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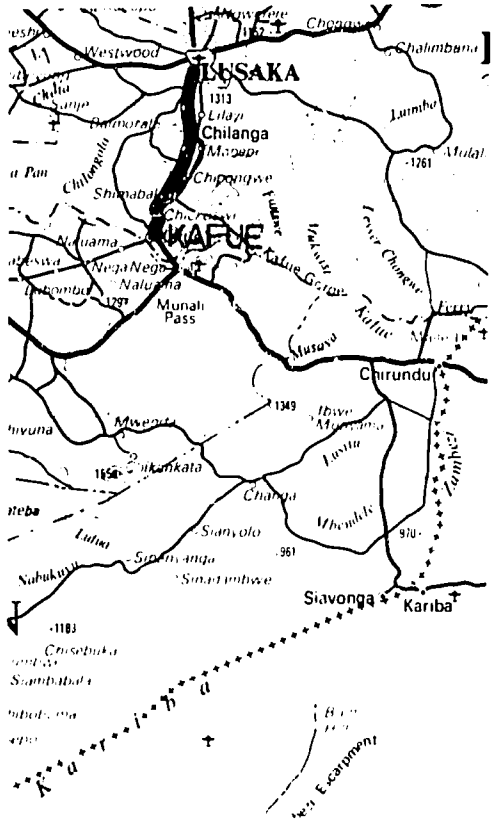

Kafue - Lusaka Road Project
Feasibility Study/Cost Analysis
Final Report

John Burrow & Partners Zambia
in Association with
John Burrow & Partners Overseas

AUGUST 1989

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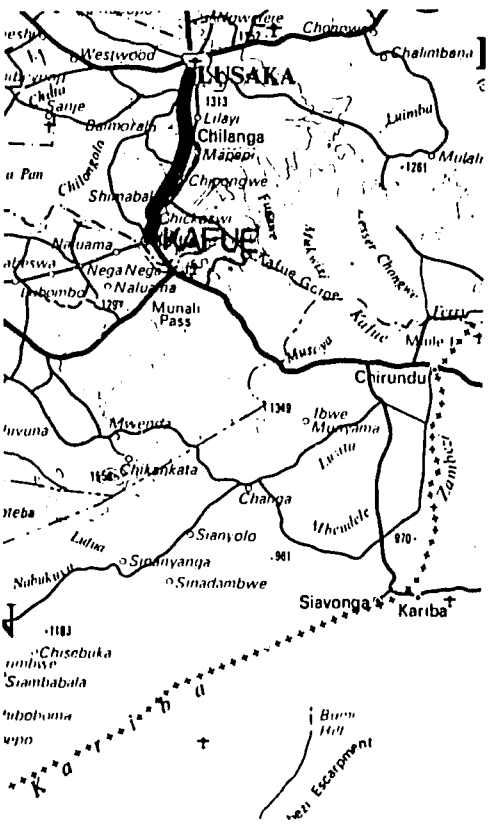

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DEFINITIONS

1 Sub Grade - The term given to the ground below the pavement layers. It is sometimes referred to as the Road Bed.

2 Base, Sub-base - The term given to the pavement layers. The layers are numbered consecutively the uppermost layer being referred to as Base 1. Alternately the uppermost layer is referred to as base and the layer below is referred to as sub-base.

3 Falling Weight Deflectometer - A tool used to Measure the deflection characteristics of the Road under a Load.

4 Dynamic Cone Penetrometer (DCP) - A standard Rod with a Cone tip driven into the pavement layers and the road bed. The resistance as measured by the number of blows is used to derive the strength of the various layers.

5 Benkelman Beam - A tool used to measure the deflection characteristics of the Road when subjected to a load.

6 Compactive Effort - Defined as the density of the material after the application of the compactive effort. This is specified as a percentage Mod. AASHTO.

7 In situ Moisture Content (mc) - Field moisture content at time of sampling. This will vary from season to season.

8 Plastic Limit (PL) - Moisture content at which a clay soil passes from the solid to the plastic state.

9 Liquid Limit (LL) - Moisture content at which a clay soil passes from the plastic to the liquid state.

10 Plasticity Index (PI) - Magnitude of range of moisture contents over which the soil is in the plastic state. (PI = LL - PL).

11 Linear Shrinkage (LS) - Shrinkage (expressed as a percentage of the original length) of a specimen after drying out from a state where the moisture content equals the Liquid Limit.

12 Grading Modulus (GM) - One hundredth part of the sum of the percentages retained on the 2.00mm, 0.425mm and 0.075mm sieves.

13 AASHTO Classification - System of soil classification which divides soils into seven groups designated A - 1 to A - 7, depending on physical properties (particle size distribution, Liquid Limit and Plasticity Index). The lower the classification number, the better the material is from a highway stability point of view.

14 Design CBR (CBR) - The California Bearing Ratio Value assigned to a subgrade and used to determine the required pavement thickness. It is the CBR of a sample compacted at Standard AASHTO optimum moisture content and soaked for minimum of four days.

15 Design Life - A period of years, e.g 10, 15, or 20, selected by the designer, for which he expects a pavement to remain serviceable before requiring strengthening or reconstruction.

16 Design Traffic - The estimated number of equivalent 8200 kN single axles which will be carried by the lane during the design life of the pavement.

17 Equivalent 8200 kN Axle - The 'standard axle load' to which all heavy vehicle axles are equated in order to estimate design traffic.

18 Heavy Vehicle - This is defined as any vehicle with a net mass exceeding 2300 Kg. Such vehicle are required by law to carry reflective rear chevron markings and have been analysed to determine axle equivalences for design purposes.

1. INTRODUCTION

1. INTRODUCTION

1.1 As part of its Southern Africa Regional Programme the United States Agency for International Development (USAID) is planning to provide funds in the financial year 1990 commencing on October 1, 1989, to rehabilitate the Kafue - Lusaka road from the Kafue River Bridge to the Makeni Road turnoff (approximately 3.9 km from the Kafue traffic circle in the centre of Lusaka City). This section of the road totalling 49.4 kilometres adjoins the Kafue Chirundu Road which was recently rehabilitated using funds provided by the USAID from the Regional Transport and Storage Development Project. The two sections, together, form the Zambian section of the Southern Road Transport route which is an important transport route for Land Locked Zambia and for the Southern Africa Region.

1.2 As the first stage of the planning process, in the implementation of rehabilitation works, John Burrow and Partners Zambia in association with John Burrow & Partners Overseas were commissioned on April 22, 1989 to undertake a feasibility study/cost analysis. The study aims at preparing a preliminary design of the rehabilitation works and an estimate of the cost of construction. The output from this study will be used by USAID for budgeting of project finance and preliminary engineering designs will become the basis for detailed engineering design and preparation of contract documentation.

1.3 The original Terms of Reference and the Consultants, proposal specified the start point at Makeni at the Lusaka end of the Study Road. However in the early days of the study the Consultants were requested to consider the start point of the study as the traffic circle at the intersection of Kafue Road/Ben Bella Drive/Cairo Road Independence Avenue. However the cost estimates have been prepared in such a way that the cost of rehabilitating this first section of the Road, traffic circle to Makeni, can easily be separated.

1.4 In accordance with the terms of reference the data utilised, study methodology, analysis and recommendations of the study are all presented in a final report in the form of this document. The plan and longitudinal topographical survey drawings and the Bills of Quantities which were used to establish the preliminary cost estimate of the project and cross section data for the road corridor are all presented in separately bound documents.

2. THE STUDY ROAD

2.1 Location

2.2 Topography

2.3 Geology

2.4 Rainfall and Climate

2.5 Vegetation and Land Use

2. THE STUDY ROAD

2.1 Location

The road under study is the T2 Trunk Road between Lusaka and Kafue Bridge, having a total length of 53.3 km. The road links the capital city Lusaka with an important industrial town, Kafue, and continues southwards to the bridge crossing the Kafue River (Figure 2.1). The road is important in a regional context, linking East and Central Africa with the region south of the Zambezi River.

The start chainage (km 0.0) of the road is on the Kafue Road side of the turning circle at the junction of Kafue Road, Bon Bella Road, Cairo Road and Independence Avenue. The end chainage (km 53.3) is at the north end of the Kafue River Bridge. Between km 0.0 and km 3.9 the road is a four lane carriageway (two lanes in each direction) with no median strip. At km 3.9 the road passes through the suburb of Makeni and then continues as a single carriageway. Between km 14 - 15 the road passes through the town of Chilanga which is at the boundary of the city of Greater Lusaka. The town of Kafue is located between km 43 - 44 and the river crossing (km 53.3) is a further 10 km southbound.

2.2 Topography

The dual carriageway section (km 0.0 to km 3.9) is flat and low-lying with evidence of standing water in ditches and on the surrounding land. After km 3.9 the vertical alignment remains flat and low-lying until km 6.0 when the road level rises above the surrounding ground.

Between km 10.0 and km 11.0 the road level dips significantly in the vicinity of a stream crossing at km 10.5. The alignment then rises slowly and falls again in the vicinity of km 12.5, where there is another stream crossing, with evidence of standing water. The road rises significantly between km 12.5 and km 13.0 and the alignment then remains fairly flat as far as Chilanga (km 14.3).

Between km 16.0 and km 23.0 the vertical alignment is rolling with dips in the alignment at km 17.0, 18.3, and km 21.8. From km 23.0 to km 30.0 the alignment is predominantly flat and low-lying. Within this

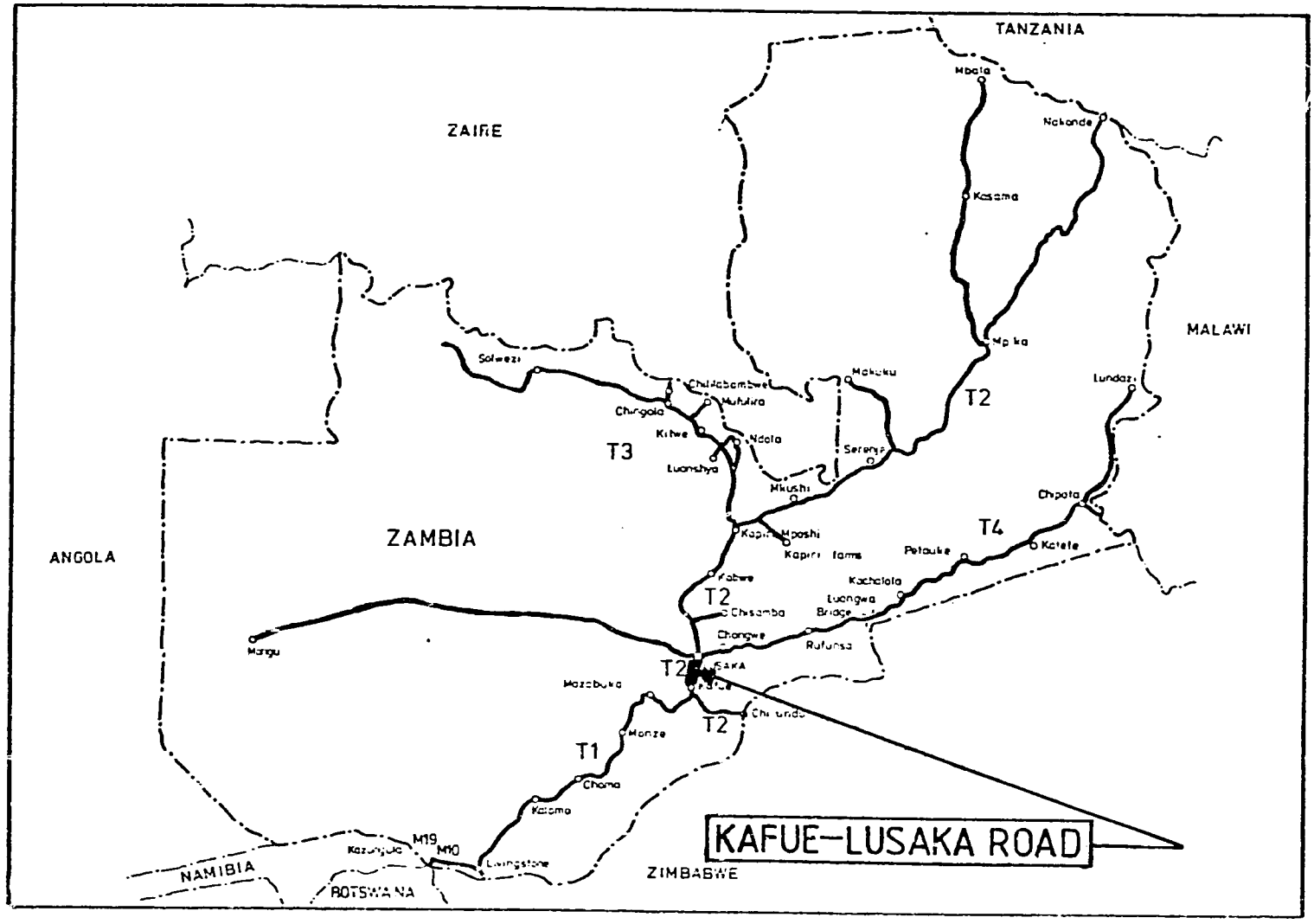

Figure 2.1 Location Plan

length there are effectively three long, straight low-lying stretches of road with hills cresting at km 25.0 and km 27.2.

From km 30.0 the horizontal alignment becomes more varied as the road passes around several hills, with a short escarpment section between km 37.0 and km 38.0.

Between Kafue(km 43.0) and Kafue Bridge (km 53.3) the vertical alignment remains predominantly flat, with a short rolling section near km 50.0.

2.3 Geology

Disregarding superficial and swamp deposits, about two-thirds of Zambia is underlain by Precambrian rocks. Extensive tracts of Karroo rocks occupy the Luangwa, Luano-Lukasashi and Zambezi rift valleys, and the smaller Rufunsa depresssion. Though it is now covered by superficial deposits, the lower Kafue valley seems to be underlain by Karroo sediments and probably represents an ancient rift valley which, unlike the other troughs, has not been so strongly rejuvenated by recent faulting.

The base of the Katanga System is the major and best-defined line of demarcation in the Precambrian. The pre-Katanga rocks comprise granites, granite-gneisses, metaquartzites, metavolcanics and altered basic rocks, with a dominant north-easterly trend. Intense deformation, metamorphism and granitisation have often destroyed stratigraphic continuity and much of the evidence of inconformities and conformable superposition of lithological units may have little or no stratigraphic significance. The main evidence for subdivision is provided by marked differences in the grade of metamorphism of rocks of comparable lithology, and the interplay of minor structures.

The area about Lusaka is underlain by ancient Late Pre-Cambrian and Lower Palaeozoic rocks of the Katanga System, which in turn overlie the Pre-Cambrian Basement Complex. They consist mainly of metamorphosed gneisses, schists, quartzites, limestones and shales. Recent alluvium occurs on the Kafue Flats and along the flood plains of minor rivers.

In general, soil development depends mainly on differences in parent material, the material on which the soil has developed climatological conditions such as temperature, precipitation, etc., and differences in topography. In the area about Lusaka there is a close relationship between the development of the soil and its underlying rock or parent material, particularly with regard to soil texture, colour, structure and other morphological features.

Around the city of Lusaka dark brown to black deep clayey soils occur, often too wet for cultivation, and related to positions in dambo depressions such as the Kalingalinga Dambo. As no general geological mapping has been carried our in the Lusaka area the approach adopted is to give brief descriptions of representative soil profiles in different areas along the road corridor.

Area 1, York Farm at km 8

York Farm is situated on the almost flat surface of the plateau about 8 km south-west of Lusaka and adjacent to the road at 1280m transitional area between the Lusaka dolomite to the north and the Cheta formation further south, characterised by muscovite-biotite psammites and quartzites mixed with metamorphosed limestone. The well-drained clayey soils (60-70%) of the Makeni Series are amongst the best soils in Zambia. Developed over basic rocks that include dolomite, limestone and gabbro (which forms a small inlier to the south of York Farm), the chemical and mineralogical data support field observations of a soil system with a high potential fertility (pH 5.5 to 7, base-saturation 70-90%). Clay mineralogical investigations demonstrate a suite with halloysite, illite, kaolinite, smectite and vermiculite . Other minerals detected are biotite, feldspar, quartz and talc. Kaolinite seems more pronounced in the upper part of the profile, but somewhat disorganised. This may reflect transformations dur to insitu development, a notion which is supported by the grain mineralogy.

Due to the very low bearing capacity of the soil when wet, farming operations cannot be carried out from mid-December to February (the wet season).

Area 2, Mount Makulu at km 15

The Mount Makulu Research Station is at Chilanga, 15 km south of Lusaka. It is at the southern limit of the Lusaka plateau where it has become gently undulating, with shallow valleys drained by streams which tend to dry up during the drier months of the year. Immediately to the south is a zone of more broken terrain leading down to the Kafue Flats. The soils are developed on the weathering products of a flaggy muscovite-biotite psammiteand quartzite, interlayered with limestone and dolomite. It is well drained, brown to yellowish red and a deep to very deep clayey soil. It has a medium base saturation and a clayey topsoil. Mineralogical examination of the Makeni Series clay fraction has shown that mixed layers of lllite-montmorillonite are present, together with illite and traaces of quartz and kaolinite.

Area 3, Mungu River Source at km 22

This area is located approximately 22 km south east of Kafue town at 1020m, and is crossed by the Mungu, a tributary of the Kafue River. The main features of the landscape are the quartz-schist hills (Chipongwe formation) encircled by piedmont slopes and extensive areas of alluvial heavy clays with low leaves found along the floodplain of the Mungu.

On the quartz-schist is a very complex pattern of soils with pockets of shallow sandy soils interspered with outcrops of gravel and rock which culminate in a low ridge and isolated hills. The piedmont slope (1-2%) gives way to a seepage zone at the margin of the valley, where sheets of laterine outcrop. The soils are well drained, brown to yellowish brown, deep to very deep loamy sand to sandy soils. They have a low to medium base saturation and are suited to a range of crops. There is a termitaria zone, and extensive alluvial and seasonally wet and poorly drained clayey soils extend as far as the levees on the banks of the Mungu River.

Area 4, Kafue Flats at km 50

The Kafue Flats are located south-west of Lusaka at an altitude of below 1 000 m above sea level. The soils are developed in recent alluvial material and are very deep and clayey, with clay percentages of more than 50

percent. They are dark grey in colour and have an acidity beetween pH 605 and 8. The alkaline reaction of the subsoil is caused by the presence of lime nodules. The surface displays a gilgai microrelief, characterised by small mounds (approximately 20 cm in height and polygonal pattern or cracks, extending at least 50 cm in depth and having a width at the surface of 4 cm or more. The formation of these features is due to alternate wetting in the rainy season and drying of the soils in the dry season. Expanding clay minerals take up water during the rainy period, expending in volume, but lose it during the dry period so that shrinkage ensues.

The heavy texture and type of clay results in a compact soil with poor internal drainage.

2.4 Rainfall and Climate

The climate of Zambia is typified by one defined wet and one dry season each year. The rain falls between November and March, these months accounting for about 93% of average annual rainfall. Temperatures are generally not excessive, the hottest months coming immediately before the annual rains.

Climatological details at Mt Makulu Research Station (Chilanga) are summarised below:-

Mean Monthly Temperature	: 15.7 - 24.6 deg C
Absolute Max Temperature	: 31.1 deg C
Absolute Min Temperature	: -1.6 deg C
Mean Relative Humidity	: 37 - 83 %
Mean Monthly Evaporation	: 111 - 284 mm
Mean Monthly Rainfall	: 0 - 219 mm
Average Annual Rainfall	: 836 mm

A typical value for the Thornthwaite Moisture Index (Im) can be estimated from the following climate classification

Moisture Index (Im)	Climate
100 and above	Perhumid
20 to 99.9	Humid
0 to 19.9	Moist subhumid
-19.9 to 0	Dry subhumid
-39.9 to 20	Semiarid
-100 to 40	Arid

Based on moist subhumid climate, the moisture under (1 m) is estimated at 10. Meteorological conditions do not change appreciably between Lusaka and Kafue, despite a change in altitude of over 200m. For study purposes, at this preliminary stage, climate conditions at Mt Makulu may be considered typical for the entire road length.

Comparison of monthly rainfall and evaporation data shows water surpluses during December, January and February. Deflection measurements for pavement evaluation should, therefore, be made in these months, or shortly thereafter.

Ambient temperatures are best quantified for pavement performance purposes by the weighted mean annual average temperature, w-MAAT (Ref 1). This value has been calculated and is shown below with comparable values for other areas from which pavement evaluation methods are sometimes borrowed:-

Location	w-MAAT
Mt. Makulu	22 deg C.
London (England)	12 deg C
Pretoria (S. Africa)	19 deg C
New Delhi (India)	28 deg C
Melborne (Australia)	15 deg C

Clearly countries like Australia and India have a considerable diversity of climates within their borders but the similarity of temperature regimes (and hence the performance of bituminous materials) in Zambia and South Africa is quite clear.

2.5 Vegetation and Land Use

The Lusaka - Kafue Road is situated towards the southern margin of the Central Plateau which, in common with all plateau areas of Zambia, is predominantly clothed in a form of dry deciduous termed 'miombo'. However, to the south of the city, the land descends through an escarpment zone to the valleys of the Kafue and Zambezi rivers. Tributary valleys dissect the scarp and penetrate the plateau to produce a region of varied relief and vegetation. The variation in vegetation is further accentuated by local edaphic controls and the influence of fire, to produce a complex mosaic of natural and semi-natural communities.

Miombo woodland covers not only areas of plateau around Lusaka, but also a large part of the dissected hill and escarpment country to the south and south-east of the city especially between Chilanga and Kafue. It is a two storeyed woodland with an open canopy of semi-evergreen trees. The canopy stands 15 - 21m above ground level and provides shade to between 60 and 80 per cent of the ground area. The underwood layer is most commonly of grass and herbs, and/or suffratices standing 0.6 - 1.3m in height, but in some locations there is an underwood of dense thicker 1.3-3m high.

Alongside perennial streams such as at chainage 33 conditions remain moist and more humid throughout the year, leading to the growth of evergreen riparian woodlands. These are dominated by trees with a dense, thicket-like shrub layer and many tangled climbers. Only a few remnants of these woodlands survive because most have been cleared or severely degraded by attempts to utilise the perennial supplies of water for the growth of crops and grazing.

The dominant trees of miombo woodland are all deciduous but the woodland is frequently described as semi-evergreen. This is because the woodland canopy is bare of leaves for a relatively short period of each year.

Periodic clearance of the woodland for agriculture appears to have little long-term effect provided the area does not suffer repeated burning during the regeneration phase. Immediately after cultivation, the absence of a tree canopy allows a luxuriant ground cover of grasses to develop while the canopy saplings and recolonisation can be restricted to fire-induced grassland or bush for as long as the area is repeatedly burnt. This is evident over most of the road corridor.

Just to the south of Lusaka much of the roadside is covered with a more open savanna woodland, called 'munga' (a vernacular word for thorn), which descends to open grassland on the 'Flats' of the Kafue valley. Munga consists of scattered or grouped trees up to 18m tall which form a discontinuous canopy over an undercover of either bush or grass and herbs, although in some areas undercover is almost entirely absent.

The soils beneath munga vegetation tend to alluvial and more fertile than the plateau and escarpment soils beneath miombo. As a result large areas were cleared for cultivation by Europeans during the colonial period. Much remains under cultivation today, but some has been abandoned or allowed to revert to grazing land.

Large areas of land are maintained as grassland or bush grazing by the use of fire and are never allowed to revert back to woodland. Grassland controlled by edaphic factors also exist within the Lusaka region. Water courses and seepage areas within both munga and miombo woodland tend to support vegetation that is dominated by grasses and other herbs. These valley grassland can be subdivided into three distinct groups. They form a sequence of communities which tend to grade into each other down valley from headwater valleys grassland or 'dambos' through riverine grasslands to flood-plain grasslands.

Dambos are infrequent and poorly developed in the Lusaka region. The presence of the plateau edge and the steep descent into the Kafue and Zambezi rivers to the south creates free draining valleys with steep gradients.

Within the built-up area of Lusaka and as far as Chilanga, little of the original vegetation survives. Most of the area was originally cleared for European agriculture and the subsequent establishment of the capital city created further demands for building materials, firewood and charcoal.

Vacant land within the built-up area and considerable areas of land on the periphery of the road are intensively cultivated for maize and vegetables. Likewise, the well-drained upper margins and termite mounds of most of the dambos have been converted into intensively cultivated vegetable gardens, producing both subsistence crops and market garden produce for sale.

One of the most important mineral products of the Lusaka area is cement, manufactured at Chilanga km 14. Its main raw material, calcite marble is worked at the Shimabala Quarry, 16 km south of Chilanga but new sources have since been discovered. An essential specification for limestone for cement manufacture is

low magnesia content free from contamination with dolomite. Additives which are produced locally are phyllite and hematite.

Marble is also quarried on the south-eastern outskirts of Lusaka for concrete and road aggregates. Here the pressure of the slightly harder dolomite gives better crushing properties than the calcite marble. At km 37 there is a quarry operated by the roads department which supplies road aggregates for surfacing and this rock has been used extensively in the study area

3. SUMMARY OF EXISTING CONDITIONS

3.1 Traffic

3.2 Pavement Condition

3.3 Schedule Significant

3.4 Drainage

3. SUMMARY OF EXISTING CONDITIONS

3.1 Traffic

On the dual carriageway section between the Kafue Road Roundabout and Makeni (km 0 - km 3.9), traffic levels are generally high, with significant delays on all approaches to Kafue Road Roundabout during peak periods. Peak flows generally occur between 7.30 - 8.30 am, 12.30 pm and 4.30 - 5.30 pm.

A significant number of vehicles leave the study road at the Makeni junction (km 3.9), and the next major junction is the Blue Boar junction close to Eureka Corner (km 7.5). There are limited overtaking apportunities between km 7.5 and km 13.5 (near Chilanga), due to the horizontal alignment. In addition, the large number of slow moving heavy vehicles, particularly in the Lusaka bound traffic lane, leads to delays for vehicles wishing to overtake.

Between Chilanga and Kafue, traffic levels are generally light, with only limited delays due to slow moving vehicles on the few hilly sections. Traffic levels reduce further between Kafue Town (km 43.5) and Kafue Bridge (km 53.3).

3.2 Pavement Condition

The existing pavement construction generally comprises 150 mm lateritic gravel sub-base and 150 mm stabilised lateritic gravel roadbase with a double surface dressing layer as wearing surface. On the dual carriageway section, the bituminous surfacing is between 40 - 60 mm thick.

On the dual carriageway section, between the Kafue Road roundabout (km 0.0) and makeni (km 3.9), the pavement condition is generally good, with only limited evidence of surface cracking or deformation. The riding quality on this section is good. However the lane widths are narrow and from a traffic safety point of view this section road should be regraded with great concern.

Between km 3.9 and km 7.0 there is extensive block cracking and some deformation in the wheeltracks. From km 7.0 (Eureka Corner) to km 11.0 the incidence of cracking is not so extensive, but there is an area of settlement at km 7.7 and extensive potholing in the vicinity of km 10.5.

Between km 11.0 and km 12.0 the pavement condition is reasonably good with only limited areas of block cracking and good riding quality. At km 12.5 there is extensive potholing, and as far as km 14.3 (Chilanga Police Station) the road exhibits quite a lot of fine cracking but maintains a reasonable riding quality. At km 14.3 there are extensive potholes, some of which have previously been backfilled with rocks which protrude from the road surface.

From km 14.3 to km 16.8 the pavement condition is fairly good. Between km 16.8 and km 23.0 the topography is rolling with dips in the alignment at km 17.0, km 18.3 and km 21.8. Extensive potholing is present in these dips, and there is extensive patching and block cracking from km 18.5 to km 23.0.

