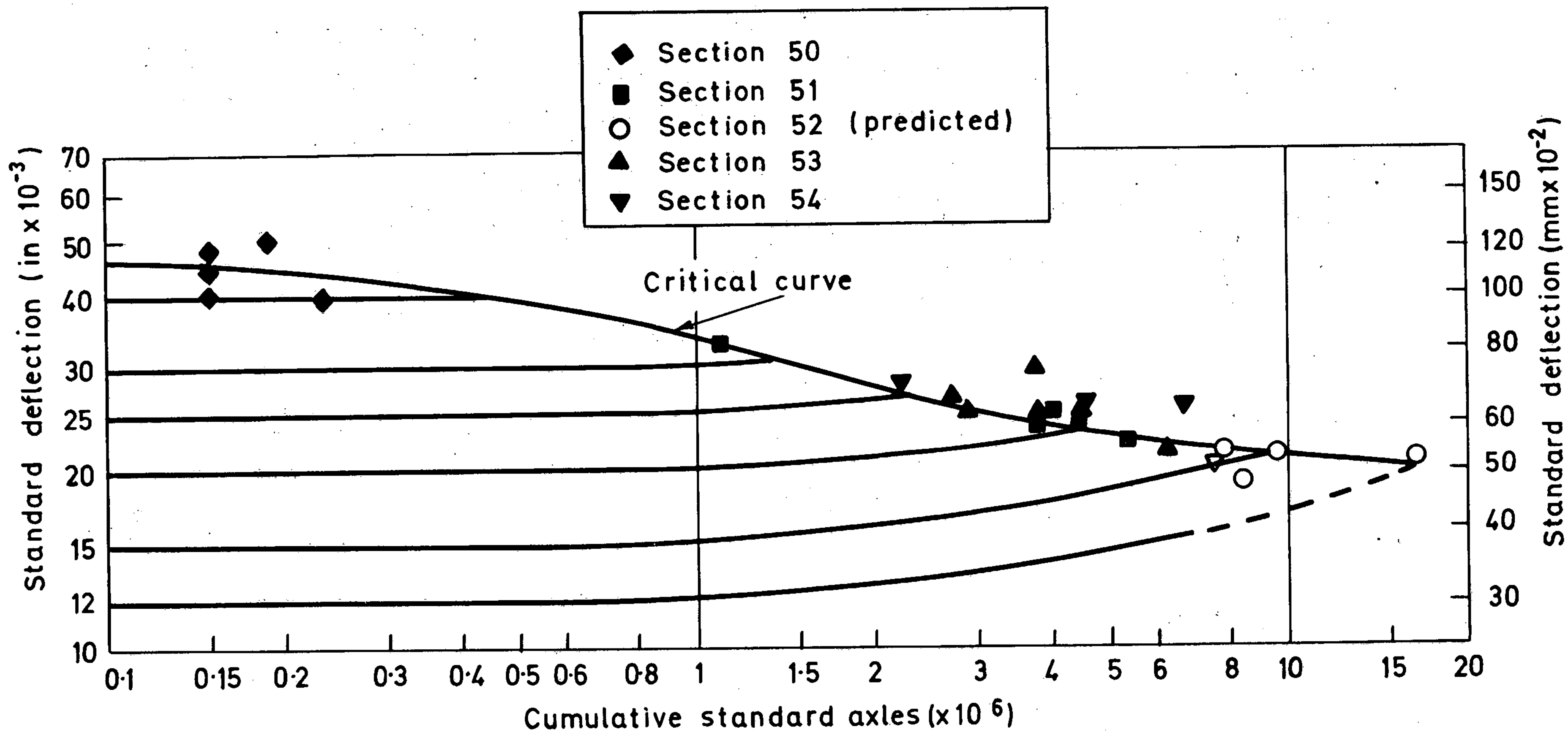

Fig. 37 THE RELATION BETWEEN DEFLECTION AND TRAFFIC UP TO THE ONSET OF CRITICAL PAVEMENT CONDITIONS IN SECTIONS WITH ROLLED ASPHALT BASES AT ALCONBURY HILL

PLATE 1

Neg No B 607/67

The Deflection Beam in position at the start of a test

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ABSTRACT

Deflection criteria for flexible pavements: N W LISTER, B.Sc., C.Eng., M.I.C.E., M.Inst.H.E.: Department of the Environment, TRRL Report LR 375: Crowthorne, 1972 (Transport and Road Research Laboratory). The long-term performance of a flexible pavement is related to the magnitude of the transient deflections which occur under traffic. Measurements made over the past 15 years on experimental pavements have shown that it is possible to relate the deflection measured under a 'standard' wheel-load to pavement life in terms of the cumulative commercial traffic carried.

Deflection criteria curves of this type have been developed for pavements with crushed stone, rolled asphalt, tarmacadam and cemented bases, under rolled asphalt surfacings. The data reported in this Paper have been collected from the Laboratory's full-scale pavement design experiments, and particularly from the Alconbury Hill experiment constructed in 1957. The deflection measurements have been made with the Deflection Beam.

The deflection criterion curves for pavements with crushed stone, rolled asphalt and coated macadam bases are identical within the limits of the experimental accuracy, but acceptable deflection levels are much lower on pavements with cemented bases.

The Paper discusses the way in which deflection criterion curves can be used to estimate future performance and future maintenance requirements for flexible pavements.

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