Between km 23.0 and km 30.0 there are three long flat low-lying stretches of road (separated by hills at km 25.0 and km 27.2) which are characterised by extensive areas of patching and block cracking. From km 30.0 to km 33.0 the vertical alignment is gently rolling and the pavement condition still exhibits significant lengths of patching and block cracking.

The pavement condition and riding quality improve significantly between km 33.0 and km 411.0, although there are considerable lengths of block cracking. The block cracking is more extensive between km 41.0 and km 44.0 (in vicinity of Chilanga), with evidence of surface deformation. After km 44.0, the pavement condition is fairly good as far as km 48.0. Thereafter (from km 48.0 to Kafue Bridge at km 53.3) there are significant lengths of block cracking and patching.

3.3 Schedule Significant features

km	Detail
0 + 000	Kafue Road Roundabout Single 4 lane carriageway road with kerbs (initially) and bus lay-bys left and right.
0 + 500	Junction right (Lumumba Road)
3 + 700	Junction left to Compound
3 + 800	Filling Station
3 + 900	Junction right to Makeni
4 + 000	End 4 lane section. Road narrows to 2 lane standard.

4 + 100	Filling Station
5 + 500	Carriageway width 5.9m. Shoulders not defined.
6 + 900	Andrews Motel
7 + 000	Filling Station
7 + 500	Blue Boar junction right
7 + 600	Left hand bend. (Eureka Corner).
9 + 200	Carriageway width 6.10m.
12 + 000	Junction right to Zimco Management College
12 + 500	Left hand bend at bottom of dip.
13 + 800	Side long ground requiring cut off drain and cross culverting.
14 + 300	Bad failed section of pavement not apparently connected with drainage
14 + 500	Junction left to TAP ltd.
14 + 900	Filling Station (BP). Entrance to Chilanga Cement Ltd.
15 + 100	Under bridge providing haul road access to Chilanga Cement factory. Start of Chilanga Township.
15 + 800	Carriageway widens to 6.9m
16 + 400	Carriageway narrows to 6.1m
16 + 900	End of Chilanga Township
17 + 500	Rolling country with good drainage potential. Drainage appears blocked.
18 + 000	Road surface condition deteriorates. Extensive potholing and patching. Carriageway width 6.4m.
18 + 900	Country becomes flat to rolling. Road level generally good. Drainage almost non-existent. Shoulders not defined.
20 + 000	Road surface condition improves.

20 + 800	Cut Section. Carriageway width 6.2m. Insufficient width for full shoulder and side drain.
21 + 600	Failed section at bottom of a dip.
23 + 800	Road surface condition improves.
24 + 500	Junction right to Arudni Farm. Carriageway width 6.5m (locally widened for junction by hanching).
	Road shape and condition good with only isolated potholes.
26 + 500	Low lying area where surface condition deteriorates.
27 + 500	Road surface condition improves.
28 + 700	Powerline crossing.
	Generally low lying section where drainage outfalls can only be obtained over a considerable distance.
30 + 500	Carriageway width 6.2m. Shoulders very wide.
31 + 900	Existing drainage appears blocked. Road level generally acceptable.
35 + 100	Road surface condition improves further. No potholing and only isolated patching evident.
36 + 500	Access to quarry
36 + 900	Country becomes hilly.
39 + 100	Carraigeway width 6.8m, shoulders need widening. Road surface condition very good.
3 + 700	Right hand bend approaching railway crossing.
40 + 000	Railway crossing.
40 + 100	Road appears to have been overlaid with a low stability material that has shoved badly.

41 + 700	Junction right to Kafue Industrial Area.
42 + 600	Start Kafue Township.
43 + 700	Filling Station(Total).
43 + 900	End of Kafue Township.
	Different vegetation and topography indicating change of sub-soil.
	Road width adequate and level acceptable. Surface condition good.
48 + 600	River Motel
53 + 300	Kafue River Bridge.

3.4 Drainage

A detailed survey has been carried out on the road from Lusaka to Kafue. It has been generally observed that the capacity of the culverts are in general adequate and no over topping has been observed in recent years. The main problem with the drainage on the road is lack of preventative maintenance, the side drains have been allowed to be totally overgrown and the mitre drains have been blocked and have not been cleaned out recently in addition some cross drainage culverts are too low and have been allowed to silt up.

These areas, illustrated by more than usual deformation of the road foundation, need to be raised and culverts and outlets cleaned out. The structural adequacy of the drainage culverts is generally good and in many cases these will only have to be extended under the road shoulders and slopes. Those that are in areas of road where the finished road level will be raised in order to prevent further deterioration due to high water tables will however either have to be raised or replaced as in many cases the cover is not adequate and therefore the pipes may not be able to take construction traffic loads.

4.	TRAFFIC SURVEYS
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4.2	Existing Data
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4. TRAFFIC SURVEYS

4.1 Introduction

For the purpose of Geometric designs, pavement design and the evaluation of economic benefits, the volume and type of current and future traffic must be established in terms of cars, light goods vehicles trucks and buses. The volume of traffic that must be catered for during the design life will determine the geometric class to which the road must be improved. The type of traffic and the weight/axle loading of the traffic is required to calculate the total damage likely to be suffered by the road pavement during the design life in order to provide for this damage in the pavement design. In the case of the economic benefit assessment in the context of this study, alternative pavement construction strategies are being considered in terms of whole life cost where in which a very significant component is the cumulative vehicle operating cost which is of course dependent on type and volume of traffic.

The Road under study between Kafue and Lusaka is important in Regional Terms, linking east and Central Africa with the region south of the Zambezi river. It is also nationally important linking the capital city Lusaka with an important industrial town, Kafue. The first 16 Km of the 53.3 Km traverses a growing peri urban zone.

At Km 3.950 a suburb called Makeni exists with a reletively significant Industrial and residential zone. At km 7.500 a location known as Blue Boar turnoff significant farming settlements exist on the western side of the road. At km 14.500 a satellite town called Chilanga now forms part of the city of Greater Lusaka.

In the longer term there is little doubt that the city of Lusaka will expand at least up to Chilanga. This development trend has already been set and the traffic levels on this first stretch of road also reflect this level of development.

4.2 Existing Data

The results of 12 - hour traffic counts, taken over a seven day period at the following four locations, was provided by the Roads Department.

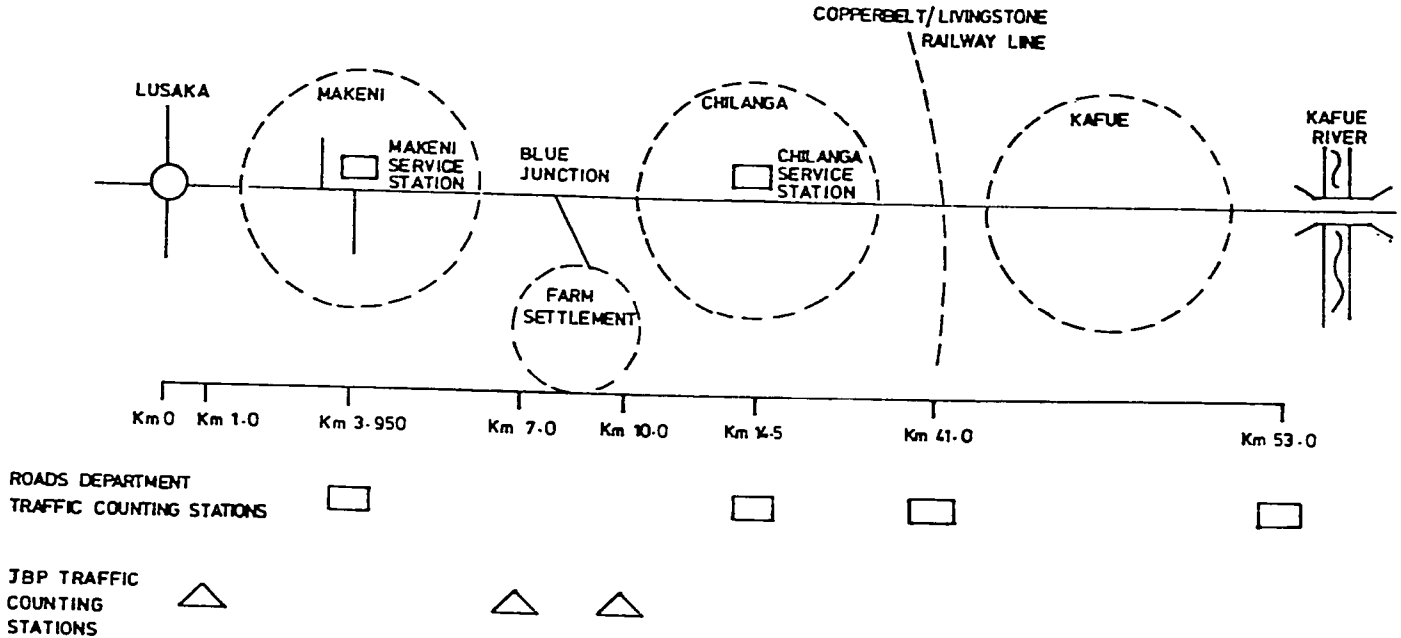


FIGURE 4·1 LOCATION OF TRAFFIC SURVEY STATIONS

Vehicle Type	Day	Apr 1989 km 3.95	Apr 1989 km 14.5	Dec 1988 km 41.0	Apr 1989 km 53.0
Light Vehicles	Mon	9758	1979	890	467
	Tue	9852	1641	960	555
	Wed	8415	2037	1297	485
	Thur	9295	2088	983	567
	Fri	10727	3139	1056	553
	Sat	8365	2027	1089	748
	Sun	6373	1948	925	752
	Total	62785	13859	7200	4127
	Av Daily	8969	1980	1029	590
Single Unit Tracks	Mon	1632	479	106	171
	Tue	1706	355	232	169
	Wed	1362	434	387	186
	Thur	1613	484	284	170
	Fri	1678	499	309	196
	Sat	1101	352	192	183
	Sun	643	257	203	137
	Total	9735	2860	1713	1212
	Av Daily	1391	409	245	173
Trailer or Semi-Trailer Trucks	Mon	313	174	71	115
	Tue	311	169	146	119
	Wed	216	217	279	139
	Thur	332	163	144	105
	Fri	334	196	129	148
	Sat	233	172	111	159
	Sun	204	139	116	115
	Total	1943	1230	996	900
	Av Daily	278	176	142	129

Table 4.1 Part I Survey of Traffic Data provided by Roads Department 2 way - 12 hour Traffic Counts

Vehicle Type	Day	Apr 1989 km 3.95	Apr 1989 km 14.5	Dec 1988 km 41.0	Apr 1989 km 53.0
Buses	Mon	314	88	53	20
	Tue	366	200	69	22
	Wed	372	120	86	16
	Thur	327	105	72	31
	Fri	391	111	96	19
	Sat	287	88	82	15
	Sun	188	58	39	13
Grand Total		76708	18719	10406	6375
Av. Daily		10958	2474	1487	911

Table 4.1 Part II Survey of Traffic Data provided by Roads Department 2 way - 12 hour Traffic Counts

Date	location Km	direction	Private Cars + light Utilities	4 WD mini buses	2-3 axle Rigid Trucks	2-3 axle Trucks + Draw bar Trailer	Articulated Trucks	Buses	12-Hour Total	Peak Hour Total	12-hour C V
Mon 29-05-89	1.0	N	4745	1305	662	115	44	191	7062	912	1012
		S	5251	1058	779	73	84	179	7424	1003	1115
Tues 30-05-89	7.0	N	1211	440	228	26	82	64	2051	267	400
		S	1316	513	254	49	49	65	2246	281	417
Wed 31-05-89	10.0	N	963	410	250	28	66	54	1771	176	398
		S	1040	409	265	68	19	51	1852	213	403

Note: Direction N = North bound towards Lusaka
S = South bound towards Chilanga

4WD = Four wheel drive
CV = Commercial Vehicle

Table 4.2 Summary of Traffic Data collected by the Consultant

Date	Location	Approx km
April 1989	Makeni Service Station	3.95
April 1989	Chilanga Service Station	14.5
Dec 1989	Near Kafue/Lusaka Rail Crossing	41.0
April 1989	Kafue Weighbridge	53.0

The results are summarised in Table 4.1

A cursory analysis of the data shows that there is a marked change of traffic level in the region of Makeni and another change in levels at Chilanga. However the precise locations of change could not be established with the Roads Department traffic data, largely as a result of the choice of junctions and petrol stations as counting points.

4.3 Traffic Surveys

To supplement the existing traffic data, a twelve hour manual classified count was undertaken at the following three locations. These locations and the location of Roads Department counting stations are shown in fig 4.1.

km 1.0	Dual carriageway section
km 7.0	Between Makeni and Blue Boar Junction
km 10.0	Between Blue Boar Junction and Chilanga.

Summary of the results of these surveys are presented in Table 4.2.

4.4 Traffic Growth

The road between Kafue and Lusaka forms the only viable road connection between East and Central Africa and areas south of the Zambezi. At Lusaka, the road network splits into main arterial routes to the north, east and west. Similarly at Kafue the road divides, one section linking Zimbabwe through the Chirundu border post, the other passing through Livingstone to Zimbabwe at the Victoria Falls and Botswana through Kazungula. In pan-African terms this road from Kafue to Lusaka is one of the most important sections of road on the continent.

By virtue of the above geo-political situation, traffic growth on the Kafue-Lusaka road will be dominated not only by factors in Zambia (population production/ consumption trends, currency valuations, availability

of foreign exchange, etc.) but also by events in surrounding countries which may be outside the control or influence of the Zambian government.The estimation of traffic growth rates in such a context is fraught with difficulties.

In projecting future traffic on a given road three types of traffic are considered. Changes in normal traffic due to the Changes in population, economic activity, social factors etc. Diverted traffic which occurs due to the improvement of the given road and the road user using this road in preference to an alternative route that he was using before. Generated traffic where the improvements to the road has resulted in increased economic activity due to reduction in journey times and costs and increased accessibility.

In the case of the Kafue - Lusaka road traffic is unlikely to be generated due to any proposed improvements in itself as a paved road has existed for over 20 years and is still in a condition which is unlikely to offer serious impendiment to transport movements. In the case of diverted traffic, as this has been the only road connection in this region there will be no traffic diverted from any other road due to the improvement of this link.

In 1985 as part of an investigation and documentation study for initiating rehabilitation of four Major Roads in Zambia (Ref 5) including the Kafue Lusaka Road, Kampsax/CarlBro reviewed the historical information that was available with respect to traffic, undertook additional surveys and projected a traffic growth rate of 2.5 percent for the Zambian trunk road network.

This study noted that in 1985 about 24 percent of all traffic on the Kafue - Lusaka Road was "International", or in other words having either an origin or a destination outside Zambia and that 9 percent was just passing through, having neither origin nor destination in Zambia. The traffic pattern changes in this International Component of the traffic in themselves have normal, diverted and generated traffic elements in them. For example, as the Port of Dar-es-salaam reaches capacity saturation and as the operational reliability of Beira Port improves and as the road link to Beira through Mozambique is improved and becomes safer to use, the volumes of international traffic will increase on the Kafue - Lusaka Road. An element of this traffic increase will be growth in the normal

traffic due to increase in population, consumption and production in the region and some will be diverted from other international routes, some of which will be even outside Zambia and the remainder will be generated due to the improvements of other transport routes in the region.

The 1985 study (Ref 5) also examined historical figures and concluded that traffic levels on the Kafue - Lusaka Road network has been declining at a rate of about 6% since 1975. While some of this was due to the poor performance of the Zambian economy, a significant element of this reduction in traffic levels would have been due to the Zimbabwe independence struggle which lasted till 1981 and some more due to the opening of the Tanzania - Zambia Rail Route which would have diverted traffic away from the southern route.

Historical traffic figures were made available by the Roads Department for one traffic counting station on this road near Kafue. The figures seperated by traffic type are shown below and are presented graphically in figure 4.2. The change in traffic level appears to be erratic with increases in some years and decreases in others but in overall term remaining somewhat static.

Year	Light Vehicles	Buses	Single	Articu-lated*	Total
1983	554	27	295	57	933
1984	756	28	279	59	1122
1985	595	23	276	66	960
1986	635	38	282	60	1015
1987	507	29	159	100	795
1988	786	13	243	161	1203
1989**	632	19	173	129	911

Source: Roads Department

* Assumed to include all commercial vehicles other than single limit trucks and buses.

** Not assessed as part of the annual traffic census.

Traffic Growth at Traffic - Point 66A 1983-89

However, one very noteable change is the growth of the articulated type of truck in preference to the single

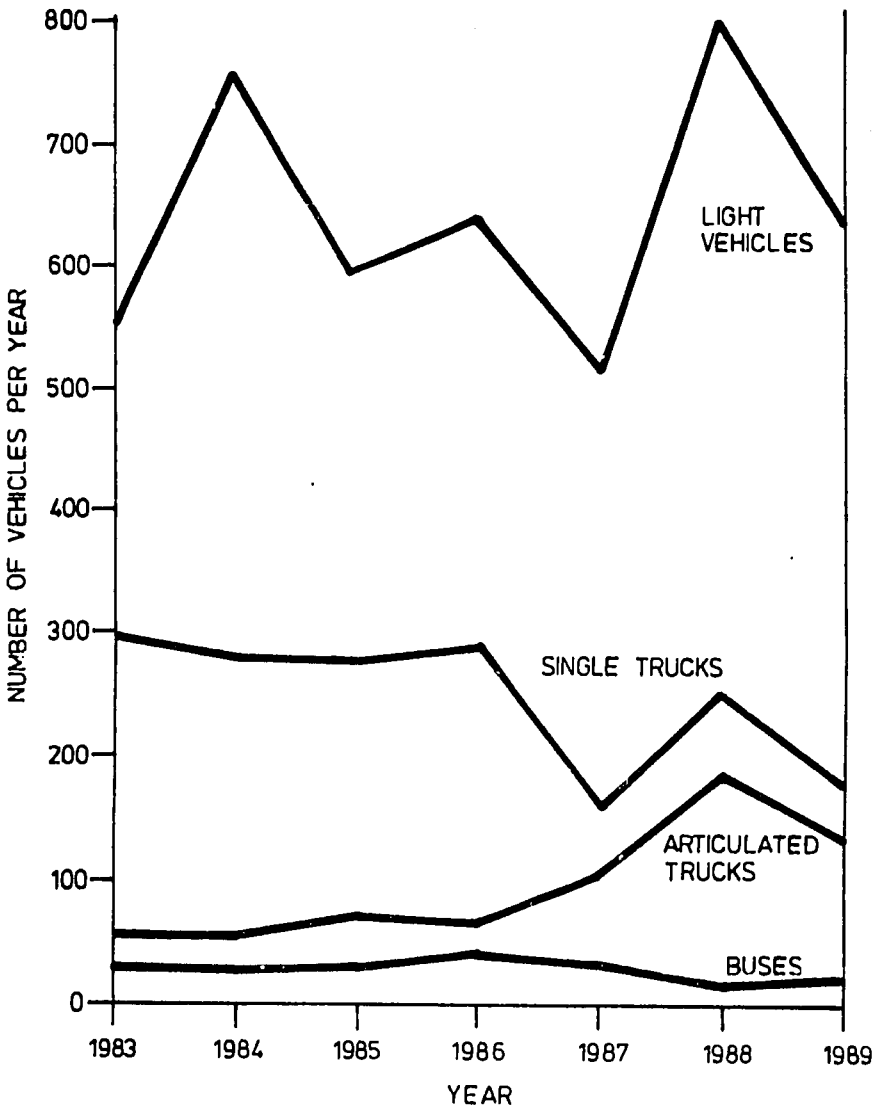


FIGURE 4.2 TRAFFIC AT KAFUE WEIGHBRIDGE (1983-1989)

truck. As a percentage of annual traffic, the truck traffic as defined as a combination of single trucks and articulated truck has remained static in the order of 30 to 35 percent. However, within this total, as the single type of truck traffic reduced the articulated type of truck traffic has significantly increased. This effect is clearly shown in Figure 4.3 where the total truck traffic has been plotted as percentage of total traffic and the articulated truck traffic component has been shown within it. The actual percentage figures are shown in a tabular form below.

Year	Single Unit Truck Traffic as % of Total Traffic	Articulated Truck Traffic as % of total Traffic	Total Truck Traffic as % of Total Traffic
1983	31.6	6.1	37.7
1984	24.9	5.3	30.2
1985	28.8	6.9	35.7
1986	27.8	5.9	33.7
1987	20.0	12.7	32.7
1988	20.2	13.4	33.6
1989	19.0	14.2	33.2

Source: Roads Department

Truck Traffic as a percentage of total traffic at Kafue Weigh bridge (1983 - 1989)

This changing trend should be expected as, the truck fleet in the region and especially in Zambia has been modernised through international doner assistance in the recent years and the newer fleet tend to be less number of larger units. In addition, the international bulk cargo traffic, which due to improved operational performance at the Beira port and due to improved access to the port is beginning to get diverted from the saturated port of Dar-es-salaam, is transported in larger articulated units.

This change in type of truck traffic is of course detrimental to the road pavement and contributes to a faster rate of deterioration of the pavement. As discussed elsewhere heavier traffic has a greater damaging power on the road. Using equivalent standard axle factors calculated from the 1985 Kampsax/CarlBro data and applying it to the roads Department traffic

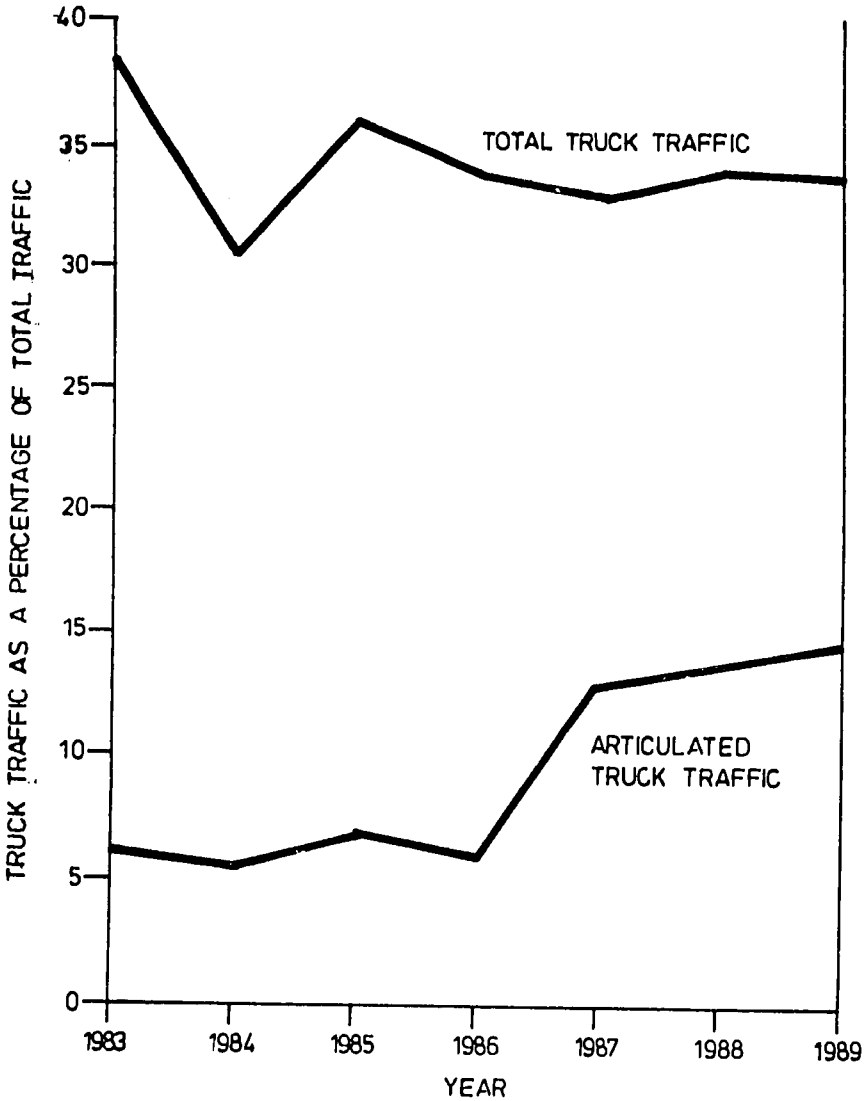


FIGURE 4.3 TRUCK TRAFFIC AS A PERCENTAGE OF TOTAL TRAFFIC AT KAFUE WEIGHBRIDGE (1983 -1989)

figure the Equivalent Standard Axle per commercial vehicle has been calculated for year 1983 to 1989, and has been presented below. As can be seen the equivalent standard axle per commercial vehicle has increased by over 60 percent during this period.

Year	ESA/Commercial Vehicles
1983	2.40
1984	2.46
1985	2.59
1986	2.43
1987	3.59
1988	3.85
1989	3.93

Source: Consultants calculation using Kampasax/Carlo-Bro data.

Average Damaging Power of Commercial Vehicles at Traffic Point 66A

All extraneous, political and economic factors being equal, the growth of commercial traffic could be reasonably estimated on the basis of Gross Domestic Production (GDP) in real terms. As can be seen below, though population has been rising, GDP at constant 1977 prices has remained, over the last few years, remarkably static.

Year	Population	GDP at Constant (1977) Prices
1980	5.86 Million	K 1996 Million
1981	5.87	1982
1982	6.05	2052
1983	6.22	2019
1984	6.42	2012
1985	6.72	2044
1986	6.75	2056
1987	7.27	2052
Avg Growth Rate	+ 3.4%	+ 0.4%

Source: Central Statistical Office

Recent Population and GDP Growth Rates

However, from the changing pattern of the type of truck traffic, where the smaller trucks have been replaced by larger articulated units, it would be true to say that the commercial traffic has grown in terms of tonnage of goods transported. A significant component of this growth is probably atributable to a growth in the international freight movement on this road.

The Fourth National Development Plan 1989-93 prepared by the National Commission for Development Planning has the objective of achieving an overall economic growth rate of 3.0% in real terms over the plan period. The plan intends to achieve this by "Growth from own Resources". Growth from within would suggest a reduction in reliance on external trade and hence a reduction in the number of "international" trucks.

However, growth of the planned nature, even if its from within, would mean increased domestic production for export which would result in an increase in that component of international traffic.

As discussed before, the Volume of traffic with the highest pavement damaging factor has been growing during the 1983 - 89 period.

Growth rate of this component of traffic = $(x/y)^{1/z}$

Where x = percentage of articulated truck traffic in year 1989

y = percentage of articulated truck traffic in year 1983

z = number of years when growth has occured (i.e 1989 - 1983)

Thus, the growth rate for this component of traffic has been 15 percent per annum during the 1983 to 1989 period.

As a first order measure of calculating traffic growth rate the growth rate of the articulated truck could be used to calculate an average rate of traffic growth.

Annual Traffic Growth rate = a x b x 100

Where a = Percentage of articulated truck traffic as a component of the whole traffic

b = the growth rate of this component of traffic

Thus, a first order estimate of traffic growth rate would be 2.1 percent. However, as discussed before traffic estimation and projection on the road must be taken into account the geopolitical situation in the SADCC Region as a whole. The following actions and events in particular will have a significant impact on the traffic levels on the road.

(i) Reduced Regional dependence on South Africa and increased regional cooperation and dependence through SADCC.

(ii) Improved political stability in Mozambique leading to increased operational security of the Beira transport corridor.

(iii) Improved facilities and grater operational reliability of the Beira Port.

(iv) Saturation of capacity of the Dar-es-salaam port.

(v) Independence of Namibia combined with the uncertainties with regard to the Wolves Bay.

Taking into account the above which would largely lead to an increase in heavy vehicle traffic the annual traffic growth should be rounded up to 2.5 percent.

Traffic growth rates are required for a project of this nature from an Engineering Point of View to determin the required volumetic capacity of the road and the pavement strengths that must be provided to cater for this traffic. In addition any Economic viability analysis will require an estimate of benefit in the form of generated activity, savings in vehicle operating cost etc. which are all traffic volume related.

In the case of pavement design for reconstructed sections, the design is largely a step functions and as such any growth factor ranging from 0 to 3.5 percent applied to the traffic figures will give the same results. In the case of volumetic capacity design irrespective of the growth rate within the 0 to 3.5 range, the section between km 0 to 3.9 would have to be dual carriageway and the section from km 15.1 to km 53.3 would have to be to class IB standard. The section of the road that is growth rate sensitive is the one between km 3.9 and km 15.1. However when considering this section of the road the following factors must be noted.

(i) The Boundary of the city of Lusaka has been moved from Makeni km 3.9 to Chilanga km 15.1, reflecting the growing Peri Urban ribbon development stretching as far as Chilanga.

(ii) Chilanga itself is a growing community where in addition to other developments a large hotel complex and a luxury diplomatic type of housing development is planned.

(iii) This section of the road runs through rich farm lands. With increased incentives to the farming sector and the area's proximity to Lusaka agricultural production is likely to grow in this belt.

Having regard to the factors outlined in all the above paragraphs including the historical situation, the consultant has based traffic projections for the Kafue - Lusaka road on current (1989) traffic levels escalated at 2.5% per annum for 10, 15 and 20 year periods. Geometric design and cross section dimensions have been based on 20 year projections but the design of pavement strengthening works has been considered for 10, 15 and 20 year periods. The World Bank Highway Design and Maintenance Model HDM-IIIhas been used to test the cost effective-ness of the additional investment that 15 and 20 year design periods would represent over and above a 10 year design.

4.5 Traffic Forecasts

4.5.1 Cumulative Standard Axles Over Design Life

The design of pavement rehabilitation measures requires a knowledge of the cumulative number of standard (8200 kg) axles to be carried during the design life. Estimations of the cumulative number of standards axles have been made for three values of design life, 10 years, 15 years and 20 years.

The estimates are based on the 12 - hour traffic counts carried out at four locations, converted to a 24 hour value by applying a factor 1.25 calculated by CarlBro/Kampsax from their traffic studies. The traffic counts provided by the Roads Department enabled calculation of daily variation factors to arrive at an average daily flow. However it should be noted that when these factors were applied to the supplementary counts the average daily daily flows calculated gave a higher traffic volume at km 10.00 than at km 7.00. This inconsistancy could have been expected as the Roads

Department counts were undertaken on different days leading to a different set of daily variation factors. Thus, these factors were not applied to the supplementary traffic counts. But only used with the Roads Department traffic figures. Seasonal variation factors were not available but are not likely to be significant.

The equivalence factors (number of standard axles/commercial vehicle) applied to each vehicle group were taken from the results of axle load surveys carried out by Kampsax/Carl Bro (Ref 1).

The estimation of the number of standard axles/12 - hour day is summarised in Table 4.3. The traffic forecasts for three values of design life are presented in Table 4.4. The forecasts are based on the relationship:

$$S = 365 \times So \times \frac{(1+r)n - 1}{r}$$

Where S = Cumulative no standard axles over design life

So = no. standard axles/24 hr day in opening year.

n = design life (years)

r = annual growth rate

For an annual growth rate of 2.5% (r = 0.025), this reduces to:

For design life n = 10 years, S = 4090 So
n = 15 years, S = 6545 So
n = 20 years, S = 9324 So

1.5.2 Total Number of Vehicles in design Year

The selection of the appropriate road cross-section (i.e. carriageway and shoulder widths) is based on the daily total number of vehicles using the road in the design year. Estimates of the daily flow (2 - way 24 hrs) after 10, 15 and 20 years have been made at four locations on the study road (Table 4.5). The estimates have been calculated using the 12 hour traffic counts carried out in 1989 at km 1.0, km 7.0 and km 53.0 and in 1988 at km 41.0 and adjusted with daily variation factors derived from the Roads Department seven day counts and in the case of the traffic figure at km 41.0 escalated by 2.5% to take into account growth from 1988 to 1989.

$$\text{Av. daily flow} = \frac{\text{Daily flow for specific day}}{\text{Variation factor for same day}}$$

Variation factor calculated from Roads Department seven day counts = Specific day flow divided by Average daily flow.

Location (Date)	Traffic Group	No of Commercial Vehicles in average 12 hr day	Standard Axles per Vehicle	SA/12 hr day	Total SA/12 hr day
km 1.0 Mon 29-5-89	2-3 Axle Rigid	662	1.5	993	
	2-3 Axle + Trailer	115	7.0	805	
	Articulated	44	8.5	374	
	Buses	191	1.2	230	2402
km 7.0 Tue 30-4-89	2-3 Axle Rigid	228	1.5	342	
	2-3 Axle + Trailer	26	7.0	182	
	Articulated	82	8.5	697	1298
	Buses	64	1.2	77	
km 10.0 Wed 31-5-89	2-3 Axle Rigid	250	1.5	375	
	2-3 Axle + Trailer	28	7.0	196	
	Articulated	66	8.5	561	1197
	Buses	54	1.2	65	
km 41.0 (Dec 1988)	Single Unit	123	1.5	185	
	Trailer/Semi trailer	71	7.8	554	782 (1988) 802 (1989)
	Buses	36	1.2	43	
km 53.0 (April 1989)	Single unit	87	1.5	131	
	Trailer/Semi-trailer	65	7.8	507	650
	Buses	10	1.2	12	

TABLE 4.3 Estimation of the Number of Standard Axles SA/12 hr Day

(One - Way Towards Lusaka)

km	Section km - km	No. Standard axles 1989		cumulative million standard axles after		
		12 hr day	24 hr day	10 yrs	15 yrs	20 yrs
1.0	0 - 3.9	2402	3003	12.3	19.7	28.0
7.0	3.9 - 15.1	1298	1623	6.6	10.6	15.1
41.0	15.1 - 43.9	802	1002	4.1	6.6	9.3
53.0	43.9 - 53.3	650	813	3.3	5.3	7.6

Note: 24 hr day = 12 hr day x 1.25 (Standard axle count)

TABLE 4.4 Estimation of Cumulative Standard Axles (Lusaka Bound)

km	Section km - km	All vehicles (2-way flow, 1989)					
		12 hour counted	12 hour average daily calculated	24 hour Average daily	24 hr flow after		
					10 yrs	15 yrs	20 yrs
1.0	0 - 3.9	14486	13228	16535	21200	24000	27100
7.0	3.9 - 15.1	4297	3852	4815	6200	7000	7900
41.0	15.3 - 43.9	-	1524	1905	2440	2760	3120
53.0	43.9 - 53.3	-	911	1139	1460	1650	1870

Note: 12 hour counted flow for table 4.2 and 12 hour average calculated as discussed in 4.5.2

Table 4.5 Estimation of the total daily flow in the design year

The Monday factor of 0.828 is used on the 12 hour count conducted at km 1.0 on Monday and a factor of 0.69 is used on the counted figure at km 7.0 to take into account the fact that the count at this location was carried out on Tuesday. Seasonal variation factors which are normally incorporated into this calculation, were not available and thus have not been used. However these are not likely to be significant as traffic on this road is not likely to be significantly influenced by such factors as climate, major harvest seasons or annual holiday seasons.

The estimates are based on the relationship

$$Fn = Fo\ (1+r)^{n}$$

Where Fn = Daily 2-way flow in n th year
Fo = daily 2-way flow in n opening year
n = design life
r = annual growth rate

For an annual growth rate of 2.5% (r=0.025) this reduces to:

for design life n = 10 years, Fn = 1.280 Fo
n = 15 years, Fn = 1.448 Fo
n = 20 years, Fn = 1.638 Fo

4.5.3 Peak hour flow in design year

Estimates of the peak hour flow in the design year have been made for the most heavily trafficked sections of the study road. The estimates are based on the 12 hour traffic counts carried out in May 1989, and incorporate a growth rate of 2.5 per cent per year. Details are given in Table 4.6.

4.6 Geometric Design Standards - Concept

Geometric design is the process whereby the layout of the road in the terrain is designed to meet the needs of the road user. The principal elements of this process are the selection of suitable design speed, horizontal and vertical alignments and road widths. The geometric design standards provide the link between the cost of building the road and the costs of the road users. Usually, the higher the geometric standard, the higher the construction cost and the lower the vehicle costs and road accident costs.

Traditionally the first step in selecting geometrical standards is to choose the 'design speed' for the road. However for geometric design purposes, the function of the road must be considered and in general the road will fall into one of the following categories:

Access Roads - Lightly trafficked roads providing basic means of communication between centres of population or between a centre of population and an existing road. Geometric standards relatively unimportant.

Collector Roads - Generally low volume roads where most important geometric considerations are road width and gradient. Road strength/condition more important than alignment considerations.

Arterial Roads - Generally high volume routes where operational efficiency is a significant factor and hence geometric design standards assume some importance.

It is clear that design standards are related to the topography. In flat undeveloped terrain, the cost is independent of the alignment with the result that a high design standard can be adopted with little cost penalty. However in hilly or mountainous areas, significant earthworks will be involved in providing a higher standard of alignment. While it is desirable from a safety point of view to provide consistent design standards along the route, it is usually cost effective to lower the alignment standards in more rugged terrain.

The basic elements of geometric design are summarised below.

Design Speed - The design speed is the maximum speed at which drivers can travel along the route under normal traffic flows, with safety. The design speed is clearly related to the topography and also sets the minimum horizontal and vertical curvature standards and sight distance.

Horizontal Alignment	-	The horizontal alignment is defined by a series of curves and straights. The standard of alignment is determined by the design speed which in turn determines the minimum curve radius. The lengths of horizontal curves should be kept as short as possible to provide maximum lengths of road with adequate overtaking sight distance. Longer curves with more gentle bends (i.e. higher radii) are not recommended as sight distance are reduced.
Vertical Alignment	-	The vertical alignment of a road has a strong influence on the operating cost of vehicles using the road and the number of accidents, as well as the cost of construction. The vertical alignment is defined by a series of curves (sag or crest) and straight gradients. For main routes, minimum values for the radii of vertical curves are generally based on safety criteria such as stopping sight distance and overtaking sight distance.
Sight Distance	-	This is the distance ahead that can be seen by the driver. The sight distance needed for safe stopping from normal travelling speed is the "stopping sight distance" while the sight distance needed to see ahead for safe overtaking is the "overtaking sight distance".
Cross Section	-	Carriageway and shoulder widths are based principally on projected traffic flows in a given design year, but the composition of traffic also has an important bearing. For example a high proportion of heavy vehicles may dictate wider lane widths and shoulder widths.

The specification of geometric design standard will include the following details.

Horizontal) alignment)	Minimum radius of curvature Minimum stopping sight distance Minimum overtaking sight distance
Vertical) alignment)	Maximum gradient Length of maximum gradient Minimum stopping sight distance or overtaking sight distance on crest curves Length of sag curves
Cross-section	Width of carriageway (running surface) Width of shoulders Width of structures Width of road reserve

4.7 Geometric Design Standard for Single Carriageways

The design standards for capacity of single carriageways, as used by the Zambian Roads Department (Ref 6), are summarised in Table 4.7. Also included, for comparison, are the latest recommendations for single carriageways from the Overseas Unit of the Transport and Road Research Laboratory, published in Overseas Road Note 6 (Ref 7).

It is clear that the Zambian standards are more conservative than those given in Overseas Road Note 6 and are given priority. The resulting carriageway widths, based on traffic flows projected to the 20th year are given in Table 4.8.

4.8 Geometric Design Standards for Dual Carriageways

The traffic flow on the existing dual carriageway section of the study road (km 0 -3.9) lies outside the range appropriate for single carriageways. The capacity of this section has been checked using the recommendations of the U.S. Bureau of Public Roads Highways Capacity Manual (Ref 8). This gives a design capacity on freeways and expressways with partial control of access and with little interference from cross traffic at 1000 passenger vehicles per hour per 3.65 m lane (i.e 2000 passenger vehicles in one direction of the dual section).

Km	Section km - km	Peak hour flow (May 1989) (Table 4.2)		Max peak hour flow (1 - way) after		
		North bound	South bound	10 yrs	15 yrs	20 yrs
1.0	0 - 3.9	912	1003	1280	1450	1640
7.0	3.9 - 15.1	267	281	360	410	460

Table 4.6 Estimation of peak one hour flow in the design year

	Design Class	Traffic Flow 2 - Way ADT	C/Way Width (m)	Shoulder Width (m)	Max Gradient * (%)	Design Speed * (km/h)
Zambian Roads Dept.	IA	1500 - 5000	7.3	3.0	4/6/7	100/100/80
	IB	500 - 1500	6.7	2-3	5/6/8	100/80-100/60 80/60
	IC	up to 500	6.1	2.0	6/7/8	100/80/60
Overseas Road Note 6 (Arterial Roads)	A	5000 - 15000	6.5	2.5	8	85/100/120
	B	1000 - 5000	6.5	1.0	8	70/85/100
	C	400 - 1000	5.5	1.0	10	60/70/85

* Flat/Rolling/Mountainous topography

Table 4.7 Comparison of Design Standards for Single Carriageways

The peak hour flow (in one direction) on the existing dual section is estimated to be 1640 vehicles after 20 years (Table 4.6). The equivalent number of passenger vehicles is estimated at 2000, based on 22% commercial vehicles and an equivalence factor of 2 passenger cars to 1 commercial vehicle.

Vehicle type	No vehicles	Passenger vehicles per vehicle	No. Passenger Vehicles
Cars	1394	1	1394
C.V	246	2	492
Total	1640		1886

This indicates that after 20 years the peak hour flow will be 2000 passenger vehicles in one direction which equals the design capacity.

A similar calculation of one way peak hour flow has been made for the road section between Makeni and Chilanga (km 3.9 - 15.1), based on 19% commercial vehicles and 2 passenger cars per commercial vehicle.

Vehicle type	No vehicles	Passenger vehicles per vehicle	No. Passenger Vehicles
Cars	386	1	386
C.V	92	2	184
Total	460		552

This gives an equivalent peak hour flow after 20 years of 552 passenger vehicles, which is well within the capacity of 1000 passenger vehicles in each direction. (For Class IA construction, carriageway width 7.3 m gives lane width 3.65 m).

		1989 + 20 years		
Section	2 - way 24 hr flow (all vehicles)	1 - way Peak hour flow (all vehicles) (Table 4.6)	1 - way Peak hour flow (Passenger Vehicle)	Capacity (Passenger Vehicles) (1 hour)
0 - 3.9	27100	1640	1886	2000
3.9-15.1	7900	450	552	1000

Peak Hour Flows between Lusaka and Chilanga

4.9 Recommended Geometric Design Standards

4.9.1 Lusaka to Makeni

Based on the peak hour traffic flows, it is recommended that the section between Lusaka and Makeni (km 0 - km 3.9) should be a dual carrigeway with 7.3 m carriageway width in each direction. A central median strip (3m wide) should be provided, and shoulder width should be 3.0 m. In selecting the dual carriageway option consideration was given to traffic safety and road maintenance aspects in addition to traffic volume levels.

4.9.2 Makeni to Chilanga

The peak hour flow predictions for the 20th year indicate that a single carriageway will provide sufficient capacity on this section (km 3.9 - Km 15.1). However, the predicted Annual Daily Traffic after 20 years (7900 vehicles) exceeds the upper limit for single carriageways (5000 vehicles) quoted in the recommendations of the Zambian Roads Department.

The option of providing a dual carriageway on this section has been considered, in view of the number of slow moving vehicles and also the limited opportunities for overtaking due to the horizontal alignment. However from capacity considerations it is considered that the dualling option is not warranted.

It is recommended that this section should be a Class IA Road, and that the existing road should be realigned at two locations, in the vicinity of km 7.5 (Eureka Corner) and km 12.5 (Stream crossing near a

left hand bend at the bottom of a dip). At both locations the horizontal alignment can be improved, and at the second the road level needs also to be raised to facilitate surface drainage.

4.9.3 Chilanga to Kafue

The design ADT for this section (km 15.1 - km 43.9) is 2440 vehicles after 10 years and 3120 vehicles after 20 years. Reference to the Zambian design standards (Table 4.5) indicates that both of these volumes are within the range (1500 - 5000) appropriate to Class IB roads. However, more recent design standards contained in Overseas Road Note 6 suggest a carriageway width of 6.5 m for the traffic range 1000 - 5000 vehicles. This carriageway width is below the Zambian standard for Class IB roads which is 6.7m.

Therefore it is recommended that this section should be a Class IB road with carriageway width 6.7m and 3 m wide shoulders.

4.9.4 Kafue to Kafue Bridge

The design ADT for this section (km 43.9 - 53.3) varies from 1460 vehicles after 10 years to 1870 vehicles after 20 years. It is recommended that this section should be class IB, with carriageway width 6.7m and 3 m wide shoulders.

4.9.5 Summary of Recommendations

The recommended geometric design standards are indicated in Table 4.9.

Km	2 way 24 hour flow all vehicles				Geometric Standards (1989 + 20 years) Zambia			ORN6		
	1989 from table 4.5	+ 10	+ 15	+ 20 yrs	Class	C/way Width (m)	Shoul-der width (m)	Class	C/Way Width (m)	Shoul-der Wi[illegible] (m)
0-3.9	16535	21200	24000	27100	n.a	n.a	n.a	-	-	-
3.9-15.1	4815	6200	7000	7900	n.a	n.a	n.a	A	6.5	3.0
15.1-43.9	1905	2440	2760	3120	IB	7.3	3.0	B	6.5	1.0
43.9-53.3	1139	1460	1650	1870	IB	7.3	3.0	B	6.5	1.0

ORN6 = Overseas Road Note No. 6
(Issued by Transport and Road Research Laboratory)

Table 4.8 Geometric Standards based on traffic flows after 20 years

Section (km)	Road Class	Width (m) C/Way	Shoulder
0 - 3.9	Dual 2 - lane	2 x 7.3	3.0
3.9 - 15.1	IA	7.3	3.0
15.1 - 43.9	IB	6.7	3.0
43.9 - 53.3	IB	6.7	3.0

Table 4.9 Recommended Geometric Design Standards

5 DEFLECTION SURVEY

5.1 Existing Data

5.2 Deflection Survey

5.3 Condition Survey Associated with Deflection Survey

5.4 Control Sections

5. DEFLECTION SURVEY

5.1 Existing Data

Deflection data was available for the road section between Chilanga (km 14.3) and Kafue Bridge (km 53.3). The deflections were measured in late 1985 by Kampsax/Carlo Bro (Ref. 5). The earlier study provided Benkelman Beam deflections using the transient procedure and a 6350 kg load at 200 m intervals and Falling Weight deflection (FWD) deflections at 500 m intervals. A correlation between deflections measured with the Benkelman Beam and FWD was established as part of the former study (Ref 5). This had the form:

$$BB = 0.62\ (FWD)^{1.15}$$

where BB = Benkelman Beam deflection
FWD = Falling Weight Deflectometer deflection.

5.2 Deflection Survey

To complete the deflection data on the study road, a deflection survey was carried out in May 1989 on the section of road between the Kafue Road turning circle (km 0.0) and Chilanga (km 14.3). The deflections were measured at 100 m intervals using a Benkelman Beam (transient procedure using 6350 kg rear axle load). Details are given elsewhere (Ref 9). The measured deflection are shown in Appendix A.

The deflections were measured with pavement surfacing temperatures in the range 20 - 40 degrees Centigrade. A temperature study was carried out whereby deflections were measured repeatedly at the same locations over a range of pavement temperatures (recorded at the base of the bituminous layer, typically 25 - 35 mm depth below the road surface). The plots of deflection vs temperature showed some scatter of results but no deflection temperature relationship was apparent.

5.3 Condition Survey Associated with Deflection Survey

At each deflection measurement point, the condition of the pavement was assessed in terms of rutting and cracking. The rut depth was measured using a 2m straight edge and calibrated wedge. The type and intensity of surface cracking was assessed within the square metre surrounding the measurement point. Details are given below.

Crack Type	Description
N	None
S	Single crack (or several single cracks not connected)
C	Connected (but not interconnected) Cracking
I	Interconnected or block craking
P	Disintegration or potholing

Crack Intensity	Amount of Cracking
0	Nil
1	Less than 0.5 m/sq.m
2	0.5 - 1
3	1 - 2
4	2 - 5
5	More than 5 m/sq.m

5.4 Control Section

In order to check the change in pavement condition between 1985 and 1989 of the road section between Chilanga (km 14.3) and Kafue Bridge (km 53.3), six control sections were selected for detailed condition survey. The control sections (each 1 km long) were chosen to represent the full range of pavement construction and condition on this section of road. The results of the surveys were then compared with the data collected by Kampsax/Carlo Bro (Ref 5) in 1985. The location of the control sections is as follows.

Control Section	Details
A	km 30.0 km 31.0, southbound carriageway. Uphill section with areas of block cracking, Relative roughness 2158 mm/km.
B	km 49.0 - km 50.0, southbound carriageway. Flat section between Kafue and Kafue Bridge with potholes. Relative Roughness 2796 mm/km.
C	km 42.0 - km 41.0, northbound carriageway. Uphill section with extensive block cracking. Relative roughness 2075 mm/km.

D	km 40.0 - km 39.0, northbound carriageway. Flat section just south of railway crossing with areas of block cracking. Relative Roughness 1539 mm/km.
E	km 29.0 km 28.0, northbound carriageway with areas of block cracking and patching. Relative Roughness 2266 mm/km.
F	km 19.0 - km 18.0, northbound carriageway. Downhill/Uphill section with drift crossing and areas of block cracking/potholes. Relative roughness 3541 mm/km.

At each control section, Benkelman Beam deflections were measured at 100m intervals (200 m intervals at control section C) and an associated condition survey carried out (refer section 5.3).

A comparison between deflections measured in 1985 by Kampsax/Carlo Bro (Ref 5) and those measured in 1989 indicates that at each control section there has been an increase in deflection level. The results are summarised in Table 5.1. The increase in deflection level is not the same for each control section, with the most significant increase occurring at control section B, which is located between Kafue and Kafue Bridge. Based on the comparison of design deflections (95th percentile), a growth factor was selected for application to be values of design deflection deduced from the 1985 deflection data. Details are given in Table 5.2

Based on the comparison between deflections measured over the same road lengths in 1985 and 1989, an annual growth rate has been deduced. This has been used to project deflections to 1991 levels to obtain the overaly design deflections. (1991 is the programmed data for start of construction). Details are given in Table 5.2. An annual growth rate of 6.8 percent has been taken for the section between Lusaka and Chilanga, based on observations at the Control Sections.

Control Section	Design Deflection mm/100		Ratio 1989/1985	Annual Growth Rate %
	1985	1989		
A	61.3	69.7	1.14	3.3
B	23.4	58.4	2.50	26.0
C	52.0	94.5	1.82	16.0
D	57.4	67.6	1.18	4.2
E	68.0	99.4	1.46	9.9
F	139.0	179.1	1.30	6.8

Table 5.1 Comparison of Deflections measured in 1985 and 1989

Section Km - Km	Annual growth rate %	Based on Control Section	Ratio 1989/1985	Ratio 1991/1989
14.5 - 23.6	6.8	F	1.30	1.14
23.6 - 32.0	9.9	E	1.46	1.21
32.0 - 45.0	4.2	D	1.18	1.09
45.0 - 53.3	26.0	B	2.50	1.60

Table 5.2 Growth Factors for Design Deflection

6. TEST PIT AND DYNAMIC CONE PENETROMETER SURVEY

6.1 Existing Data

6.2 Test Programme

6.3 DCP Survey

6.4 Overlay Design Based on DCP Structure Number

6.5 Materials Test Results

6.6 Pavement Moisture Conditions

6. TEST PIT AND DYNAMIC CONE PENETROMETER SURVEY

6.1 Existing Data

The Kampsax.CarlBro study report (Ref 5) gives details of pavement layer thicknesses at several locations between Chilanga (km 14.3) and Kafue Bridge (km 53.3), based on Dynamic Cone Penetrometer (DCP) tests. In order to supplement the data for this section and to provide layer thickness/materials information for the section between Lusaka (km 0.0) and Chilanga (km 14.3), a programme of Test Pits and DCP tests was undertaken in May 1989.

6.2 Test Programme

Between Lusaka (km 0.0) and Chilanga (km 14.3), test pits were excavated at approximately 2 km intervals (on alternate sides of the road) to determine layer thicknesses and obtain material samples from the pavement layers and the subgrade for identification testing and in situ moisture content determination.

Between Chilanga (km 14.3) and Kafue Bridge (km 53.3) a further series of test pits were excavated to supplement the data provided by Kampsax/CarlBro (Ref 5). The location of the test pits were chosen so as to represent the full range of pavement condition and topography on the study road. This included a test pit at or close to each of the six control sections (refer section 5.4).

A schematic plan showing the location of the test pit/DCP sites is given in Figure 6.1. Test pits 1 to 16 were excavated in the hard shoulder at the edge of the carriageway, and material samples collected from the exposed face adjacent to the carriageway. Test Pits 17 and 18 were excavated in the outside wheeltrack of the Lusaka bound traffic lane. The DCP tests were carried out in the outside wheeltrack of the carriageway adjacent to each test pit location. A schedule of Test Pit and DCP test locations is included in Table 6.1.

6.3 DCP Survey

The DCP tests were carried out in the outside wheeltrack adjacent to each test pit location. Before commencing the test procedure, the bituminous surfacing was removed in the vicinity of the test point, and the test started from the top of the roadbase layer.

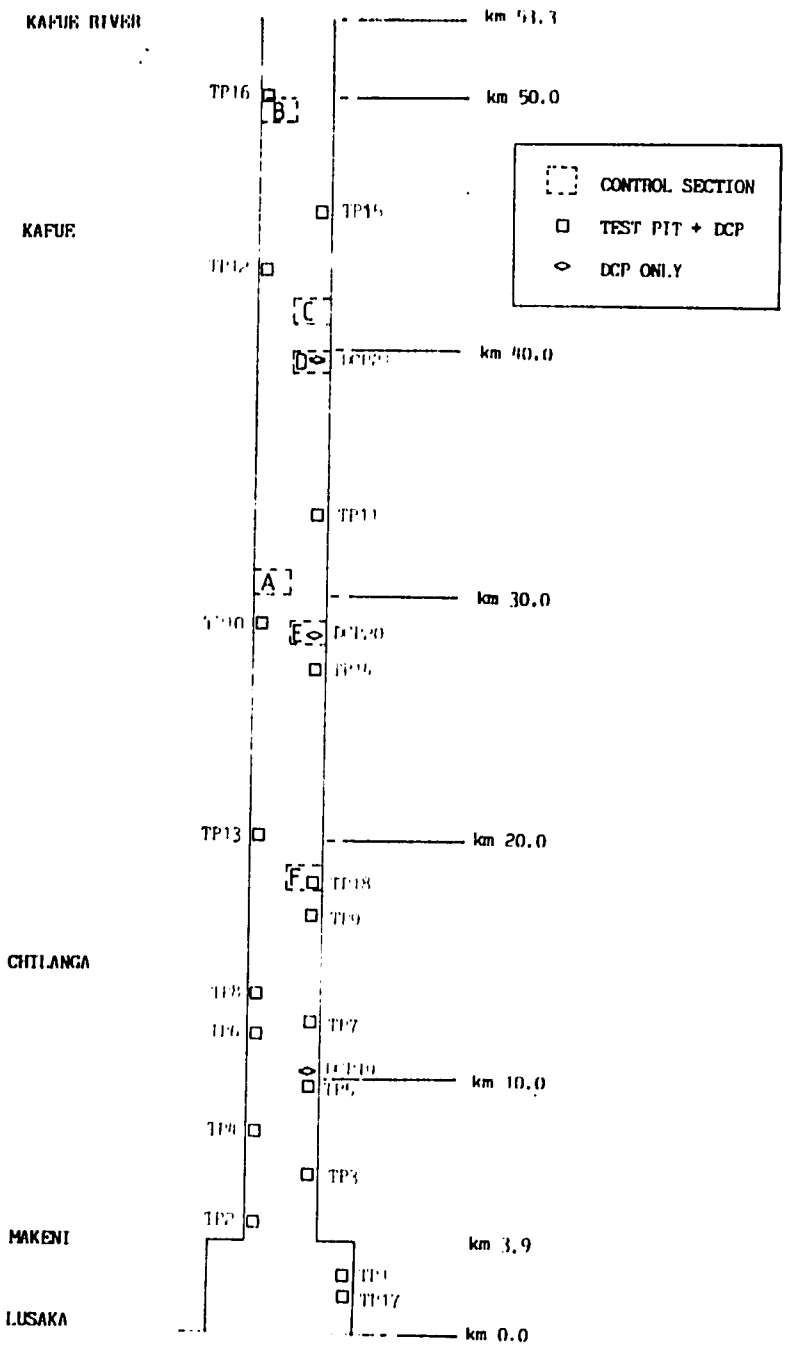


Figure 6.1 Test Pit and DCP test locations

Km	Carriageway	TP	DCP
1.5	N	17	17
2.2	N	1	1
4.5	S	2	2
6.4	N	3	3
8.1	S	4	4
9.9	N	5	5
10.5	N	-	19
12.0	S	6	6
12.5	N	7	7
13.7	S	8	8
16.9	N	9	9
18.2	N	18	18
20.2	S	13	13
27.0	N	14	14
28.4	N	-	20
28.8	S	10	10
33.2	N	11	11
39.5	N	-	21
43.1	S	12	12
45.5	N	15	15
50.1	S	16	16

Note: TP Trail Pit Number
DCP Dynamic Core Penetration Test Location Number

Table 6.1 Location of test sites

The DCP survey results are presented (Appendix B) in the form of a graphical plot of penetration against the cumulative number of blows applied. Each layer in the pavement construction is represented by a penetration vs. blows line of uniform slope, and layer boundaries can generally be identified by a change in the slope of this line. The in situ CBR of each layer may be deduced from the slope of the line plotted (mm/blow) using the relationship produced by Kleyn and Van Heerden (Ref 10), given in Figure 6.2. The resulting layer thicknesses and in situ CBR values are included in Table 6.2. Also included in Table 6.2 are values of the DCP Structural Number (DSN) and Balance Number (BN). These parameters are discussed further in Section 6.4.

The variation of the pavement layer CBR values (as deduced from the DCP tests) along the study road is shown in Figure 6.3. Between km 0 and km 10.0, the CBR value deduced for the roadbase layer varies between 42% and 100%. Sub-base CBR values are more variable, ranging between 30% and 100% along the length of the road.

The variation in subgrade CBR is also shown in Figure 6.3. This is used in the determination of subgrade resilient modulus for mechanistic analysis (refer Section 10.5). Between Lusaka and Chilanga the subgrade CBR varies between 2% and 60% with a mean value of 20%. The lowest recorded value occurs at km 2.2 which is within one of the low lying sections scheduled for raising to facilitate surface drainage (refer Section 8.2). Between Chilanga and km 20.0 the deduced subgrade CBR values remain uniform at 20%, but become more variable between km 20.0 and km 32.0, with a low value of 4% recorded at km 28.4. The low value occurs in a relatively flat, low lying section and in this vicinity the in situ moisture content is relatively high. Between km 32.0 and the kafue Bridge, the deduced subgrade CBR levels are consistently higher than 20%, with the highest levels recorded at km 33.2 and km 45.5.

The design CBR values assumed for mechanistic analysis (Section 10.5) are indicated on Figure 6.3. The resilient modulus of the subgrade layer is based on the CBR value, and a CBR of 20% gives the maximum value (120 MN/m2) which can be taken for this parameter.

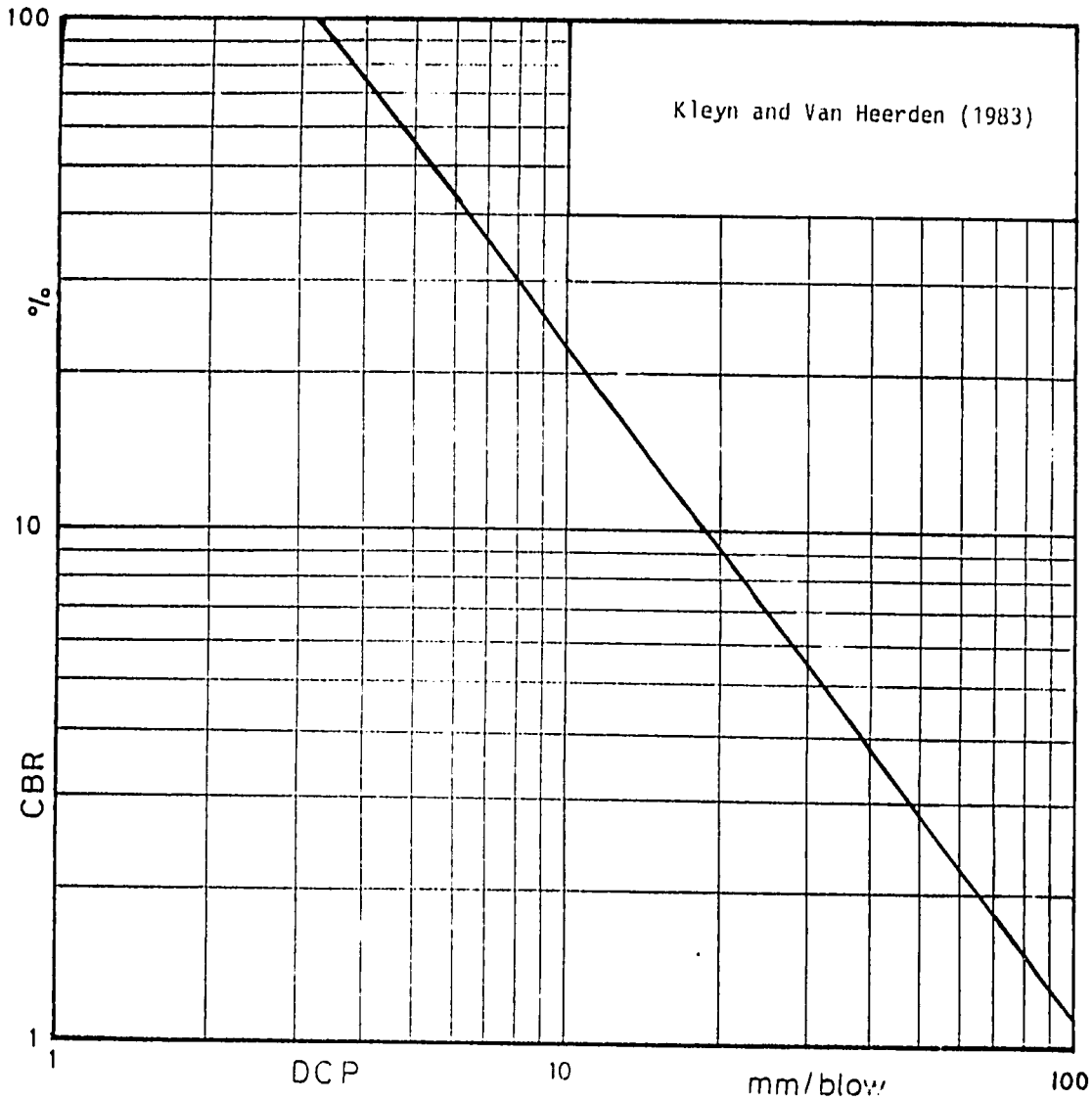


Figure 6.2 DCP/CBR RELATIONSHIP (60 deg cone)

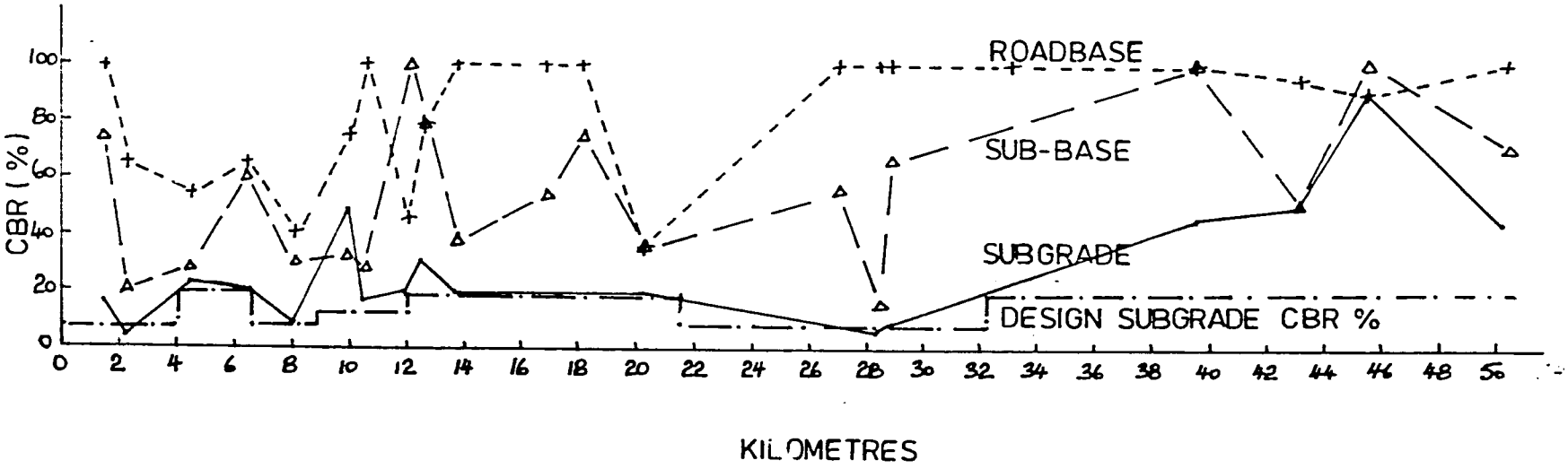


FIGURE 6.3 VIBRATION OF PAVEMENT LAYER CBR VALUES

6.4 Overlay Design based on DCP Structure Number

The accumulated pavement strength with depth is indicated on the Strength-Balance curves (Refs 11,12) contained in Appendix B. At a given depth, the accumulated strength is simply the percentage of the DCP Structural Number (DSN) achieved at that depth. The DCP Structural Number is defined as the number of blows needed to reach a penetration of 800 mm, which is normally taken as the total pavement depth. The Balance Number (BN) shown on the curves is the percentage of the DCP Structure Number achieved at a depth of 100mm (12.5% of the pavement depth).

$$BN\ (\%) = \frac{\text{No of blows to reach 100mm}}{\text{No of blows to reach 800mm}} \times 100$$

The higher the Balance Number, the more the upper layers contribute to the overall structural strength.

Values for DSN and BN have been calculated at each DCP test location (Table 6.2). The mean values over defined sections of road are given below.

Section	DSN	BN(%)	Comments
0 - 3.9	113	24	Weak/well balanced
3.9 - 15.1	151	19	Not very strong/well balanced
15.1 - 30.0	156	21	Not very strong/well balanced
30.0 - 43.9	251	8	Strong/deep balanced
43.9 - 53.3	350	12	Strong/deep balanced

The values indicated ignore any sturctural contribution from the existing surfacing.

Overlay thickness requirements may be estimated using a design DSN value based on future traffic levels. The relationship used to predict the design DSN value is given by:

$$\text{Design DSN} = (N \times 10^{9}/\text{cm})^{0.2857}$$

where N = design traffic in m.s.a.

Km	Layer Thicknesses (MM)			C B R (%)			DSN	BN %
	Surfacing	Roadbase	Sub-base	Roadbase	Sub-base	Subgrade		
1.5	35	165	220	100	74	16	154	26
2.2	25	180	270	65	35/20	2	72	21
4.5	25	180	150	55	28	23	105	14
6.4	25	175	360	65	100/60	20	158	11
8.1	30	200	160	42/100	30	8	98	12
9.9	25	110	320	75	33/90	50	143	13
10.5	25	75	330	100	28/40	16	144	28
12.0	25	180	180	95/45	100	47/20	152	13
12.5	25	165	145	100/80	100/80	31	154	18
13.7	25	165	145	100	75/38	20	255	44
16.9	25	155	100	100	55	20	117	21
18.2	35	95	290	100	75	20	154	23
20.2	25	145	250	35	60/35	20	92	12
27.0	25	155	110	100	55	(75)	254	14
28.4	30	370	180	100	15	4	-	-
28.8	15	120	205	100	100/66	8	162	33
33.2	R E	F U S	A L	100	-	-	-	-
39.5	40	105	135	100	100	45	271	9.2
43.1	50	270	140	95	50	50	232	6.4
45.5	10	190	180	90	100	90	475	7.8
50.1	25	170	70	100	70	90/43	225	16.4

Note 1: Refusal at km 33.2 due to presence of boulders and cobbles in the pavement layer the the dynamic core penetrometer rod could not be drive into the pavement.

Note 2: CBR - California Bearing Ratio
DSN - DCP Structure Number
BN - Balance Number (Ref 6.4)

Table 6.2 Summary of DCP Test Results

Cm is a constant, the value of which depends on the moisture conditions which would generally apply during the design life.

Cm = 64	dry conditions
Cm = 30	optimum moisture content (omc).
Cm = 6.5	wet conditions

The above relationship is based on experience in S. Africa (Ref 12).

The basis for overlay design by this approach is to compare the design DSN and existing DSN values, and make up the difference with bituminous material, assuming a strength (penetration rate) of 0.6mm/blow. The resulting overlay designs for a 10 year design life are summarised in Table 6.3. It is considered that the thicknesses derived for wet conditions are the most appropriate. However, it is felt that overlay design by this method can at best be regarded as a "broad brush" approach since the DCP tests are carried out at intervals of 2 km are more. Deflection analysis, based on deflections measured at 100 - 200m intervals, has been adopted for the final overlay thickness selection (refer Section 10.4).

6.5 Materials Test Results

6.5.1 Introduction

At each test pit, the separate pavement layers were identified by visual examination of the profile at the exposed face of the pit adjacent to the carriageway. There was generally quite good agreement between layer thicknesses identified from the test pits and those determined from the results of the adjacent DCP test. The layers identified from the test pits were referenced from the surface, with layer 1 representing the bituminous surfacing (typically 25 - 35mm thick).

The material properties deduced from the laboratory testing programme are given in Table 6.4, and may be defined as follows.

In situ moisture content (mc)	: Field moisture content at time of sampling. This will vary from season to season.

Km	Existing DSN	Design DSN			Estimated Overlay (mm)		
		Dry	omc	Wet	Dry	omc	Wet
0 - 3.9	113	232	289	447	70	110	200
3.9 - 15.1	151	195	242	374	40	55	135
15.1 - 30.0	156	170	211	327	40	40	100
30.0 - 43.9	251	170	211	327	-	-	50
43.9 - 53.3	350	160	198	307	-	-	-

Table 6.3A Estimated overlay requirements for 10 year life based on DCP structure number (DSN)

Km	15 yrs msa	Existing DSN	Design DSN			Overlay (mm)		
			Dry	omc	Wet	Dry	omc	Wet
0 - 3.9	19.7	113	266	330	512	90	130	240
3.9 - 15.1	10.6	151	223	277	429	45	75	170
15.1 - 30.0	6.6	156	196	242	374	40	50	130
30.0 - 43.9	6.6	251	195	242	374	-	-	75
43.9 - 53.3	5.3	350	183	227	352	-	-	-

Note: msa = Million Standard Axles

Table 6.3B Estimated overlay requirements for 15 year life based on DCP structure number (DSN)

Km	15 yrs msa	Existing DSN	Design DSN			Overlay (mm)		
			Dry	omc	Wet	Dry	omc	Wet
0 - 3.9	28.0	113	294	365	566	110	150	275
3.9 - 15.1	15.1	151	247	306	474	60	95	195
15.1 - 30.0	9.3	156	215	267	413	40	70	155
30.0 - 43.9	9.3	251	215	267	413	-	40	40
43.9 - 53.3	7.6	350	203	252	390	-	-	40

Note: msa = Million Standard Axles

Table 6.3C Estimated overlay requirements for 20 year life based on DCP structure number (DSN)

Location	Test Pit No	Layer	Test Pit Layer No.	In situ moisture content %	Liquid limit	Plasticity Index	Linear shrinkage %	Grading modulus	AASHTO classification
1.5	17	Subgrade	3	9.3	NP	NP	0	1.81	A1 - b
2.2	1	Roadbase	2	8.0	SP	SP	2	1.67	A2 - 4
		Sub-base	3	8.0	SP	SP	2	1.71	A2 - 4
		Subgrade	4	8.0	NP	NP	0	1.23	A2 - 4
4.5	2	Roadbase	2	6.5	SP	SP	1	2.05	A1 - b
		Sub-base	3	8.0	SP	SP	2	2.05	A1 - b
		Subgrade	4	8.5	SP	SP	2	0.69	A4
6.4	3	Roadbase	2	7.0	NP	NP	0	1.83	A1 - b
		Sub-base	3	8.0	SP	SP	1	1.79	A2 - 4
		Subgrade	4	17.0	30	16	8	0.03	A6
8.1	4	Roadbase	2	7.5	NP	NP	0	1.73	A1 - b
		Sub-base	3	6.5	NP	NP	0	1.71	A1 - b
		Subgrade	4	10.0	SP	SP	2	0.63	A4
9.9	5	Roadbase	2	9.5	30	10	5	1.77	A2 - 4
		Sub-base	3	9.3	25	10	6	1.89	A2 - 4
		Sub-base	4	9.2	40	13	7	1.60	A2 - 6
		Sub-base	5	9.3	SP	SP	2	1.93	A1 - b
		Subgrade	6	8.9	38	14	8	0.98	A6
12.0	6	Rodbase	2	9.3	26	10	5	1.45	A4
		Sub-base	3	9.6	22	6	3	1.46	A2 - 4
		Sub-base	4	9.5	27	10	5	1.78	A2 - 4
		Subgrade	5	8.9	36	10	6	0.88	A4
12.5	7	Roadbase	2	9.2	28	8	3	2.01	A2 - 4
		Sub-base	3	8.8	24	7	3	1.55	A2 - 4
13.7	8	Roadbase	2	9.4	21	6	3	1.83	A2 - 4
		Sub-base	3	9.3	31	11	5	1.99	A2 - 6
		Subgrade	4	8.8	30	15	7	0.45	A6
16.9	9	Roadbase	2	5.0	28	9	4	1.64	A2 - 4
		Subgrade	3	8.5	31	9	4	1.44	A4
18.2	18	Subgrade	3	8.7	SP	SP	1	1.85	A2 - 4
20.2	13	Roadbase	2	9.0	38	10	5	1.84	A2 - 4
		Subgrade	3	8.5	35	9	5	1.68	A2 - 4

Table 6.4 Part 1 Summary of Pavement Materials Test Results

Location	Test Pit No	Layer	Test Pit Layer No.	In situ moisture content %	Liquid limit	Plasti-city Index	Linear shrinkage %	Grading modulus	AASHTO classi-fication
27.0	14	Roadbase	2	3.5	23	6	3	2.17	A2 - 4
		Sub-base	3	6.0	SP	SP	1	2.12	A1 - b
		Subgrade	4	6.5	23	7	3	1.93	A2 - 4
28.8	10	Roadbase	2	7.5	25	11	5	2.62	A2 - 6
		Subgrade	3	13.0	32	14	7	1.08	A6
33.2	11	Roadbase	2	2.5	24	12	6	2.26	A2 - 6
		Subgrade	3	4.5	27	13	7	1.86	A2 - 6
43.1	12	Roadbase	2	3.0	23	7	4	2.21	A2 - 4
		Sub-base	3	6.0	21	6	3	1.97	A2 - 4
45.5	15	Roadbase	2	3.5	25	10	4	2.07	A2 - 4
50.1	16	Roadbase	2	5.0	39	10	5	2.35	A2 - 4
		Subgrade	3	4.0	SP	SP	1	2.09	A1 - b

Table 6.4 Part II Summary of Pavement Materails Test Results

Plastic Limit (PL)	: Moisture content at which a clay soil passes from the solid to the plastic state.
Liquid Limit (LL)	: Moisture content at which a clay soil passes from the plastic to the liquid state.
Plasticity Index (PI)	: Magnitude of range of moisture contents over which the soil is in the plastic state. (PI = LL - PL)
Linear Shrinkage (LS)	: Shrinkage (expressed as a percentage of the original length) of a specimen after drying out from a state where the moisture content equals the Liquid Limit.
Grading Modulus (GM)	: One hundredth part of the sum of the percentages retained on the 2.00mm, 0.425mm and 0.075mm sieves.
AASHTO Classification	: System of soil classification which divides soils into seven groups designated A - 1 to A - 7, depending on physical properties (particle size distribution, Liquid Limit and Plasticity Index). The lower the classification number, the better the material is from a highway stability point of view.

The grading curves for each of the pavement layers identified at the test pits are presented in Appendix C.

6.5.2 Subgrade

The variation in subgrade material properties along the study road is shown below. The values indicated are the average values for the particular section.

Lusaka	km 0		
		Silty Sand	GM = 1.5 mc = 9%
Makeni	km 3.9		
		Clay	GM < 0.7 LL = 30 PI = 16 mc = 17%
Eureka Corner	km 7.5		
		Silty or clayey Sand and gravel	GM < 0.75 LL = 32 PI = 12 mc = 9.1%
Chilanga	km 14.3		
		Silty or clayey Sand and gravel	GM = 1.6 LL = 30 PI = 10 mc = 9.1%
Kafue Hills	km 30.0		
		Silty sand and gravel	GM = 2.0 LL = 27 PI = 13 mc = 4.3%
Kafue Bridge	km 53.3		

Subgrade material should ideally have a GM > 0.75 and a PI < 30 x GM. On this basis the subgrade material between Makeni and Chilanga is only just acceptable. In addition, the in situ moisture content is highest between Makeni and Eureka Corner. Subgrade quality is good between Lusaka and Makeni and between Chilanga and Kafue Bridge.

6.5.3 Sub-base

The sub-base material falls broadly into three types. At km 1.5, the material consists of a silty sand with no gravel. Between km 2.0 and 7.5, the material is sand and gravel with low fines content. Between km 7.5 and km 53.3, the amterial consists of silty sand and gravel. The variation in material propeties along the study road is indicated below.

km 0		
	Silty sand	GM = 1.7 mc = 8%
km 2.0		
	Sand and gravel	GM = 1.9 mc = 8%
Km 7.5		
	Silty sand and gravel	GM = 1.9 LL = 30 PI = 10 mc = 8%
km 53.3		

Sub-base material should have a GM > 1.5 and a PI < 10. On this basis the sub-base quality is generally just acceptable.

6.5.4 Roadbase

The roadbase material consists of sand and gravel with a low fines content between km 0 and km 8.1. From km 8.1 to km 53.3 the material is a silty sand and gravel with a PI of 10.

The high moisture contents between km 0 and km 14.3 are measured mainly at sections which are scheduled for raising to facilitate surface drainage (refer Section 8.2).

km 0		
	Sand and gravel A1 - b	GM = 1.8 mc = 7.3%
km 8.1		
	Silty sand and gravel A2 - 4	GM = 1.8 LL = 26 PI = 9 mc = 9.4%
km 14.3		
	Silty sand and gravel A2 - 4	GM = 2.1 LL = 27 PI = 10 mc = 6.1%
km 30.0		
	Silty sand and gravel A2 - 4	GM = 2.2 LL = 28 PI = 10 mc = 3.5%
km 53.3		

A stabilised roadbase should ahve a GM > 1.75 and a PI < 6. On this basis the material between km 8.1 and km 53.2 has generally high PI values.

6.6 Pavement Moisture Conditions

Various methods of predicting in situ moisture conditions are available. By comparing the measured moisture content against the predicted equilibrium value, it is possible to check whether a drainage problem exists.

In the early 1970's Burrow (Ref 13) carried out an extensive survey of pavement conditions in the Transvaal and found that the Equilibrium Moisture Content (EMC) was very close to the Optimum Moisture Content (OMC) for Proctor compaction. The relationship used at NITRR, based on work by Haupt (Ref 14) is:

$$EMC\ (\%) = (1.086 \times OMC) - 3.39$$

An alternative approach by Emery and Paterson (Ref 15) predicts Equilibrium Moisture Content from climate and the percentage of fines in the material, using the relationship:

$$EMC = 0.25P + 0.019\ Im + 1.81$$

where P = Percentage passing 75 micron sieve
Im = Thornthwaite's Moisture Index.

The climate classifications based on Thornthwaite's Moisture Index, Im, are given below.

Moisture Index (IM)	Climate
100 and above	Perhumid
20 to 99.9	Humid
0 to 19.9	Moist subhumid
-19.9 to 0	Dry subhumid
-39.9 to -20	Semiarid
-100 to -40.	Arid

The value adopted for the Moisture Index, Im, was 10, being midway in the range appropriate for a moist subhumid climate.

The equilibrium moisture content has been predicted for each pavement layer and the subgrade using the approach by Emery and Paterson (Ref 15), described above. The

predicted Equilibrium values and in situ moisture contents for the subgrade and pavement layers are shown in Figure 6.4. In general the in situ values are lower than the predicted Equilibrium values, wtih the exception of the sections of road from km 0 - 8.0 and in the vicinity of km 29.0. At these locations the subgrade moisture content reaches peak levels.

The NITRR approach based on OMC has been used to predict the subgrade Equilibrium Moisture Content at two locations, km 1.5 and km 18.2. This approach gives lower values for EMC, as indicated on Figure 6.4.

The prediction of Equilibrium Moisture Content has indicated the variable nature of moisture conditions particularly in the subgrade. The peak values, in general, are found in areas where drainage problems are known to exist (refer section 8.2).

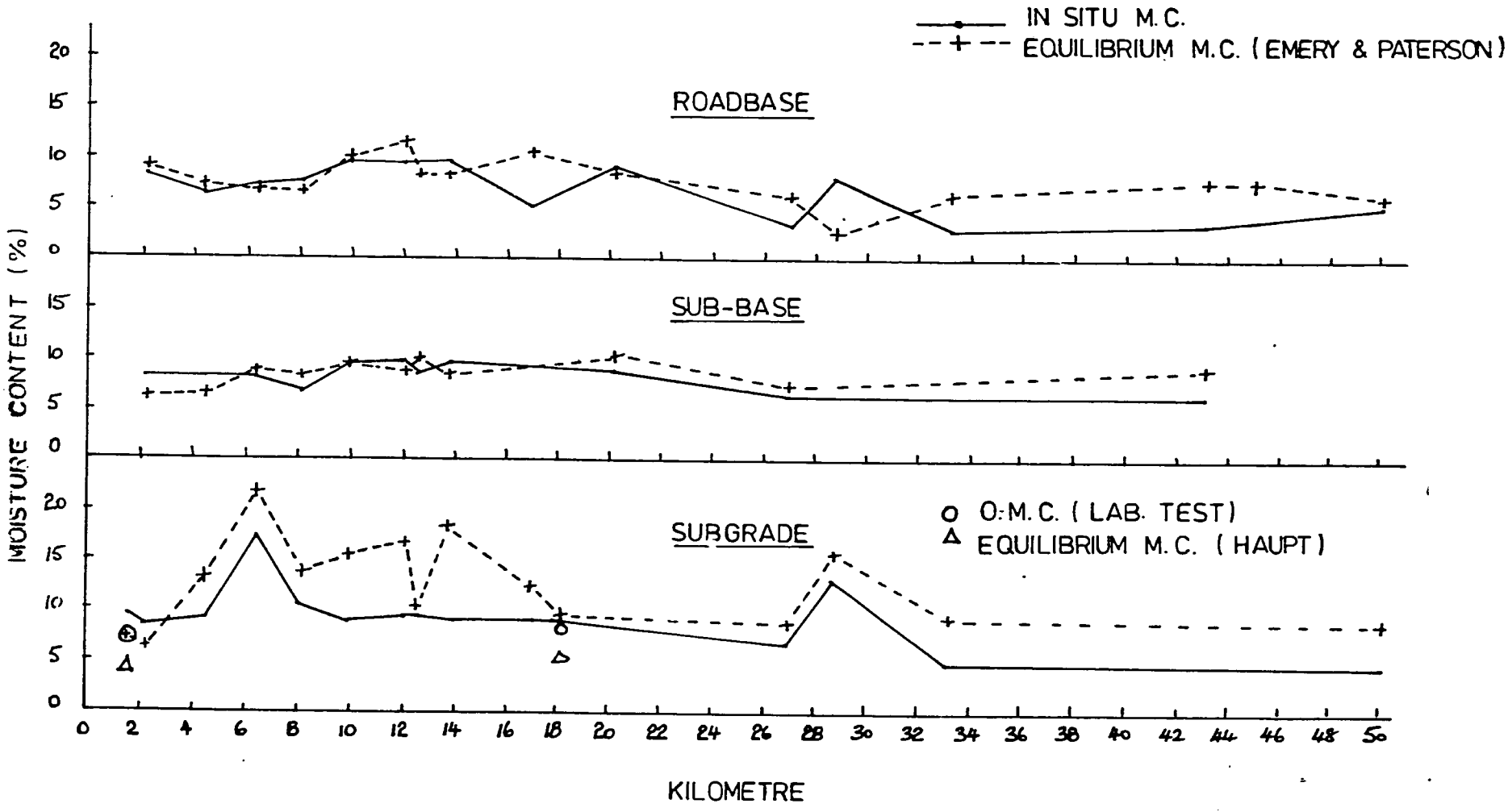


FIGURE 6.4 MOISTURE CONDITIONS

7 ROUGHNESS AND POTHOLE SURVEY

7.1 Existing Data

7.2 Roughness Survey

7.3 Pothole Survey

7. ROUGHNESS A /THOLE SURVEY

7.1 Existing Data

The Kampsax/CarlBro study (Ref 5) included a roughness survey which was carried out using a towed fifth wheel Bump Integrator and also a Vehicle Mounted Bump Integrator (VMBI). As part of that study, a correlation between the two devices was established, which had the form:

$$BI = 17.28 \times BI\ (\text{measures})^{0.67}$$

where BI (measure) was termed the "true roughness" in mm/km

BI was the value measured by the VMBI in mm/km

The results of this former survey were converted to the International Roughness Index (IRI), using a relationship developed by the World Bank, given below:

$$IRI = 0.0032 \times (BI_{32})^{0.89}$$

where BI = Roughness count from fifth wheel towed at 32 kph.

7.2 Roughness Survey

Owing to both mechanical and electrical defects, the towed fifth wheel was not available for use within the data collection study period (Ref 9). However, a measure of the relative roughness of the study road was obtained using a VMBI. The device was fitted to a Volkswagon Combi Van, and the roughness values (expressed in inches/km) were measured in each direction between Lusaka and Kafue Bridge. The apparent roughness value was obtained using the relationship:

$$BI = 17.28 \times (25.4 \times BI_v)^{0.67}$$

where BI is the apparent roughness in mm/km
BIv is the value measured by the VMBI in inches/km

This relationship was based on that derived previously (refer Section 7.1).

The results of the roughness surveys are given in Table 7.1, which shows the values obtained in each direction of travel between Lusaka and Kafue. The average value from each carriageway for each kilometre length is included in Appendix A. From Appendix A it is clear that a broad correlation exists between apparent roughness and deflection.

The values of apparent roughness have been used principally for the economic analysis of different maintenance strategies based on the World Bank Computer model HDM 3. This makes use of the Vehicle Operating Costs (VOC) which vary mainly as a function of the road roughness. It should be noted that, as the vehicle operating cost and thus the whole life cost of a road increase as the road roughness (BI Value) increases it is the aim of all rehabilitation or maintenance projects to reduce road roughness.

The average road roughness on the Kafue Lusaka road ranges from 1500 mm/km to over 3000 mm/km. Forthe level of traffic on this road, the aim of the rehabilitation/periodic maintenance strategies should be to achieve a roughness of the order of 1500 mm/km throughout.

7.3 Pothole Survey

The area of potholes has been measured over each kilometre of the study road between, Lusaka and Chilanga as part of an additional data collection study by John Burrow & Partners (Ref 9)

In order to update the survey carried out in 1985 by Kampsax/Carl Bro (Ref 5), measurements taken at six control sections between Chilanga and Kafue Bridge in May 1989. These measurements confirmed that the area of potholes had not changed significantly since the last survey. Consequently the results of the 1985 survey have been adopted for the road section between Chilanga and Kafue Bridge.

km	Southbound BIv inches/km	Southbound BI mm/km	Northbound BIv inches/km	Northbound BI mm/km	Average BI mm/km
0-1	52	2131	56	2239	2185
1-2	53	2158	60	2345	2252
2-3	61	2371	59	2319	2345
3-4	63	2423	69	2575	2499
4-5	55	2212	60	2345	2279
5-6	38	1727	62	2397	2075
6-7	28	1107	38	1727	1571
7-8	34	1603	37	1696	1650
8-9	21	1161	46	1963	1587
9-10	25	1304	37	1696	1507
10-11	44	1905	59	2319	2117
11-12	24	1269	30	1474	1373
12-13	39	1757	49	2047	1905
13-14	25	1304	57	2266	1817
14-15	82	2891	99	3280	3088
15-16	34	1603	38	1727	1665
16-17	11	1817	40	1787	1802
17-18	87	3008	77	2772	2891
18-19	106	3434	111	3541	3488
19-20	36	1665	75	2723	2226
20-21	43	1876	72	2650	2279
21-22	66	2500	88	3031	2772
22-23	60	2345	82	2891	2625
23-24	44	1905	65	2474	2199
24-25	45	1934	64	2449	2199
25-26	36	1665	62	2397	2047
26-27	55	2212	71	2625	2423
27-28	58	2292	78	2796	2550
28-29	47	1991	57	2266	2131
29-30	55	2212	87	3008	2625
30-31	53	2158	67	2525	2345
31-32	59	2319	75	2723	2525
32-33	53	2158	69	2575	2371
33-34	42	1847	47	1991	1919
34-35	48	2019	50	2075	2047
35-36	50	2075	51	2103	2089
36-37	41	1905	57	2266	2089
37-38	42	1847	56	2239	2047
38-39	47	1991	51	2103	2047
39-40	41	1817	32	1539	1681
40-41	71	2625	79	2820	2723
41-42	44	1905	50	2075	1991
42-43	60	2345	62	2397	2371
43-44	58	2292	69	2575	2436
44-45	50	2075	60	2345	2212
45-46	38	1727	37	1696	1712
46-47	43	1876	53	2158	2019
47-48	41	1817	54	2185	2005
48-49	64	2119	71	2625	2538
49-50	78	2796	76	2747	2772
50-51	71	2625	66	2500	2563
51-52	47	1991	55	2212	2103
52-53	57	2266	52	2131	2199
53-53.3	67	2525	83	2915	2723

Table 7.1 Roughness Survey Results

8. DRAINAGE SURVEY

8.1 Introduction

8. DRAINAGE SURVEY

8.1 Introduction

In order to assess the future drainage requirements for the project it was first necessary to carry out an in-situ investigation of the existing drainage provisions. This has been carried out as thoroughly as possible but some small culverts may have been missed, largely due to growth of vegetation or total burial of the pipes.

It has in general been observed that cross drainage culverts (and not relief culverts) in general have not been over - topped this year, the wettest since 1978 and all are in reasonably good condition structurally. However, the construction of some of these culverts is such that adding lengths to extend the culvert while widening carriageway and shoulders may not be possible and such the culverts may need to be totally replaced. What is also noticeable in several cases is the lack of a proper depth of cover and these culverts are unlikely to stand up to a re-construction programme. These culverts together with those which are at too low a level to function optimaly will need to be removed and relaid or replaced as necessary.

In general it should be noted that preventative maintainance on open drains and culverts has been minimal and this has added to the deterioration of the road surface.

The condition and location of the culverts on the project are listed in Table 8.1. In addition, the section of the road where the carriageway level has to be raised from drainage considerations has also been included in this schedule. It should also be noted that, especially in the Chilanga area, several dams have been constructed for agricultural and other uses and that these structures have a major balancing effect on the actual stream flows across the project road. The extent of the balancing effect is indeterminate at present, but as the existing culverts have proved to be adequate in practice, it is not proposed to increase these in size even though theoretical calculations might indicate otherwise.

CHAINAGE	DIAMETER OR SIZE	TYPE	HEAD WALLS L	HEAD WALLS R	WING WALLS L	WING WALLS R	TO BE REPLACED	SCOUR PROTECTION REQUIRED	COMMENTS
1.32	1 x 300	CONCRETE PIPE	BRICK	BRICK	NIL	NIL	NO	SMALL	LIMITED COVER BUT APPARENTLY IN REASONABLE CONDITION
2.4	1 x 600	CONCRETE PIPE	BRICK	BRICK	BRICK	BRICK	NO	NO	CONCRETE CHANNEL AT INLET AND OUTLET
4.55	1 x 800	STEEL PIPE	NIL	NIL	NIL	NIL	NO	YES	SOFFIT AT ROAD LEVEL
4.75	1 x 900	STEEL PIPE	NIL	BRICK	NIL	NIL	NO	YES	SOFFIT ABOUT 150MM BELOW ROAD LEVEL
5.45	600 x 600	CONCRETE BASE	CONCRETE	CONCRETE	NIL	NIL	YES	YES	CULVERT EQUALISING AND ALMOST SILTED UP
10.3	3 x 600	CONCRETE PIPES	CONCRETE	CONCRETE	NIL	NIL	NO	YES	OUTLET BLOCKED BY TRACTOR TRAFFIC CAUSING SWAMP AREA ON BOTH SIDES
12.5	3 x 900	CONCRETE PIPE	BRICK	BRICK	NIL	NIL	YES	YES	CULVERT TO BE LIFTED, ROAD TO BE RAISED OR MOVED
12.52	3 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	NO	YES	APPEAR TO BE SUBSTITUTED BY 12.50
13.46	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	YES	YES	ROAD TO BE RAISED
13.5	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	YES	YES	ROAD TO BE RAISED
13.89	1 x 600	CONCRETE PIPE	CONCRETE	NIL	NIL	NIL	YES	YES	TO BE ABANDONED
13.9	1 x 600	CONCRETE PIPE	NIL	CONCRETE	NIL	NIL	YES	YES	ABOUT 60% SILTED
14.7	1 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	YES	CULVERT NEEDS TO BE RAISED BY 600 MM
15.09	2.7mx4.3m	CONCRETE BRIDGE	CONCRETE	CONCRETE	YES	YES	NO	NO	ROAD BRIDGE
15.4	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	YES	YES	RAISE CULVERT BY 500MM

TABLE 8.1-PART I - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

15.9	2 x 1.8m	CONCRETE ARCH	CONCRETE	CONCRETE	CONCRETE	CONCRETE	NO	YES	MASS CONCRETE ARCH 1.8M DIA ON 750 UP STAND
16.2	1 x 450	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	YES	TO BE LOWERED BY 300MM
16.5	1 x 450	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	YES	50 MM FRESH WATER PIPE PASSES DIRECT
17.2	2 x 1900	AMCO CONCRETE	CONCRETE	CONCRETE	YES	YES	YES	YES	WING WALLS NOT LONG ENOUGH. CULVERT NEEDS RAISING
17.6	1 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	NO	NO	INTAKE PIPE 900MM DIA 3M DEEP PROVIDED ON LEFT SIDE
17.9	1 x 600	CONCRETE	CONC	CONC	NIL	NIL	NO	YES	SILTED SLIGHTLY @ RHS - SCOUR PROTECTION
18.2	1 x 1100	AMCO CONCRETE	CONCRETE	CONCRETE	NIL	NIL	NO	YES	ADEQUATE
18.7	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	NO	YES	DRAIN TO BE SCOUR PROTECTED ON LHS
19.1	1 x 600	CONCRETE PIPE	MASONRE	MASONRY	NIL	NIL	NO	YES	RAISED ADEQUATELY
19.2	1 x 450	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	NO	YES	DRAINS INTO PROPER FURROW
19.45	2 x 600	CONCRETE PIPE	MASONRY	MASONRY	YES	NIL	NO	YES	NEEDS WING WALL ON RIGHT SIDE
20.1	1x1.7mx1.7m	CONCRETE ARCH	CONCRETE	CONCRETE	YES	YES	NO	YES	DRAINS FROM PROPER LINED CHANNEL ON RIGHT - SCOURED ON LEFT
20.3	1. x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	NO	YES	TO BE PROTECTED AGAINST SCOUR ON LEFT SIDE
20.32	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	NO	YES	PROTECTION REQUIRED BOTH SIDES
20.41	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	YES	YES	SILTED SLIGHTLY
20.51	1 x 1.7 x 1.7	CONCRETE ARCH	CONC	CONC	NIL	NIL	NO	YES	PROTECTION ON LHS

TABLE 8.1-PART II - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

21.4	1 x 600	CONCRETE PIPE	NIL	NIL	NIL	NIL	YES	YES	PIPE EXPOSED AT ENDS NO CONCRETE
21.6	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED RIGHT AND LEFT ROAD TO BE RAISED
21.99	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED RIGHT AND LEFT RAISE ROAD-NO OVER BURDEN
22.4	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED LEFT AND RIGHT RAISE ROAD
22.6	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	NO	YES	DRAINS PROPERLY
23.2	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED RUNS OBLIQUE APPROX. 25M LONG
23.45	2 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	NO	YES	SILTED ON RIGHT SIDE
23.7	2 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	NO	YES	SILTED ON RIGHT SIDE
23.98	1 x 450	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED ON RIGHT SIDE
24.5	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED ON RIGHT SIDE
24.6	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NO		YES	NO	DRAINS WELL
24.9	2 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	NO	REQUIRES RAISING BY 250 MM ROAD IS LOW
25.4	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	NO	HEAVILY SILTED NEEDS RAISING - ROAD VERY LOW
26.4	1 x 450	CONCRETE PIPE	MASON RY	NIL	NIL	NIL	NO	NO	DRAINS PROPERLY
26.8	1 x 1700	CONCRETE ARCH	NIL	NIL	NIL	NIL	NO	NO	SILTATION ON RIGHT SIDE
28.2	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	NO	YES	SILTATION ON RIGHT SIDE
28.75	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	SILTED ON RIGHT SIDE

TABLE 8.1-PART III - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

28.9	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	NO	SILTED ON RIGHT SIDE
29.65	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	NO	NO	SILTED ON RIGHT SIDE
31.8	3 x 600	CONCRETE PIPE	NIL	MASON RY	NIL	NIL	YES	NO	BOTH SIDES SILTED ROAD LOW - LIFT ROAD
32.4	1 x 450	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	BLOCKED COMPLETELY ON HILL
32.7	1 x 2.5m x 7.5m	CONCRETE BRIDGE	CONCRE TE	CONCRE TE	YES	YES	YES	YES	BRIDGE SILTED BOTTOM
34.7	1 x 2.5m x 1.75m	CONCRETE BRIDGE	CONCRE TE	CONCRE TE	YES	YES	NO	YES	FAIR CONDITION
35.25	1 x 1200	CONCRETE ARCH	NIL	NIL	NIL	NIL	YES	NO	SILTED ON BOTTOM
35.55	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	YES	FULLY SILTED CULVERT CAN BE RAISED BY 250MM
36.2	1 x 700	CONCRETE PIPE	CONCRE TE	CONCRE TE	NIL	NIL	NO	NO	RUNS OBLIQUE SILTED ON LEFT SIDE
36.55	1 x 3200	CONCRETE ARMCO	CONCRE TE	CONCRE TE	NIL	NIL	NO	YES	UPSTREAM WING WALL NECESSARY, SILTED ON RIGHT SIDE
37	1 x 100	CONCRETE	CONCRE TE	CONCRE TE	NIL	NIL	YES	NO	HEAVILY SILTED
38.4	2 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	NO	SILTED, TO BE RAISED BY 300 MM
38.55	1 x 600	CONCRETE PIPE	MASON RY	MASON RY	NIL	NIL	YES	NO	FULLY SILTED
39.5	1 x 600	CONCRETE PIPE	MASON RY	CONCRE TE	MASON RY	NIL	NO	YES	SILTED 300 M BELOW ROAD LEVEL - AREA LOW-LYING, LIFT ROAD
39.7	1 x 600	CONCRETE PIPE	CONCRE TE	CONCRE TE	NIL	NIL	YES	NO	SILTED BOTH SIDES
39.8	1 x 3500	CONCRETE ARMCO	MASON RY	NIL	MASON RY	MASON RY	NO	YES	WING WALLS TO BE IMPROVED

TABLE 8.1-PART IV - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

39.9	2 x 600	CONCRETE PIPES	CONCRETE	CONCRETE	NIL	NIL	YES	NO	FULLY SILTED - AREA FLAT
40.1	2 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	NO	FULLY SILTED 100 MM AC PIPES FOR SIGNAL CABLES - LEVEL CROSSING
41.11	2 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	NO	FULLY SILTED - LEVEL CROSSING
40.12	2 x 600	CONCRETE PIPE	CONCRETE	CONCRETE	NIL	NIL	YES	NO	FULLY SILTED
40.8	1100 x 700	BOX CULVERT	CONCRETE	CONCRETE	YES	YES	NO	NO	IN GOOD CONDITION
41.1	1 x 600	CONCRETE PIPE	MASONRY	MASONRY	NIL	NIL	NO	NO	PARTLY SILTED
42.5	2 x 600	CONCRETE PIPE	BRICK	BRICK	NIL	NIL	YES	NO	FULLY SILTED, RAISE CULVERT - LIFT ROAD
42.9	2 x 600	CONCRETE PIPE	BRICK	BRICK	NIL	NIL	NO	NO	PARTLY SILTED
43.1	2 x 600	CONCRETE PIPE	CONC BLOCK	CONC BLOCK	NIL	NIL	NO	NO	PARTLY SILTED LEFT SIDE HEAD WALL BROKEN
43.12	1 x 600	CONCRETE PIPE	CONC	CONC	YES	YES	YES	NO	NEEDS RAISING SLIGHTLY
43.305	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	YES	NO	SILTED - AREA LOW-LYING
43.31	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	YES	NO	PARTLY SILTED
43.7	1 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	NO	NO	PARTLY SILTED RIGHT WING WALL BROKEN
44.15	2 x [illegible] x 1	BOX CULVERTS	CONC	CONC	YES	YES	YES	NO	ONE COMPARTMENT FULLY SILTED OTHER PARTLY LIFT ROAD
44.25	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	YES	NO	SILTED PARTLY - ROAD LOW LIFT ROAD
44.4	2 x 450	CONCRETE PIPE	CONC	CONC	NIL	NIL	YES	NO	FULLY SILTED NEEDS RAISING - LIFT ROAD

TABLE 8.1-PART V - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

46.1	2 x 5.4 x 2	BOX CULVERT	CONC	CONC	YES	YES	NO	YES	WING WALL TO BE EXTENDED
47.98	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	NO	YES	DRAINS PROPERLY - STONE PITCHED APRONS
48.4	1 x 2.5 x 2	BOX CULVERT	CONC	CONC	YES	YES	NO	YES	DRAINS PROPERLY - LARGE ENOUGH
48.9	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	NO	YES	DRAINS PROPERLY
49.6	2 x 600	CONCRETE PIPE	CONC	CONC	NIL	NIL	YES	NO	FULLY SILTED - LIFT ROAD
49.8	1 x 600	CONCRETE	CONC	CONC	NIL	NIL	NO	NO	GOOD WORKING ORDER
50.12	2 x 600	CONCRETE	CONC	CONC	NIL	NIL	NO	YES	SCOUR PROTECTION ON RIGHT SIDE
50.4	2 x 600	CONCRETE	CONC	CONC	NIL	NIL	NO	YES	JUST REQUIRES CLEANING
50.88	1 x 600	CONCRETE	CONC	CONC	NIL	NIL	YES	YES	SILTED - LIFT ROAD
51.5	3.0mx2.5m	BOX CULVERT	CONC	CONC	YES	YES	NO	NO	
52.05	1 x 600	CONCRETE	CONC	CONC	NIL	NIL	YES	YES	FULLY SILTED - LOW LIFT ROAD

TABLE 8.1-PART VI - LUSAKA - KAFUE ROAD - CONDITION OF EXISTING DRAINAGE

9. DRAINAGE DESIGN

9.1 Introduction

9.2 Design Criteria for Storm Water Drainage

9. DRAINAGE DESIGN

9.1. Introduction to the Concept

One of the most important aspects of the design of a road is the provision made for protecting the road from surface water or ground water. If water is allowed to enter the structure of the road, the pavement will be weakened and it will be much more susceptible to damage by traffic. Water can enter the road as a result of rain penetrating the surface or as a result of the infiltration of ground water. The road surface must be constructed with a camber so that it sheds rainwater quickly and the formation of the road must be raised above the level of the local water table to prevent it from absorbing ground water.

Water can also have a harmful effect on shoulders, slopes, ditches and other features. High water velocities can cause erosion which, when severe, can lead to the road link being severed. Alternatively, low velocities in drainage facilities can lead to silt being deposited which, in turn, can lead to a blockage. Blockage in turn often result in further erosion.

A good road drainage system, which is properly maintained, is vital to the successful operation of a road and the following items must all be considered.

Drainage of the Pavement Layers

The road cross section is designed either to shed water completely, or alternatively to permit the egress of any water that may enter it. When impermeable base materials are used, drainage of the base is not necessary. If permeable base materials are used, particular attention must be given to the drainage of the base layers and shoulders, and to the details of the side drains.

Road Drains and Outfalls

The various forms of drains are described below, and the main functions are:-

- To provide a reasonable capacity to accommodate surface water run-off.

- To dispose the collected water to a natural drainage channel or onto the surrounding ground.

- To minimise the hazard to pedestrians and vehicle traffic.

- To ensure that the flow in the drain and at outfall causes minimum erosion.

Side and Mitre Drains

Side and mitre drains are usually situated in open areas and the bottom widths should have the minimum dimension to suit shaping by a grader. These wide drains with gentle batters reduce the velocity of flow and are less prone to erosion and blockages and will require less maintenance.

Lining of Drains

Table 9.1 Shows the generally accepted permissible flow velocity in drains before lining is required for various types of soils based on the Cassagrande classification. The values shown should only be used as a guide and the values used in practice would be less conservative. Lining of drains normally consists of 75 mm thick concrete, or, if local sands are available, sand/cement linings varying from 50 mm to 100 mm, depending on the size of drain, and are used at outlets to culverts and in large drains where the volumes and velocities are high.

Erosion and Grassing

Erosion due to rainfall occurs when the intensity of precipitation is greater than 25 mm per hour. The areas most vulnerable are the shoulders, side drains and verges, and surfaces of cuttings and embankments.

Grassing of the verges and drains assists in stabilising the areas vulnerable to erosion. Grassing could take the form of seeding or planting. Grass should be planted just prior to, or at the beginning of the rainy season, and observed during its growth in order that areas that have failed to root satisfactorily can be replanted.

Cuttings and Embankments

Generally, cuttings and fills are avoided as much as possible, but where cuttings and embankments are required side slopes should be kept as steep as possible for economy. An effort should be made to encourage some form of vegetation to protect the slopes from erosion. In areas where there is the possibility of the flow of water over the face of the cutting, a cut-off drain along the top of the cutting should be constructed. In the case of embankments, it is usual practice to carry the surface run-off from the road to selected points and discharge down the embankments face by means of a lined drain, depending on the extent of the embankment and to protect the edges by the use of kerbs.

Box Culverts

It is difficult to say at what flow, consideration should be given to changing from a pipe to a box culvert, because the decision is governed by the local price of pipes and materials. Generally, where the storm water flow is greater than 12 m3/s, the construction of a box culvert instead of multiple pipe culverts is worth considering. Cost comparisons should be made to compare the two approaches as well as in this case, construction time.

Sub-Soil Drainage

Where high water tables are encountered, the road will be raised to a level where the formation is above the water table, or a sub-soil drainage system can be constructed to lower the water table.

In areas where the finished road level is critical and perched water tables or isolated wet areas are encountered, deep open cut-off drains or subsurface drainage of the pavement must be considered.

9.2 Design Criteria for Storm Water Drainage

The estimation of the required size of storm water drainage structures is not an exact science, however a satisfactory method is described below and has been used in this study.

Rational Method

No local formula has been developed in Zambia for the calculation of storm water run-off and, as such, the rational method has been used extensively in the design of storm water drainage. It has proved to be a reliable method where the catchment area does not exceed 500 ha.

The basic imperial formula was Q = k CAI and the original imperial formula has been converted to

Q = 2,755 x 10-3 CAI, where
Q = storm water run-off in cumecs
C = run-off coefficient
A = catchment area in hectares
I = rainfall intensity in mm/hr.

Design Flood

Smaller Pipe Culverts - a 10 year flood return period has been used for the project. The culverts are able to cope with larger flows over short periods, as the velocity and discharge will increase significantly under surcharge conditions but the possibility of overtopping must not be ruled out.

Box and Large Pipe Culverts - a 20 year flood return period has been used for the project, where the total flow would be in excess of 12m/s.

The choice of return period is as much a matter for managerial consideration as for technical decesion. Selecting a 10 year return period means that the design flood has a probability of 0.1 of being exceeded in any year. In general the risk R of the design flood being exceeded is

$$R = 1 - (1 - \frac{1}{10})L$$

where L is the number of years. However, it must be noted that a design life of greater than 10 years would lead in the end to an uneconomic solution.

Rainfall Intensity

N.P. Snelling a meteological researcher derived a formula for the calculation of rainfall intensity from eight stations in Zimbabwe and three stations in Zambia. This took the metric form of:

$$I = \frac{2\,050 \log (nR) - 3\,000}{t + 20}$$

Where I = rainfall intensity in millimetres per hour

R = mean annual rainfall in millimetres per annum for catchment area

n = design period in years (period over which, on an average, the given storm will occur once only).

t = time of concentration, time in minutes during which the average rainfall intensity is I. This should also include a time of entry (te) which is normally taken as 5 minutes.

Time of Concentration (t)

This is related to the shape of catchment and is given by:

$$t = \frac{(8.707\ L3)\ 0{,}385}{H \times 10^{10}}$$

where t = time of concentration in hours

L = length of critical path of catchment in metres

H = difference in elevation between watershed and design point in metres (all waterfalls are ignored).

The values calculated from this equation is taken as reasonable for catchment areas of area less than 15 km2. Where the area is more than 15 km2 it is proposed that the time of concentration is increased by the factors of

15 < A < 30 km2	Tc = 1.5t	for areas between 15 and 30 sq.km
30 < A	Tc = 2.0t	for areas greater than 30 sq. km

C, Run-Off Coefficient

The value of the coefficient C often gives rise to misunderstanding. If it is interpreted as a runoff coefficient many people would ascribe values close to unity to it in areas where flash floods occur owing to rapid runoff capability of the catchment. However,

this attitude is incorrect because C is no more than a factor (or a parameter) which has to be adjusted, according to physical circumstances, to make the calculated flood agree with the actual magnitude occuring with the required frequency. Values of C for use in varying conditions of slope and vegetation are given in tables in textbooks on drainage and for most conditions these values lie between 0.1 and 0.5, exceeding the latter value only in extreme cases. the values used here are shown below.

High Density Area	0.55
Medium Density Area	0.40
Low Density Area	0.30
Savannah Land	0.30
Farm Land	0.25

Flow in Open Drains

The general Formulae used to calculate the capacity of preselected Vee, Standard & Trapezoidal drain sizes, for both lined and unlined drains are:-

$$Q = VA$$

and

$$V = Cm^{0.5} i^{0.5}$$

Where

Q = Carrying Capacity of drain in m3/s
V = Velocity of flow in m/s
A = Waterway area m2
C = Bazin C $\frac{87}{1 + N .5 m}$

m = Hydraulic mean depth = $\frac{A}{P}$ in metres

i = Invert grade of the open drain
N = Roughness Coefficient
= 1,3 for unlined drains
= 0,3 for concrete lined drains
P = Wetted Perimeter in metres

Calculations

Average rainfall = 836 mms
Design Period required; 10 and 20 years

Time of concentraction t for largest catchment, ignoring waterfalls and lakes.

$$t = \left[\frac{8.707 \times 10\ 000^{3}}{10} \right]^{0.385}$$

$$= 100 \times 10^{0.385}$$

$$= [\ 8.707\ ] \qquad = 2.30 = 78 \text{ min}$$

and adding 5 min for time of entry

$$\Rightarrow I\ 10 = 2050 \log \left(\frac{10 \times 836}{781 + 20 + 5} \right) - 3000$$

$$= 49 \text{ mm/hr}$$

and I 20 = 55 mm/hr

With graded gravel - sand mixture little or no fines	GW	1,8 - 2,1 m/sec
Poorly graded gravel - sand mixture lettle or no fines	GP	2,1 - 2,4 m/sec
Well graded gravel - sand with small clay content	GC	0,6 - 1,5 m/sec
Gravel-sand mixture with excess fines	GF	1,5 - 2,1 m/sec
Well graded sands and gravelly sands, little or no fines	SW	0.3 - 0,6 m/sec
Poorly graded sands with little or no fines	SP	0,3 - 0,6 m/sec
Well graded sands with small clay content	SC	0,6 - 0,9 m/sec
Sand with excess of fines	SF	0,9 - 1,2 m/sec
Clay silts and clays of medium plasticity	CL & CI	0,6 - 0,9 m/sec
Silt clays or sandy clays	MI	0,9 - 1,2 m/sec
Silts, silty fine sands with slight plasticity	ML	0,9 - 1,2 m/sec
Organic silts of low plasticity	OL	0,6 - 0,9 m/sec
Organic clays of medium plasticity	OI	0,6 - 0,9 m/sec
Clays of high plasticity	CH	0,6 - 0,9 m/sec
Highly compressible micaseous or diatomaceous earth	MH	0,9 - 1,5 m/sec
Organic clays of high plasticity	OH	0,6 - 0,9 m/sec

Table 9.1 Maximum Permissible Velocity of Flow in Open Drains

10. PAVEMENT ASSESSMENT

10.1 Introduction

10.2 Deflection Criterion

10.3 Drainage Considerations

10.4 Deflection Analysis

10.5 Mechanistic Analysis

10.6 Overlay Thickness Design

10.7 New Pavement Design

10. PAVEMENT ASSESSMENT

10.1 Introduction

The first stage of formulating rehabilitation proposals is the assessement of the condition of the existing pavement. As discussed in Sections 5 to 8, the pavement has been examined both visually and by non destructive testing methods. Test pits were also excavated and the samples recovered were tested in the laboratory to establish the subgrade and pavement layer properties and condition. The visual assessment established the condition of the pavement and shoulder surfaces and the condition of the drainage system.

No proven method of overlay design exists for tropical conditions. However, an assessement of the likely performance of a bituminous overlay can be made by reference to studies such as that carried out by the United Kingdom Transport and Road Research Laboratory (TRRL) in Kenya (Ref 16). Their study describes the performance of 60 experimental bituminous overlays varying from 30 to 170 mm thickness over a period of some ten years. It was found that the principal mode of failure was cracking which began at the top surface and propagated downwards through the overlay. The experience of the TRRL Overseas Unit is that cracking of this form is by far the most common mode of failure in the tropics, traditional fatigue failure being rare and occurring on extremely underdesigned pavements.The occurrence of cracking which starts at the pavement surface is also common in South Africa (Ref 17), and has also been reported in the United Kingdom (Ref 18), where it has been noted that on sites with stabilised bases, extra movement of the surface in the vicinity of existing shrinkage cracks in the stabilised base can cause reflective cracking which propagates from the top surface downwards.

The explanation offered for this type of cracking in tropical environments (Ref 16) is one of age hardening of the bitumen in the mix with the top few millimetres hardening much more, giving a very weak surface skin which easily fractures under stress. It is suggested (Ref 16) that the most effective method of reducing the cracking problem is to place a bitumen rich coating (eg surface dressing) on top of the overlay. This effectively protects the overlay from the harsh environmental conditions which lead to age hardening.The coating itself will of course need to be replaced periodically.

10.2 Deflection Criterion

In the Lusaka - Kafue study the overlay design is based on mechanistic methods discussed in Section 10.5. This approach requires a design criterion, typically a deflection - life relationship, in order to estimate the design life from a known deflection level after overlaying. An attempt has been made to derive such a relationship for the Lusaka - Kafue Road. The basic approach is to classify the pavement condition for each point at which deflection is measured, as sound, critical or failed, on the basis of observed cracking intensity. The average critical condition is then used in conjunction with historical traffic data to establish points on the deflection life relationship. On the study road, axle equivalence factors (number of equivalent standard axles per commercial vehicle) are available for the Lusaka bound traffic lane (as reported by Kampsax/Carl Bro). The cumulative number of standard axles over the 25 year life of the road has been estimated based on the 1989 traffic levels (using zero growth rate). Details are given in Table 10.1. The condition classification is based on intensity of cracking alone, as defined in Table 10.2.

The 'critical' points have been plotted on a deflection vs. life chart taken from TRRL Laboratory Report 833 (Ref 19), as shown on Figure 10.1. It is clear that the points lie on a line parallel to the TRRL 'critical condition' line.

However, it must be noted that with the data produced from the Lusaka - Kafue Road surveys, the demarcation between sound, critical and failed deflection levels is not well defined. Furthermore, the cumulative traffic levels on the section between km 0 and km 3.9 are based on a life of 25 years, when in fact this road was overlaid some two years ago. However, it is considered that the thickness of overlay was insufficient to make this section "as new" at that time, so the assumption of a 25 year life to give current condition levels is a better estimate than assuming a two year life (2 msa cumulative traffic) to give current condition. Clearly there is some uncertainty surrounding the location of the "critical condition" on the deflection life plot. Consequently the deflection life relationship adopted for this study is based on the 50% probability curve

Km	Cumulative standard axles to 1989	Sound		Critical		Failed	
		Mean	SD	Mean	SD	Mean	SD
0 - 3.9	27.4	39	8.4	50	9.6	38	3.5
3.9 - 15.1	14.8	69	24	63	18	62	22

Note: SD = Standard Deviation (Ref to section 10.4)
Mean = Mean Deflection

Table 10.1 Cumulative traffic up to 1989

Condition	Cracking Index m/sq.m
Sound	0
Critical	1 2
Failed	3 4 5

Table 10.2 Classification of pavement condition

	MODEL A			MODEL B			MODEL C		
	H (mm)	Mr (MN/m2)	V	H (mm)	Mr (MN/m2)	V	H (mm)	Mr (MN/m2)	V
Road base	155	varies	0.35	150	varies	0.35	110	varies	0.35
Sub-base	110	120	0.35	150	160	0.35	160	240	0.35
Subgrade	-	60	0.35	-	80	0.35	-	120	0.35
Subgrade CBR	8%			12%			20%		

Loading details: Contact pressure 590 kN/m2) Wheel load 31 kN
Radius of loaded area 130 mm)

Note: H = Layer thickness Mr = Resilient Modules V = Poisons Ratio

Table 10.3 Details of Analytical Pavement Models

presented in Figure 10.1. As the deflection over the life of the pavement is not uniform, a simple transformation (Figure 10.2) has been applied to give a direct relationship between early life deflection and expected life (msa) which is defined by the relationship:

$$N = \frac{262100}{d^{2.69}}$$

Where N = Cumulative traffic (msa) to reach a defined failure condition.

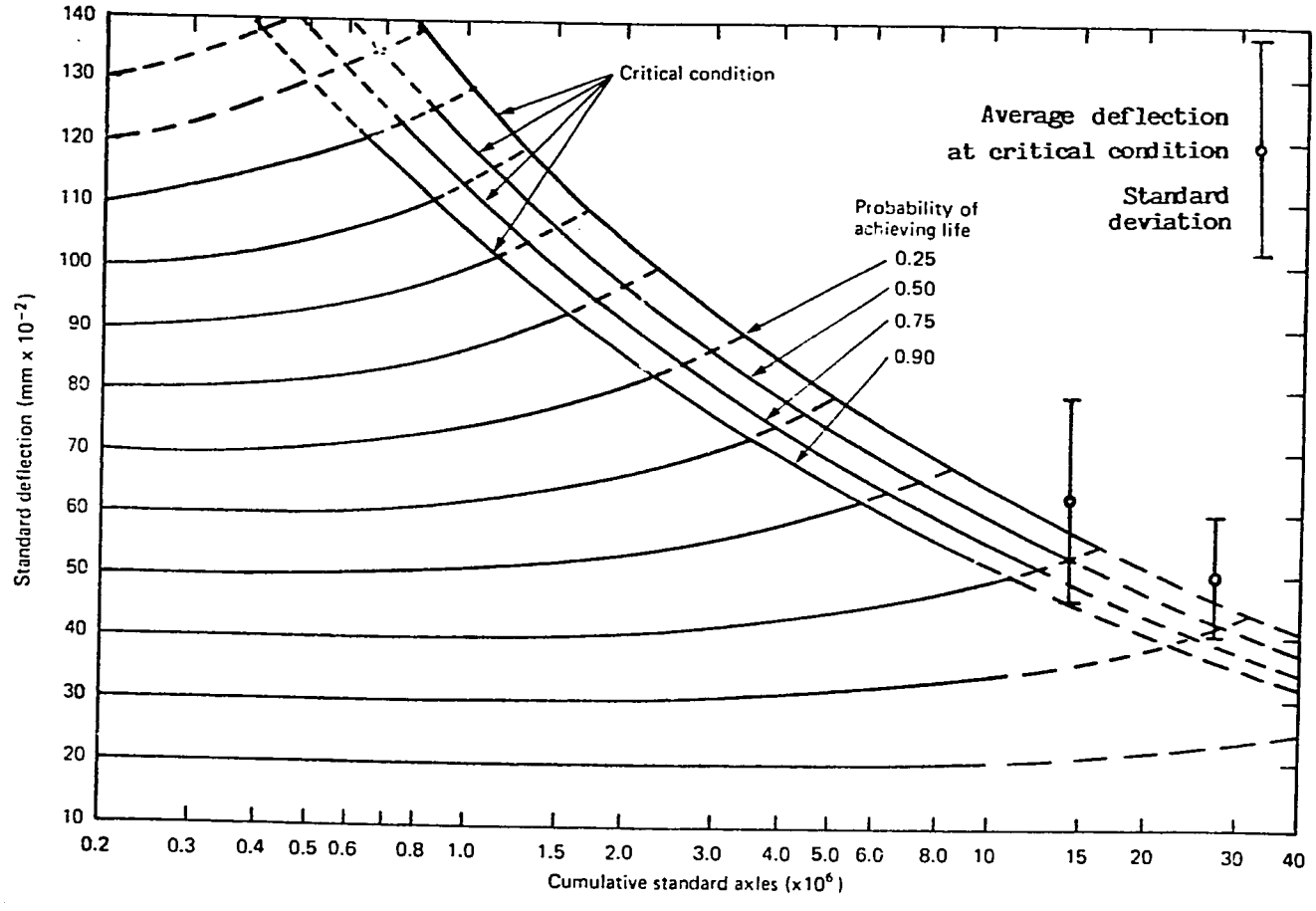
Where d = Early life deflection (mm/100) measured under a 31 kN wheel load moving at creep speed.

This relationship is shown in Figure 10.3

10.3 Drainage Considerations

As discussed in Section 8, one of the most important aspects of the design of a road is the provision made for protecting the road from both surface water and ground water. This is provided for by ensuring that the road in general is above the adjacent ground level, by ensuring that the pavement is constructed with sufficient camber so that it sheds rainwater quickly and by providing an adequate drainage system to disperse the collected water away from the road.

At several locations on this road the level of the road requires raising to provide adequate surface drainage. These sections are listed in Section 8.2. Consequently the deflection analysis has not been carried out for these sections of road as it is intended that the pavement will be reconstructed. However, the extent of the pavement reconstruction at each location has been based on local deflection levels, which in general are relatively high at the sections requiring raising. The high deflection measurements also confirm that a problem exists at these locations.



RELATION BETWEEN STANDARD DEFLECTION AND LIFE FOR PAVEMENTS WITH NON-CEMENTING GRANULAR ROAD BASES

Figure 10.1 Typical TRRL Deflection - Performance Chart.

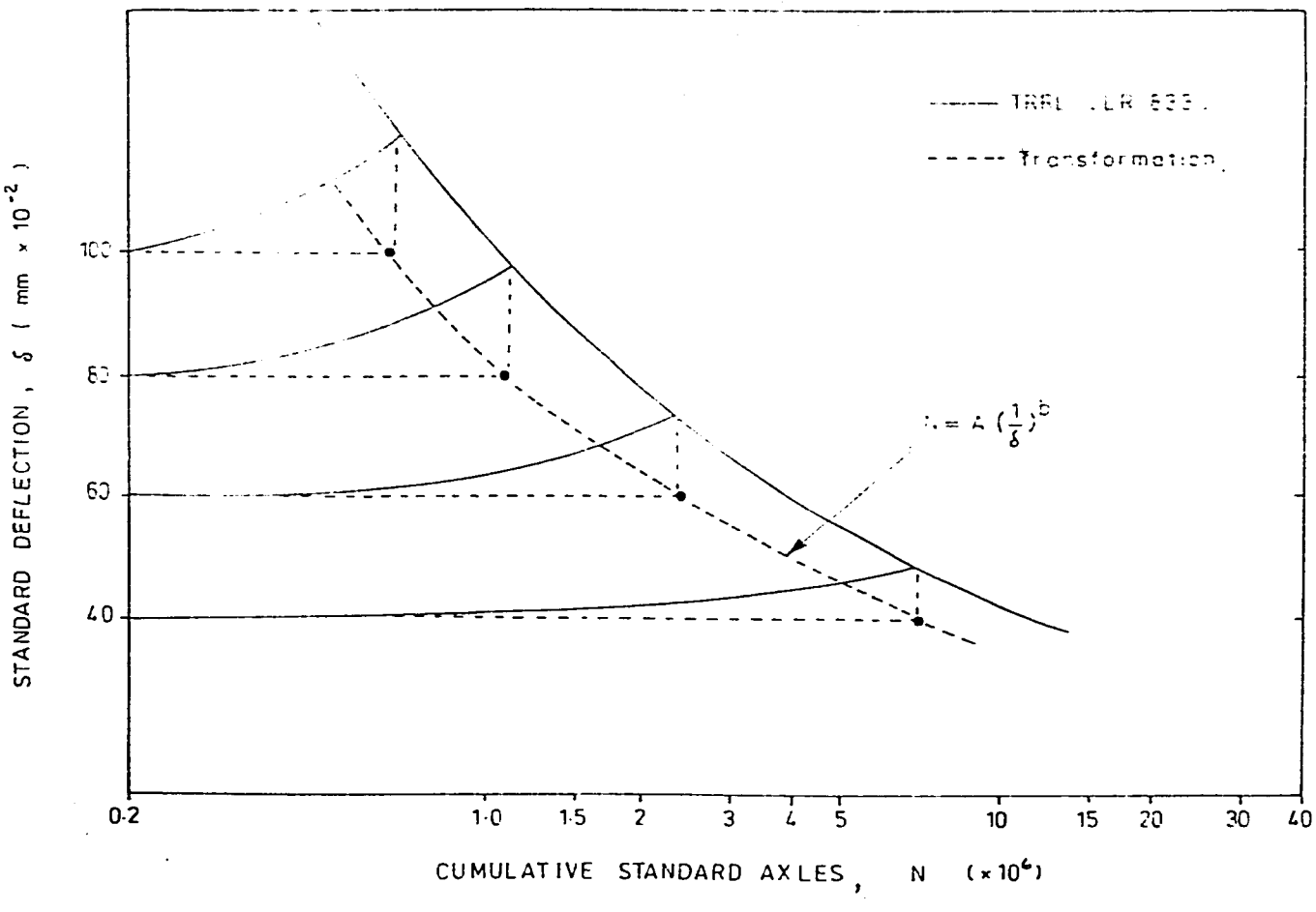


Figure 10.2 Transformation to give Deflection - Life Relationship

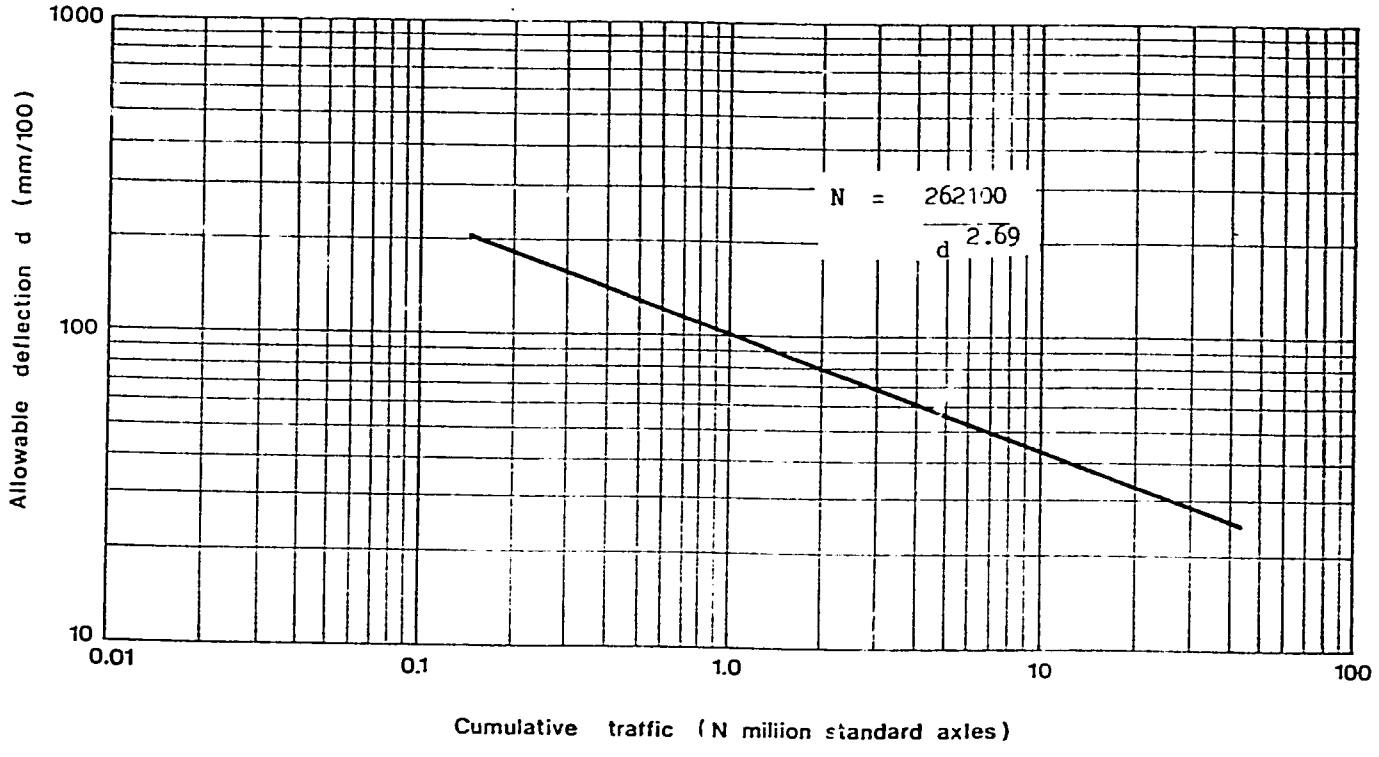


Figure 10.3 Deflection - Life Relationship

10.4 Deflection Analysis

The pavement deflection data for the study road has been collected in two phases. In late 1985, Kampax/CarlBro (Ref 5) collected data for the section between Chilanga Police Station and Kafue Bridge (km 14.3 - km 53.3). In May 1989, John Burrow & Partners carried out the field data collection(Ref 9) for the remaining section between Lusaka and Chilanga (km 0 - km 14.3), and in addition collected data at several control sections (between km 14.3 and km 53.3) in order to update the information collected by Kampax/CarlBro in 1985. Details of the surveys carried out at the control sections are given in Section 5.4.

The design of the pavement strengthening overlays has been based primarily on the measured Benkelman Beam deflection, updated where necessary to appropriate 1989 and 1991 levels. The economic analysis was based on 1989 deflection levels while the final overlay design was based on deflection levels expected in 1991, the date when the rehabilitation works are programmed to commence. Additional data on existing layer thicknesses and subgrade strengths have been deduced from the test pits and DCP test results.

A design deflection has been deduced for each successive length of road having a uniform level of deflection. The design value has been taken at the 95th percentile, given by:

Design deflection = Mean + (1.645 x Standard deviation)

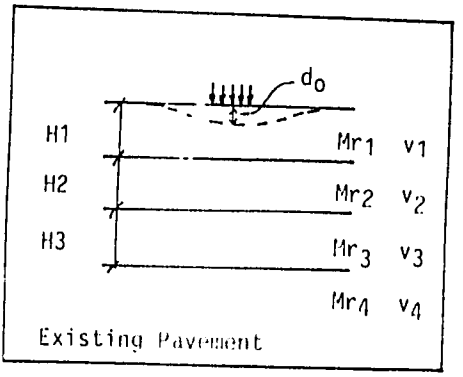
The 1989 design deflections are indicated on the deflection profiles contained in Appendix A. The growth factors used to project these deflections to 1991 levels are given in Section 5.4.

10.5 Mechanistic Analysis

The overlay thickness design has been based on mechanistic analysis in preference to the direct use of overlay charts (Ref 19) derived under different environmental and material conditions. The mechanistic approach to overlay design is summarised in Figure 10.4 and is based on the micro-computer elastic analysis program PAN (Ref 20). A typical pavement model representing the existing construction is shown in Figure 10.5.

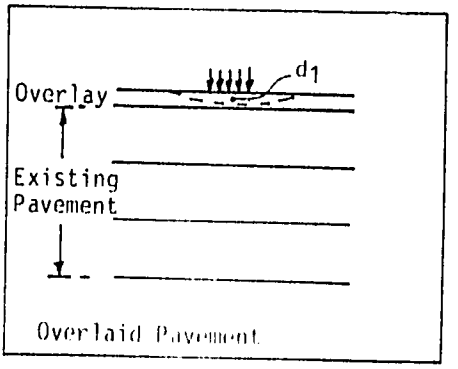
Define Pavement Model

Back Analyse to obtain Mr_1 based on deflection before overlaying (d_o)

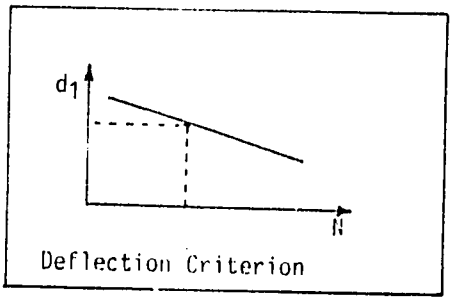


Apply overlay thickness as extra layer.

Analyse to obtain d_1 (new early life deflection) for different overlay thicknesses.



Predict Design Life N for different overlay thicknesses using deflection criterion.



Iterate for various values of d_o

Plot Overlay Thickness vs Design Life.

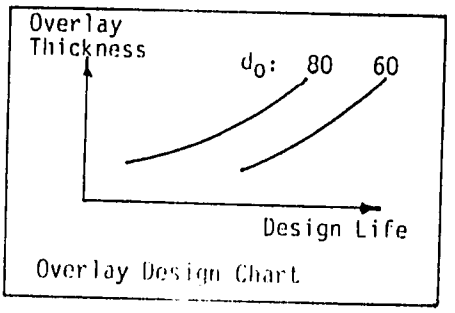
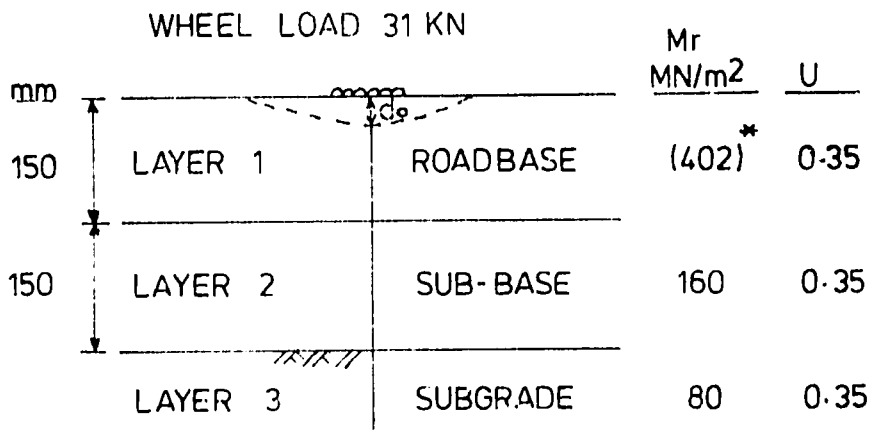


Figure 10.4 Design Methodology for Overlays



* d_o = 80 mm/100

FIGURE 10·5 EXISTING PAVEMENT CONSTRUCTION-MODEL B

The existing construction is represented by three layers, the roadbase, sub-base and subgrade. The existing surfacing layer (generally 25 - 35 mm thick) is not included in the model as a layer of this thickness contributes very little structural strength to the overall pavement structure, acting merely as a protective seal coat. The existing construction is defined in terms of layer thicknesses and material properties represented by the Resilient Modulus (Mr) and Poisson's Ratio (v). Values for Resilient Modulus are assigned to each layer as follows:

<u>Subgrade</u> $Mr3\ (MN/m2) = 17.6\ (CBR)^{0.64}$

where CBR = in situ CBR (%)

Maximum value 120 MN/m2, typical of insitu CBR 20% or greater.

This relationship is based on U.K. experience (Ref 21), and gives a more conservative estimate of the resilient modulus than the Shell relationship (Ref 22) given by:

$Mr3\ (MN/m2) = 10\ (CBR)$

The two relationships are shown in Figure 10.6

<u>Sub-base</u> $Mr2\ (MN/m2) = 2\ Mr3$
Maximum value 240 MN/m2
This relationship is based on the findings of Shell researchers (Ref 22).

<u>Roadbase</u> The resilient modulus for the roadbase layer is deduced by a back analysis procedure in the analysis program PAN. Simply, if all material properties are known except Mr1, then this value is found based on the measured deflection under a known wheel load. Typical values for a lightly cemented roadbase layer are in the range 400 to 1000 MN/m2.

For each layer, the value assigned to Poisson's Ratio was 0.35.

The reduction in surface (Benkelman Beam) deflection achieved by the addition of a bituminous overlay is predicted by the addition of an extra layer to the pavement model. The resilient modulus assigned to this layer was 2500 MN/m2, respresenting a vehicle speed of 3 kph, corresponding to the creep speed of the Benkelman Beam vehicle. The design life of the overlaid pavement is estimated using the deflection - life relationship described in Section 10.2.

By applying increasing thicknesses of overlay, the corresponding deflection and associated life of the overlaid pavement can be deduced, and an overlay chart produced relating design life to overlay thickness. Different design deflections (deflection before overlaying) are characterised by variations in the resilient modulus of the roadbase layer, and thus a family of curves can be produced for a range of design deflections.

10.6 Overlay Thickness Design

Based on analysis of the material laboratory test results and DCP test results, the study road can be represented by three different pavement models (A, B and C), based primarily on three values of design CBR, 8%, 12% and 20%. A sensitivity study has indicated that the predicted overlay thickness is not particularly sensitive to variations in existing layer thicknesses, but rather to variations in material properties. Details of the three models are given in Table 10.3.

Using the approach outlined in Section 10.4, design deflections have been assigned to successive lengths of the study road. Overlay requirements for three values of design life (10, 15 and 20 years) have been predicted using the mechanistic approach described in Section 10.5.

For each of the three pavement models, a corresponding design chart has been produced which relates design life after overlaying to applied overlay thickness. The derivation of a typical overlay chart is summarised in Table 10.4, and the design charts (A,B and C) are presented in Figures 10.7 to 10.9.

		Layer	H (mm)	M (MN/m²)	U
Overlay		1	Varies	2500	0.35
Existing	Roadbase	2	150	Mr_2	0.35
	Sub-base	3	150	160	0.35
	Subgrade	4	-	80	0.35

Loading details: p = 590 kN/m² Wheel load 31 kN
a = 130 mm

d_o = 60 mm/100
Mr_2 = 945 MN/m²

Overlay (mm)	40	70	100	140
d_1 (mm/100)	51	44	39	33
N (msa)	6.7	9.9	13.8	21.6

d_o = 80 mm/100
Mr_2 = 402 MN/m²

Overlay (mm)	40	70	100	140
d_1 (mm/100)	65	54	46	38
N (msa)	3.5	5.7	8.8	14.8

d_o = 100 mm/100
Mr_2 = 207 MN/m²

Overlay (mm)	40	70	100	140
d_1 (mm/100)	80	64	53	43
N (msa)	2.0	3.6	6.0	10.6

d_o = 120 mm/100
Mr_2 = 121 MN/m²

Overlay (mm)	40	70	100	140
d_1 (mm/100)	97	73	59	46
N (msa)	1.2	2.5	4.5	8.8

d_o = deflection before overlay

d_1 = deflection after overlay

Mr_2 = Resilient Modulus of Roadbase layer, deduced by Back Analysis

H = LAYER THICKNESS
M = RESILIENT MODULUS
V = POISSONS RATIO

Table 10.4 Derivation of Overlay Design Chart B

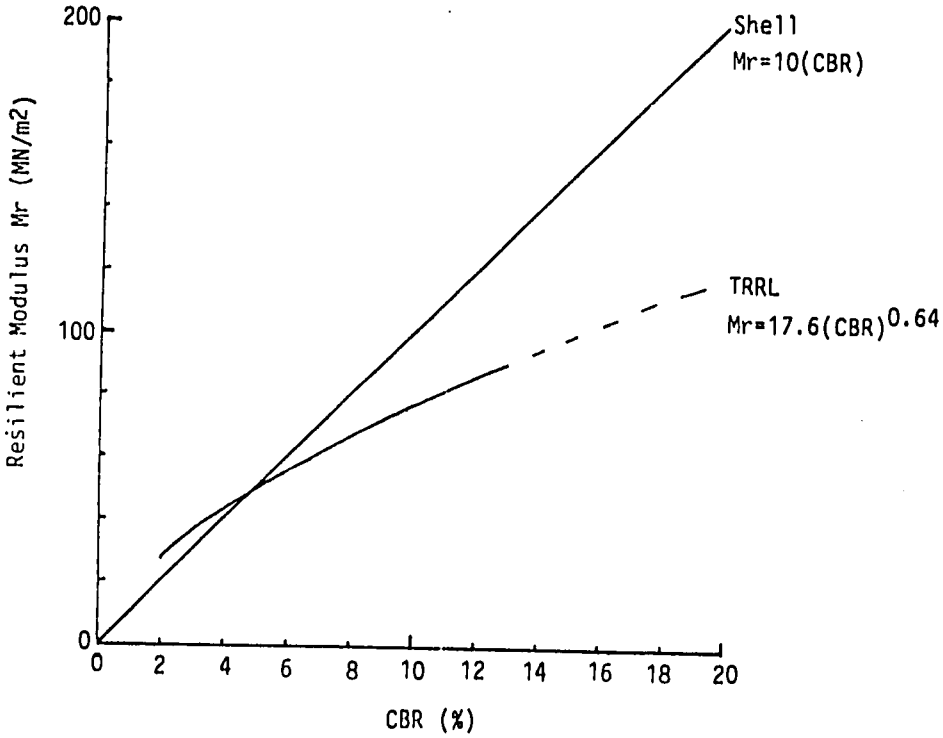


Figure 10.6 Resilient Modulus - CBR Relationships

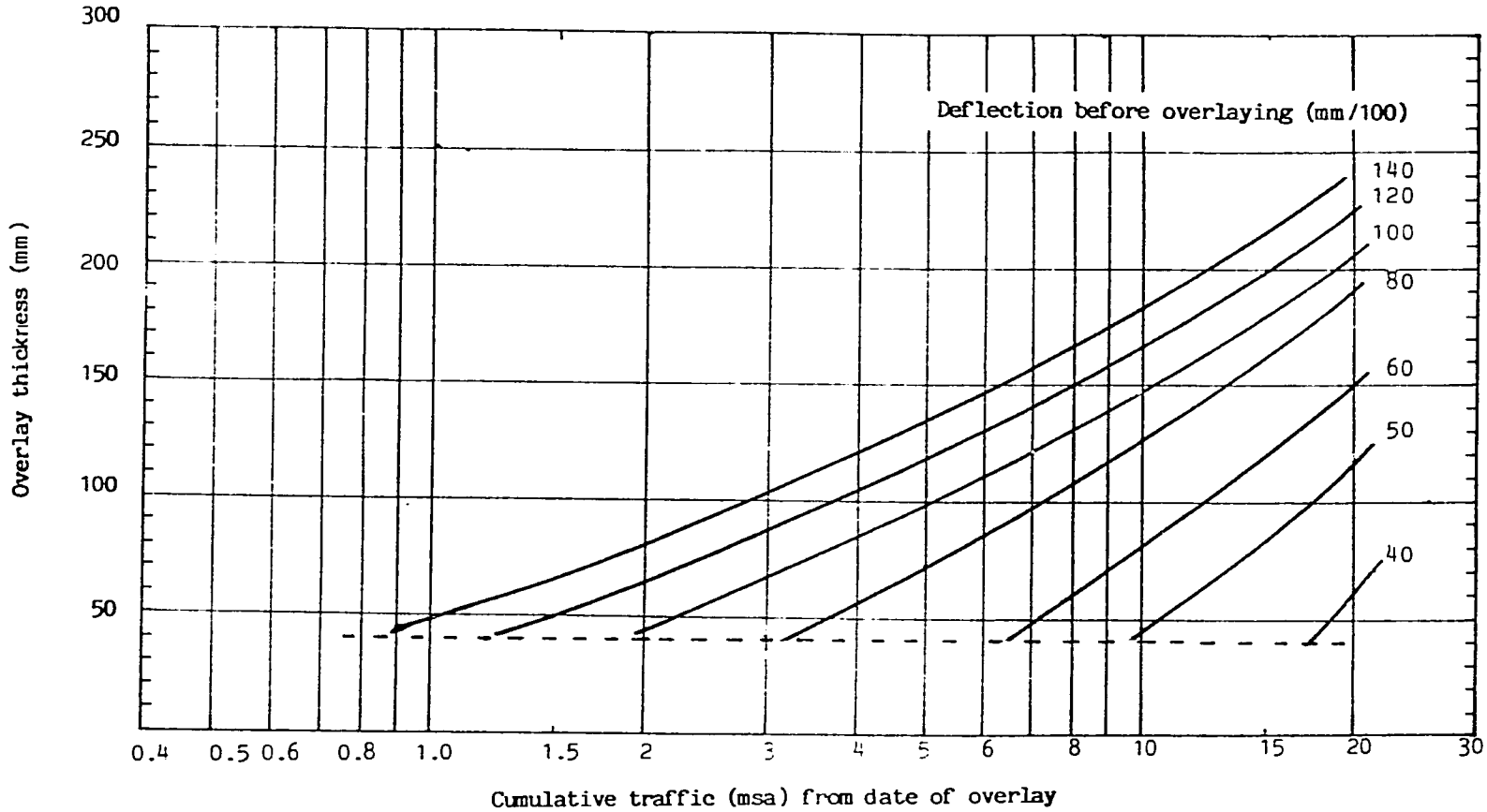


Fig 10.7 Overlay Design Chart A (Subgrade CBR 8%)

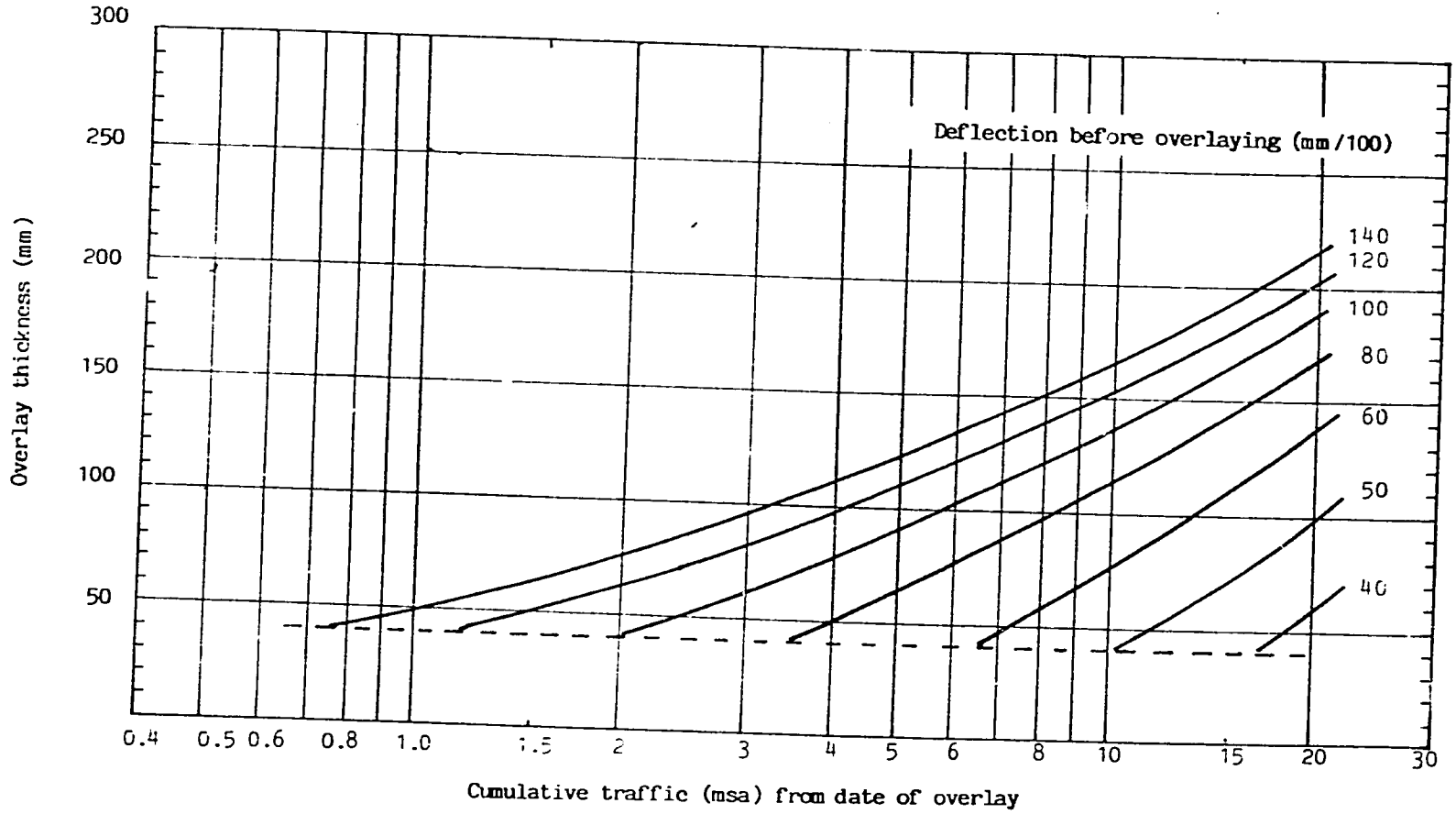


Fig 10.8 Overlay Design Chart B (Subgrade CBR 12%)

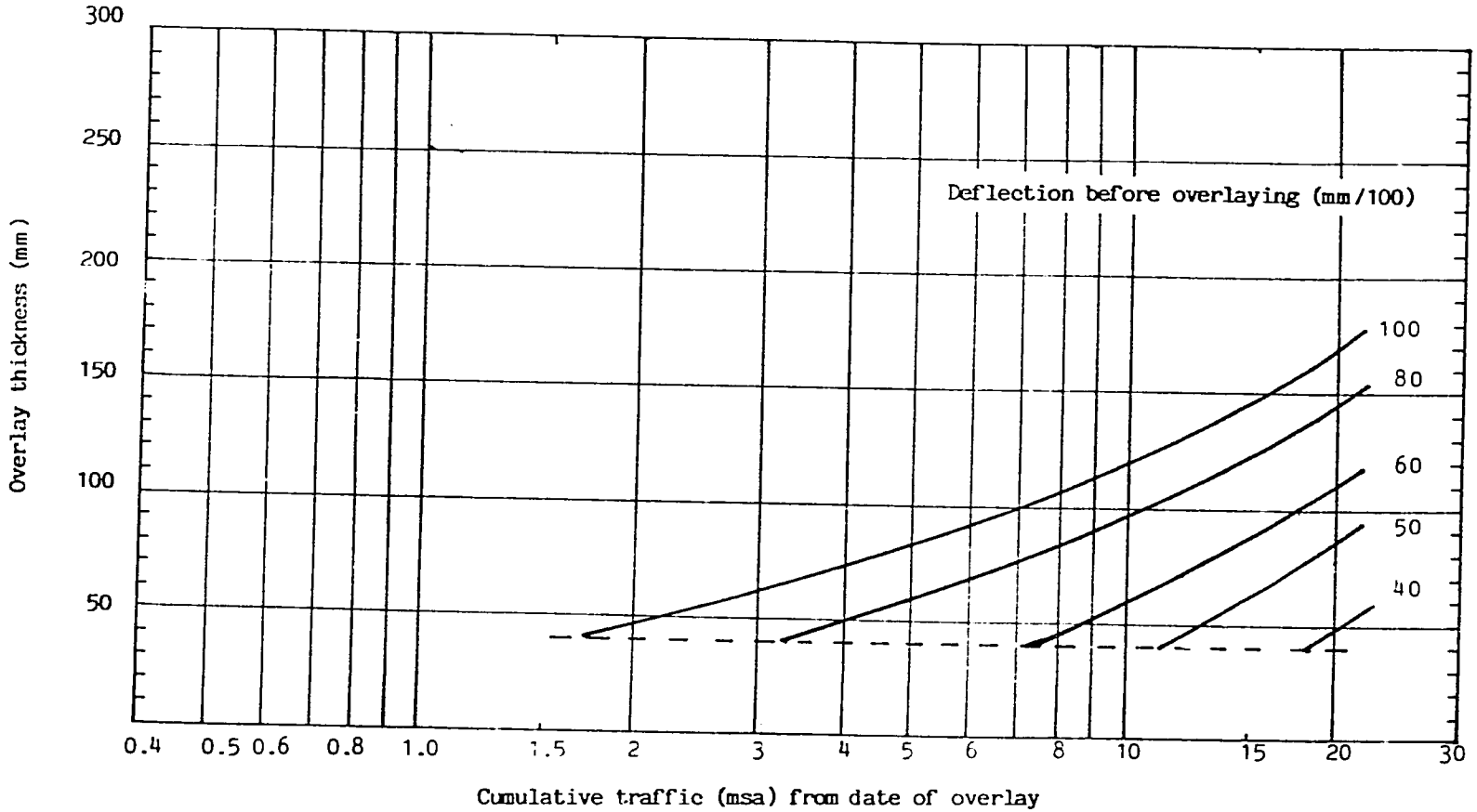


Fig 10.9 Overlay Design Chart C (Subgrade CBR 20%)

The predicted overlay requirements based on 1989 deflection levels are given in Table 10.5. These values have been used for the economic evaluation of different maintenance strategies discussed in section 11. As the rehabilitation works are programmed to commence in 1991, the final overlay thickness design is based on deflection levels for that year (using growth factors given in Table 5.2), and a design life of 10 years. Details are given in Table 10.6. Reconstruction has been recommended where the final overlay thickness was found to exceed 120mm, based on economic considerations.

The estimation of overlay requirements based on DCP Structure Number is described in Section 6.4. However, this approach does not give good correlation with the estimated overlay requirements based on deflection analysis. It is considered that the DCP approach (as used in Section 6.4) gives only a general indication of overlay needs since the DCP tests were carried out at intervals of 2km or more. The adopted overlay design approach is that based on deflections measured at intervals of 100 - 200m.

10.7 New Pavement Design

At those sections requiring reconstruction, it is envisaged that the existing roadbase layer will form the subgrade for the new construction (refer Section 12.4). The in situ CBR value of the existing roadbase, as determined from the DCP tests (refer Section 6.4) varies from 40% to 100%. At those sections where the road level requires raising from drainage considerations, the in situ CBR of the new subgrade (selected fill) is taken to be at least 15%.

The new pavement design is based on the recommendations of Road Note 31 (Ref 23) and TRH4 (Ref 24). The design is based on a minimum subgrade CBR of 15% and a life of 10 years (refer Table 4.4).

The new pavement for the dual carriageway section comprises 300mm cemented natural gravel sub base, 150mm crushed stone roadbase and 60mm DBM surfacing with a double surface treatment. Details are given in Figure 10.10.

A common design is indicated (Figure 10.10) for the Road Class 1A and 1B sections, comprising 300mm natural gravel sub base and 150mm crushed stone roadbase with a double surface treatment.

km	1989 Design Deflection (mm/100)	Design CBR %	Overlay (mm)		
			10 yrs	15 yrs	20 yrs
5.8 - 6.7	83	20	80	110	130
7.1 - 7.5	93	8	110	150	180
7.8 - 8.8	90	8	110	150	180
9.1 - 10.1	61	12	40	80	120
10.8 - 12.1	70	12	60	100	130
12.7 - 13.3	94	20	90	120	140
13.6 - 14.1	104	20	100	130	150
14.1 - 14.6	91	20	100	130	160
14.8 - 15.1	91	20	100	130	160
15.2 - 15.3	113	20	90	110	130
15.5 - 16.8	113	20	90	110	130
17.3 - 18.1	122	20	100	120	140
18.5 - 21.5	124	20	100	120	140
22.1 - 22.3	132	8	120	150	170
22.5 - 23.6	132	8	120	150	170
23.6 - 24.8	83	8	60	100	130
25.0 - 25.3	83	8	60	100	130
25.5 - 26.5	83	8	60	100	130
27.0 - 27.6	165	8	R	R	R
27.6 - 28.2	101	8	90	120	150
28.2 - 31.7	83	8	60	100	130
32.3 - 35.45	54	20	P+S	P+S	40
35.65- 39.4	54	20	P+S	P+S	40
39.6 - 42.4	54	20	P+S	P+S	40
42.6 - 44.05	87	20	70	100	120
44.5 - 49.5	110	20	70	90	110
49.7 - 50.0	110	20	70	90	110
50.2 - 51.95	110	20	70	90	110
52.15- 53.3	110	20	70	90	110

Design traffic (10 years / 15 years /20 years)

km 0 - 3.9	:	12.3/19.7/28.0 msa
3.9 - 15.1	:	06.6/10.6/15.1 msa
15.1 - 43.9	:	04.1/06.6/09.3 msa
43.9 - 53.3	:	03.3/05.3/07.6 msa

Note: R = Reconstruct P+S = Patch and Seal

Table 10.5 Predicted overlay requirements based on 1989 deflection levels

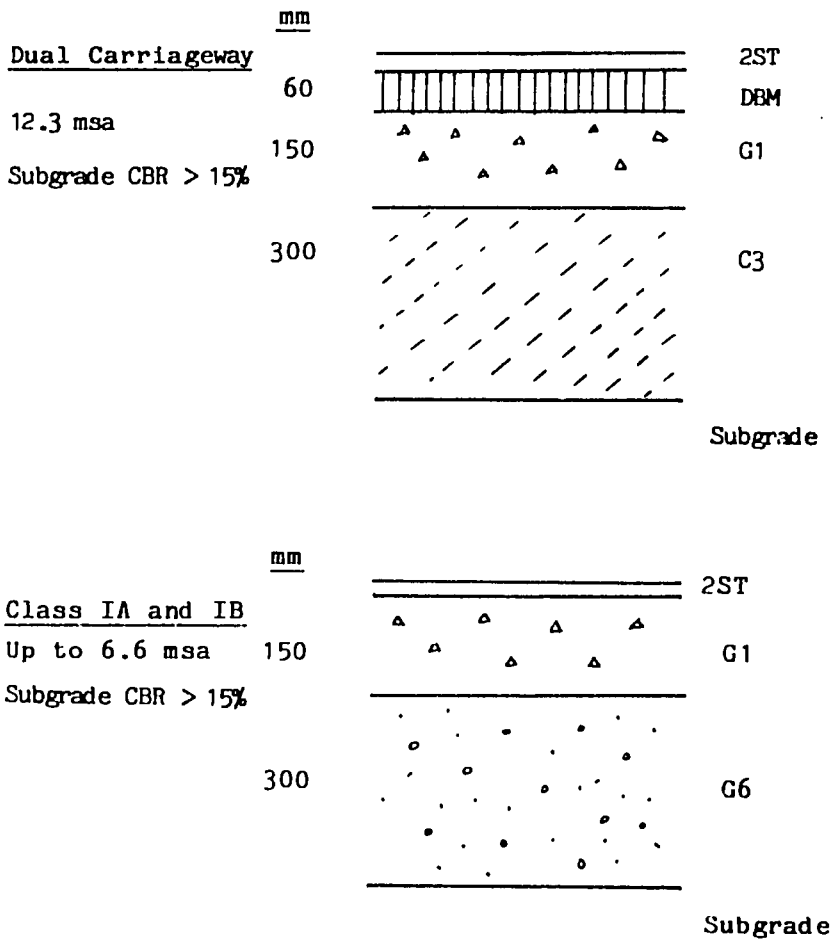
km		1991 Design Deflection (mm/100)	Design CBR %	Overlays (mm) 10 yrs
5.8	6.7	95	20	90
7.1	7.5	106	8	R
7.8	8.8	103	8	R
9.1	10.1	70	12	70
10.8	12.1	80	12	80
12.7	13.3	107	20	R
13.6	14.1	119	20	R
14.1	14.6	104	20	100
14.8	15.1	104	20	100
15.2	15.3	129	20	100
15.5	16.8	129	20	100
17.3	18.1	139	20	R
18.5	21.5	141	20	R
22.1	22.3	150	8	R
22.5	23.6	150	8	R
23.6	24.8	100	8	90
25.0	25.3	100	8	90
25.5	26.5	100	8	90
27.0	27.6	200	8	R
27.6	28.2	122	8	110
28.2	31.7	100	8	90
32.3	35.45	59	20	PtS
35.65	39.4	59	20	PtS
39.6	42.4	59	20	PtS
42.6	44.05	95	20	70
44.5	49.5	176	20	R
49.7	50.0	176	20	R
50.2	51.95	176	20	R
52.15	53.3	176	20	R

Design traffic (10 years)

km			
0	3.9	:	12.3 msa
3.9	15.1	:	6.6 msa
15.1	43.9	:	4.1 msa
43.9	53.3	:	3.3 msa

Note: R = Reconstruct when overlay exceeds 120 mm
PtS = Patch and Seal

Table 10.6 Predicted overlay requirements based on 1991 deflection levels



Legend

2 ST	Double Surface Treatment
DBM	Dense Bitumen Macadam
C3	Cemented Natural Gravel
G1	Graded Crushed Stone
G6	Natural Gravel

Figure 10.10 New Pavement Construction

11. ECONOMIC ANALYSIS

11.1 Background

11.2 Specification of Alternative Option

11.3 Output of Economic Analysis

11.4 Recommendation

11 ECONOMIC ANALYSIS

11.1 Background

In order to test the validity of the design basis proposed, a simplified comparative economic analysis has been carried out using HDM III, the World Bank Highway Design and Maintenance Standards Model.

Within the time constraints of the study it was not possible to calibrate the deterioration models and vehicle operating cost relationships within HDM - III (as had originally been anticipated), nor was it possible to use the revised vehicle operating cost relationship derived by Carl Bro-Kampsax in their study. However this is unlikely to affect the validity of the comparative study of the economics, though the absolute value of the results may be questioned.

HDM III is used principally to compare the whole life cost of the road for alternative rehabilitation design life periods. Design lives of 10, 15 and 20 years has been examined. The different design lives were compared with a "do-minimum" case to estimate the likely economic return on invested capital. In addition two low cost improvements were also investigated to test the effect of periodic maintenance below the standard of full rehabilitation, in the event that sufficient finance for full rehabilitation cannot be allocated.

Two, one kilometre sample lengths of road were investigated representing average conditions (before and after improvement) between Makeni and Chilanga, and between Chilanga and Kafue. No analysis was carried out between Lusaka and Makeni, this section being scheduled for complete reconstruction to dual carriageway standard using a pavement design that does not significantly change between the 10, 15 and 20 year designs.

11.2 Specification of Alternative Options

The two typical one-kilometre road lengths were analysed as follows.

Option 1 Do minimum case - Existing road and existing condition + routine maintenance + 100% pothole patching.

Option 2 Periodic Maintenance Option A - Existing road and existing condition + routine maintenance + 100% pothole patching + surface dressing whenever wide cracking exceeds 15% of pavement area.

Option 3 Periodic Maintenance Option B - Existing roads and existing conditions + routine maintenance + 100% pothole patching + surface dressing whenever wide cracking exceeds 15% of pavement area + 50 mm overlay whenever roughness exceeds 2500 mm/km.

Option 4 10 year Design Option - Road widened to Class IA and IB standard (as appropriate) and strengthened to accommodate the 10 year design traffic with 90 mm overlays for the Makeni - Chilanga section and 80 mm overlays for the Chilanga - Kafue section and 50 mm thick overlays at 10 year intervals thereafter.

Option 5 15 year Design Option - As above but strengthened to accommodate the 15 year design traffic with 120 mm overalys for the Makeni - Chilanga sections and 110 mm overlays for the Chilanga - Kafue section and 50 mm thick overlays at 15 year intervals thereafter.

Option 6 20 year Design Option - As above but strengthened to accommodate the 20 year design traffic with 150 mm overlays for the Makeni - Chilanga section and 135mm overlays for the Chilanga Kafue section and 50 mm thick overlays at 20 year intervals thereafter.

Traffic volumes were the same in every case. It was assumed that there would be no generation of additional traffic by virtue of the improvements themselves.

Inputs to the model were obtained from surveys on the existing road and from data supplied by the Roads Department and by suppliers in Lusaka. The analysis has been undertaken in kwacha at official rates of exchange of US$ = K10 for the financial costing and at a rate of US$1 = K50 for the economic costing. In arriving at a shadow rate of US$1 = K50, the current parallel market rate of US$1 = K60 and a 20 percent

local Costs and 80 percent foreign costs for construction work of this nature was taken into account. The economic costings also exclude tax and duty elements but no adjustment has been made for the opportunity cost of labour or the effect of government subsidies other then through exchange rate valuation. Construction prices are based on the assumption that contractors will subsidise the Kwacha element of the project to obtain dollars and an exchange rate of US$ 1 to K30 has been applied to the financial costings on this element of the costings only.

Details of the various inputs to the economic model are given in Table 11.1.

For the purpose this economic analysis average characteristics for the existing surface condition and mean deflections were used for both the study sections. The whole life cost analysis was undertaken for a 30 year economic analysis period. The rehabilitation options 4,5 and 6 have been based on full width rehabilitation including shoulder widening in accordance with the standard recommended road cross sections.

It should be noted that during the latter stages of the study, on July 14, 1989, the Zambian Currency was devalued by the Central Bank, the new official rate of the currency is US$1 = K16. All project costings were undertaken in United States Dollars this currency value change will have little effect on the estimated Project Cost. Even in the case of the comparetive study of the economics there will be little effect, though it must be noted that the absolute value of the results may be questioned.

11.3 Output of Economic Analysis

The calculated whole life Financial, Economic and Foreign Exchange Cost of the Road on a cost per kilometre of road per year in Zambian Kwacha terms for the two sections of the road are presented in Tables 11.2 and 11.3. The costs have been calculated at 0 and 12 percent discount rates.

The option where the existing conditions of the road with routine maintenance, 100% pothole patching, surface dressing wherever wide cracking exceeds 15% of the pavement area and provision of a 50 mm overlay wherever roughness exceeds 2500 mm/km refered to as Periodic maintenance option B or generally as option 3 appears to offer the least cost in financial, economic and foreign exchange terms for both sections of the road.

	Private and light	4 Wheel and Min Bus	2/3 axle Rigid Truck	Rigid Trucks Plus Draw-Bar Trailer	Articulated Truck	Bus
HDM3 Veh Type	1	4	8	9	10	5
No of Tyres	4	4	6	14	14	6
Gross Veh. Wght	-	-	9.0	30.0	39.0	5.68
E S A	0.001	0.001	1.5	7.0	8.5	1.2
Average Axles	2	2	2	5	5	2
Financial Costs						
Vehicle	302500	298400	850000	1420000	1130000	477000
Tyres	510	900	3370	3370	4080	2660
Maint. Labr/Hr	90	150	150	150	150	150
Crew Cost/Hr	0	6.25	6.25	12.5	12.5	6.25
Interest Rate	25%	25%	25%	25%	25%	25%
Fuel:	Petrol =	5.16/lt	Diesel =	3.10/lt	Lubricants=	19.17/lt
Economic Costs						
Vehicle	692000	994700	3001000	5066000	4021000	1428200
Tyres	1100	1940	7390	7390	8940	5840
Maint.Labr/Hr	90	150	150	150	150	150
Crew cost/Hr	0	6.25	6.25	12.5	12.5	6.25
Interest	25%	25%	25%	25%	25%	25%
Fuel	Petrol =	5.42/lt	Diesel =	3.26/lt	Lubricants =	20.15/lt
Foreign Exchange Costs						
Vehicle	133100	191000	569000	943800	757000	277000
Tyres	150	270	1040	1040	1260	820
Maint.Labr/Hr	0	0	0	0	0	0
Crew Cost/Hr	0	0	0	0	0	0
Interest	25%	25%	25%	25%	25%	25%
Fuel	Petrol	4.74/lt	Diesel	2.85/lt	Lubricants	17.64/lt
Km Driven/Yr	20000	45000	50000	90000	90000	90000

Table 11.1 - Basic Input Data for the Economic Analysis costs in Kwacha

Note: ESA = Equivalent Standard Axle

	Discount Rate 0%			Discount Rate 12%		
	Financial Costs Yr/km	Economic Costs Yr/km	Foreign Exchange Costs Yr/km	Financial Costs Yr/km	Economic Costs Yr/km	Foreign Exchange Costs Yr/km
Option 1	19.260	47.017	11.997	4.173	10.117	2.673
Option 2	17.343	42.276	11.013	3.827	9.283	2.500
Option 3	14.193	34.277	9.450	3.547	8.557	2.360
Option 4	14.210	34.213	9.470	3.667	8.783	2.430
Option 5	14.280	34.400	9.503	3.697	8.840	2.447
Option 6	14.353	34.557	9.540	3.727	8.893	2.463

Table 11.2 - Average Road Cost (million Zambian Kwacha) per year per kilometre of road for the Makeni - Chilanga section for the do minimum option, two maintenance option and three rehabilitation options.

	Discount Rate 0%			Discount Rate 12%		
	Financial Costs Yr/km	Economic Costs Yr/km	Foreign Exchange Costs Yr/km	Financial Costs Yr/km	Economic Costs Yr/km	Foreign Exchange Costs Yr/km
Option 1	7.003	17.170	4.357	1.573	3.823	1.000
Option 2	6.330	15.600	4.013	1.400	3.437	0.913
Option 3	4.977	12.110	3.357	1.263	3.060	0.850
Option 4	5.053	12.230	3.403	1.377	3.287	0.913
Option 5	5.173	12.290	3.410	1.397	3.327	0.923
Option 6	5.117	12.387	3.433	1.417	3.370	0.933

Table 11.3 - Average Road Cost (million Zambian Kwacha) per year per kilometre of road for the Chilanga - Kafue section for the do minimum option, two maintenance options and three rehabilitation options.

However, this option is very closely followed by the 10 year rehabilitation design option referred to as option 4 where the road will be widened to class IA standard with 90 mm overlays to accommodate 10 year design traffic in the case of Makeni - Chilanga section and class IB standard width 80 mm overlays to accommodate 10 year design traffic in the case of Chilanga - Kafue section and with a provision for 50 mm thick overlays at 10 years intervals for both sections.

In order to evaluate the reletive merits of do nothing and the periodic maintenance options against the rehabilitation options, Internal Rates of Return were calculated for each rehabilitation option with each of the periodic maintenance options and the do nothing options as the base case. The results of this analysis are presented in Tables 11.4 and 11.5.

When compared against the base case of do nothing (or minimal maintenance) the 10 year design life rehabilitation option appears to be more attractive than the 15 and the 20 year options.

In addition as can be expected the Internal Rate of Return diminishes in value for all the rehabilitation design lives when compared with the periodic maintenance option A and further when compared with periodic maintenance option B.

11.4 Recommendation

From the economic analysis the two options worth further consideration are the periodic maintenance option B and the 10 year design life rehabilitation option. The average road costs in Financial, Economic and Foreign exchange terms at a 12 percent discount rate can be compared as follows.

A. Makeni - Chilanga section

	Financial Cost	Economic Cost	Foreign Exchange Cost
Periodic Maintenance	3.547	8.557	2.360
Rehabilitation	3.667	8.783	2.340
% difference	3.34%	2.64%	2.97%

	Option 4	Option 5	Option 6
Option 1	29.9%	27.7%	26.1%
Option 2	18.8%	17.7%	16.7%
Option 3	0.7%	-2.1%	-4.6%

For example, the Internal Rate of Return of the 10 year Design option improvement project (option 4) is 29.9% when the base case is the Do Minimum Option (option 1) and is 18.8% when the base case is Periodic option A (option 2) and so on.

Table 11.4 - Internal Rates of Return for the options on the Makeni - Chilanga section.

	Option 4	Option 5	Option 6
Option 1	24.6%	22.4%	20.7%
Option 2	15.3%	14.2%	13.2%
Option 3	-2.7%	-4.2%	-5.5%

For example, the Internal Rates of Return of the 10 years design option improvement project (option 4) is 24.6 when the base case is the Do Minimum Option (option 1) and is 15.3% when the base case is Periodic option A (option 2) and so on.

Table 11.5 - Internal Rates of Return for the option on the Chilanga - Kafue section.

B Chilanga - Kafue section

	Financial Cost	Economic Cost	Foreign Exchange Cost
Periodic Maintenance	1.263	3.060	0.850
Rehabilitation	1.377	3.281	0.913
% difference	8.29%	6.97%	6.90%

The Periodic maintenance option assumes that adequate funding will be available whenever required to undertake the periodic maintenance tasks that have been assumed will be undertaken at the correct times. Therefore, in the case of this option:

* Significant levels of funding will be required annually for pothole patching and the patching work must be undertaken to a defined standard of materials and workmanship.

* Significant level of funding will be required for surface dreesing of sections where the cracking exceeds 15% of the pavement area.

* Funding will be required at a five year cycle for 50 mm overlays which will be required whenever the roughness exceed 2500 mm/km.

Considering that with the best of intentions, even in developed countries it could be argued that adequate levels of funding is never provided for routine or periodic maintenance and that there are significant social and political pressures on administrators of road angencies in developing countries to allocate the scarce resources available for expanding the paved road network rather than provided for maintenance of existing roads, it could be argued that an option which minimises the annual recurrent road maintenance expenditure will be more preferable even at a higher capital cost.

In the case of the Kafue - Lusaka road where the road cost difference is only in the order of 3 percent in Financial, Economic or Foreign Exchange Terms (i.e. where the rehabilitation option is only 3% more expensive than the periodic maintenance option) for the Makeni - Chilanga section, and 8% in the case of Chilanga - Kafue section, it is recommended that the rehabilitation option be adopted.

Within the rehabilitation options the 10 year design life is quite clearly more attractive then the 15 and 20 year options. The Internal Rates of Return for the three design lives are as follows:

Design Life	Makeni - Chilanga	Chilanga - Kafue
10 years	29.9%	24.6%
15 years	27.7%	22.4%
20 years	26.1%	20.7%

Therefore it is recommended that the 10 year design life option be adopted for the rehabilitation project for both sections of road.

12. REHABILITATION PROPOSALS

12.1 Alignment

12.2 Cross Section

12.3 Drainage

12.4 Pavement Rehabilitation

12.5 Junction, Accesses and Railway Crossing

12.6 Road Furniture

12. REHABILITATION PROPOSALS

12.1 Alignment

The existing alignment of the study road is generally acceptable, and complies with the requirements of the Zambian Roads Department Standards. However, there are two locations on the more heavily trafficked section of the road (between Makeni and Chilanga), where the overtaking sight distances can be improved by realignment of the carriageway. At both locations the road level requires raising to facilitate surface drainage, so it is reasonable to consider a change in horizontal alignment at the same time.

The locations are as follows:

Location	Details
km 7.5	Left hand bend (Eureka Corner)
km 12.5	Left hand bend

12.2 Cross Section

The proposed Road Design Class is summarised below.

Section	Road Class	Width (m) C/way	Shoulder
00.0 - 03.9	Dual 2 - lane	2 x 7.3	3.0
03.9 - 15.1	IA	7.3	3.0
15.1 - 43.7	IB	6.7	2.0
43.9 - 53.3	IB	6.7	2.0

Any necessary embankment widening will be achieved by benching existing earthworks in steps not greater than 300mm deep (refer Figure 12.5).

12.3 DRAINAGE

A schedule of drainage proposals is given in 8.2.

12.4 PAVEMENT REHABILITATION

A schedule of pavement rehabilitation proposals is given in Appendix F.

12.4.1 Reconstruction - Dual Carriageway Section

Between km 0.5 and km 3.9 it is proposed that the existing carriageway level should be raised by approximately 500mm. the existing bituminous surfacing will be removed to expose the roadbase which will be scarified and recompacted to form the subgrade for new construction. The new pavement construction comprise 300mm cement stabilised natural gravel sub-base, 150mm crushed stone roadbase, and 60mm Dense Bitumen macadam surfacing, with a double bituminous surface treatment.

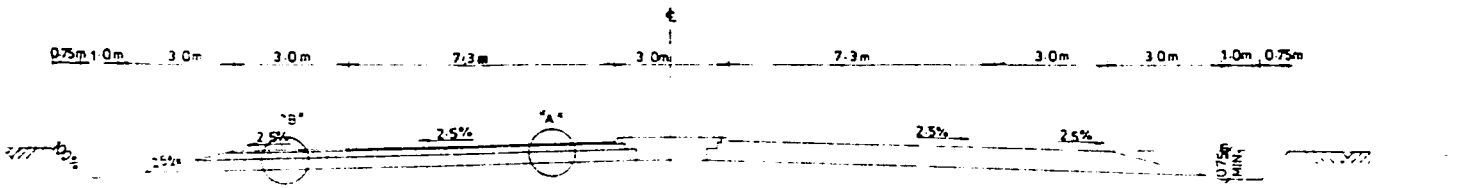
The shoulder widening will comprise a lower layer of sub-base quality natural material and a 300mm selected gravel layer. The carriageway roadbase (crushed stone) and surfacing (DBM) layers will be continued across the shoulder. A Cape Seal (single bituminous surface treatment plus slurry or sand seal) is applied to the shoulder. Reconstruction details are given in Figure 12.1.

12.4.2 Reconstruction - Single Carriageway Section

At those sections requiring raising or reconstruction, the existing bituminous layer will be removed to expose the roadbase which will be scarified and recompacted. The new pavement construction (for both Class IA and Class IB Sections) comprises 300mm selected gravel sub-base and 150mm crushed stone roadbase, with a double bituminous surface treatment. The Carriageway roadbase and sub base layer will be continued across the shoulders, and a Sand Seal surfacing applied to the shoulders. Reconstruction details are given in Figures 12.2 and 12.3.

12.4.3 Preparation of Existing Surface

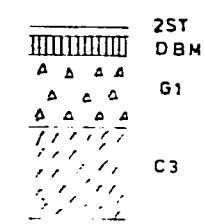
On those sections of road which are to be overlaid, all existing poor quality patchwork must be removed and redone and any potholes must be repaired. Existing cracks shall be sealed with bitumen and if the crack width exceeds 1mm, the existing surfacing shall be removed and replaced. Before application of the overlay the existing surfacing (repaired where necessary) shall be sealed with a single surface dressing which will help to delay the initiation of reflection cracking (Refer Section 12.4.5)



DUAL CARRIAGEWAY SECTION

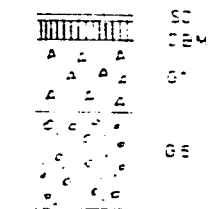
TYPICAL CROSS SECTION

(NTS)



DETAIL "A"

(NTS)



DETAIL "B"

(NTS)

Notes:

1. Design based on MSA and minimum subgrade CBR ≮ 15. Where not achieved selected subgrade layer is to be used.
2. Prime coat (where required) to be MC 30 @ 1.0 l/m2.
3. Tack coat (where required) to be K1-40 emulsion.
4. Lighting columns to be relocated in central reserve.

G1- Crushed stone roadbase and upper shoulder layer PI ≯ 6

G6- Selected gravel lower shoulder layer PI ≯ 12 CBR ≮ 25

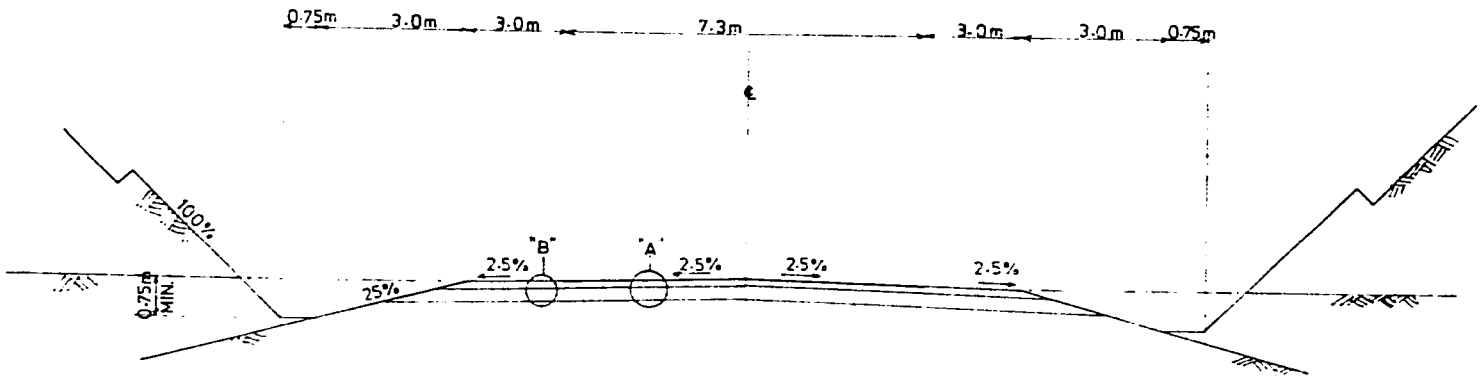
C3- Cement stabilised natural gravel sub-base UCS 1.5-3.0 M Pa

SC- Cape seal (single bituminous surface treatment plus slurry or sand seal) First seal 80/100 pen bitumen @ 1.3 l/m2 13.2mm chipping @ 80m2/m3. Second seal 80/100 pen bitumen @ 1.0 l/m2 crusher dust @ 0.0075m3/m2

2ST- Double bituminous surface treatment. First seal 80/100 pen bitumen @ 1.5l/m2 19mm chipping @ 70m2/m3. Second seal 80/100 pen bitumen @ 1.3 l/m2 13.2mm chipping @ 80m2/m3

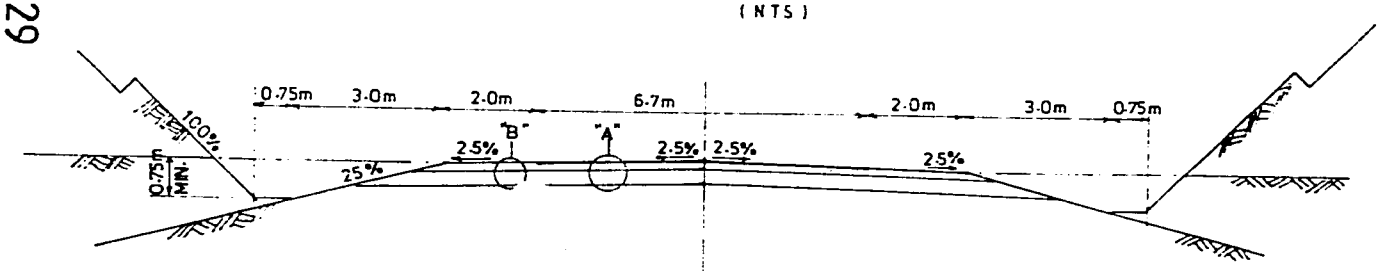
DBM- Dense bitumen macadam recipe mix (Nom. size depending on thickness) to base consequently (approx 4.4%, binder content) using 80/100 pen bitumen.

John Burrow & Partners CONSULTING ENGINEERS	KAFUE LUSAKA ROAD FEASIBILITY STUDY/COST ANALYSIS	RECONSTRUCTION DETAILS DUAL CARRIAGEWAY SECTION	Fig. 12·1



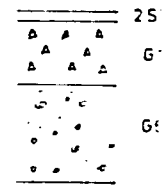
TYPICAL CROSS SECTION CLASS IA (RECONSTRUCTION)

(NTS)



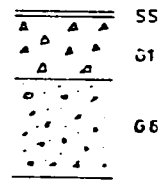
TYPICAL CROSS SECTION CLASS IB (RECONSTRUCTION)

(NTS)



DETAIL A

(NTS)



DETAIL B

(NTS)

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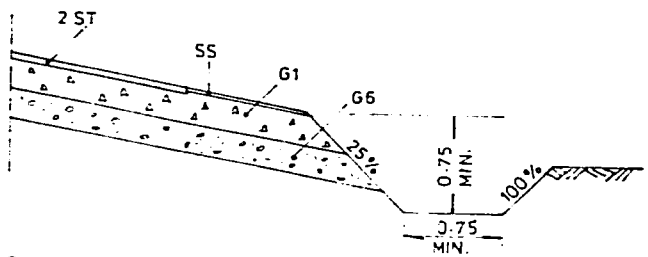
KAFUE LUSAKA ROAD
FEASIBILITY STUDY/COST ANALYSIS

RECONSTRUCTION DETAILS
SINGLE CARRIAGEWAY (Class IA and IB)

Fig. 12·2



CLASS IA AND I B SECTION (RECONSTRUCTION)

TYPICAL CROSS SECTION (NTS)

2ST - Double bituminous surface treatment
First seal 80/100 pen bitumen @ 1.5 l/m2 19mm chipping @ 70 m2/m3
Second seal 80/100 pen bitumen @ 1.3 l/m2 13.2mm chipping @ 80m2/m3

SS - Sand seal surface on shoulders
Seal 80/100 pen bitumen @ 1.0 l/m2
Crusher dust @ 120 m2/m3

G1 Crushed stone road base and shoulder material PI>6 CBR<80

G6 Selected gravel sub-base PI>12 CBR<25

Notes

1 Design based on 10 (Class IA) and 5(Class IB) m.s.a.

2 Prime coat (where required) to be MC30 cutback @ 1.0 l/m2

3 Tack coat (where required) to be RI-40 emulsion @ 0.6 l/m2

4 Cross section widths according to design standard.

5 Existing base course material (after removal of bituminous surfacing may be incorporated into the sub-base layer where required to match levels.

John Burrow & Partners CONSULTING ENGINEERS	KAFUE LUSAKA ROAD FEASIBILITY STUDY/COST ANALYSIS	RECONSTRUCTION DETAILS SINGLE CARRIAGEWAY (Class IA and IB)	Fig. 12·3

The above recommendations also apply to those sections of road which require "patch and seal" treatment. Typical construction details are given in Figure 12.4.

12.4.4 Overlay Materials

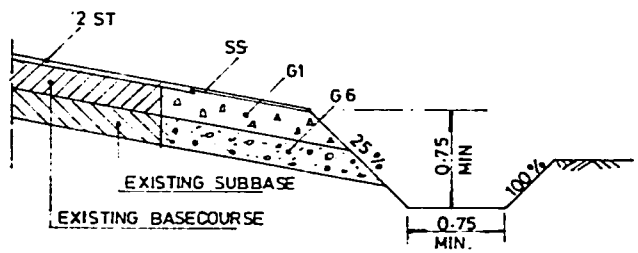
It is proposed that strengthening overlays for the road should be constructed using dense bitumen macadam basecourse material (binder course not roadbase) laid over a single spreay and chip surface dressing to retard reflection cracking, and with a double road surface treatment as a final wearing surface to resist enviromental oxidation of the bitumen. Typical details are given in Figure 12.5.

Dense bitumen macadam

Dense bitumen macadam, is a continuously graded hot mix asphalt produced to a receipe specification with a given deficiency of fine aggregate. This deficiency forces the mix to rely primarily on aggregate interlock for its load spreading properties and leads to a slightly higher void content than other continuously graded asphalts (such as asphaltic concrete).

Good aggregate interlock provides the least resistance to deformation under the adverse temperature and loading conditions that are considered to exist on this particular road. Very heavy individual axle loads can be expected on a small number of the commercial vehicles using the road. During the hottest periods of the year these will be instrumental in initiating track rutting in any less than stable materials. The high voids content can be overcome by sealing the surface with a thick bitumen film, such as the double seal surface treatment proposed. This will slow down the oxidation process by preventing the ingress of air and water.

Because the surface of the overlay material is sealed, a base course specification is appropriate with a suitablely reduced bitumen content (as much as 1% lower than if the same material was used as wearing course). This will represent a significant cost saving which will, at least partly, offset the cost of the surface seal.



CLASS IA AND IB SECTION (PATCH & RESEAL WITH OR WITHOUT WIDENING)

TYPICAL CROSS SECTION (N TS)

2ST - Existing surface patched and prepared by sealling cracks and applying a double bituminous surface treatment comprising: first seal 80/100 pen bitumen @ 1.5 l/m2 19mm chipping @ 70m2/m3 Second seal 80/100 pen bitumen @ 1.3 l/m2 13mm chipping @ 80m2/m3

SS - Sand seal on shoulders. Seal 80/100 pen bitumen @ 1.0 l/m2. Crusher dust @ 120m2/m3

G1 - Crushed stone road base and shoulder material PI $\geq$6 CBR $\geq$80

G6 - Selected gravel material PI $\geq$12 CBR $\geq$25

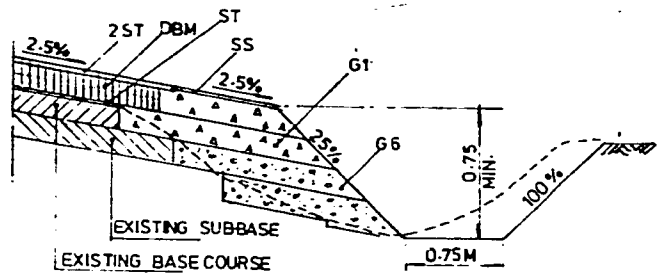
Notes

1 Only to be used where residual life exceeds 5(Class IA) and 2.5 (Class IB) m.s.a

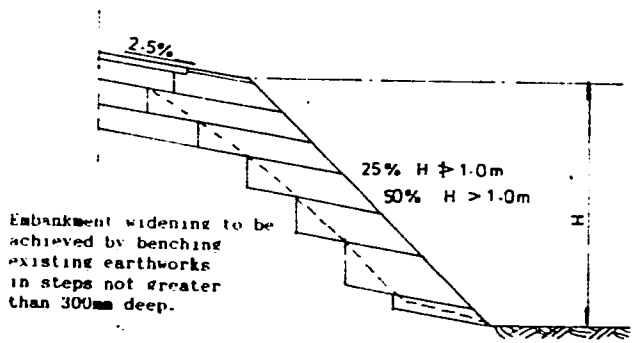
2 Prime coat (where required) to be MC30 cutback @ 1.0 l/m2

3 Tack coat (where required) to be K1.40 emulsion @ 0.6l/m3

4 Cross section widths according to design standard



CLASS IA AND IB SECTIONS (OVERLAYING WITH OR WITHOUT WIDENING)
(NTS)



CLASS IA AND IB SECTIONS (EMBANKMENT WIDENING)
(NTS)

2ST - Double bituminous surface treatment
First seal 80/100 pen bitumen @ 1.5 l/m2 19mm chipping @ 70m2/m3
Second seal 80/100 pen bitumen @ 1.31/m2 13.2mm chipping @ 80m2/m3

DBM - Dense bitumen macadam overlay in 2 layers Recipe mix (nom. size depending on thickness) to base course quality (approximately 4.4% binder content) using 80/100 pen bitumen.

ST - Existing surface patched and prepared by sealing cracks and applying a single bitumious surface treatment comprising 80/100 pen bitumen @ 1.3 l/m2 13 mm chipping @ 80m2/m3

SS - Sand seal surface on shoulders.
Seal 80/100 pen bitumen @ 3.0 l/m2
Crusher dust @ 120m2/m3

G1 - Crushed stone road base and shoulder material PI≯6 CBR≮80

G6 - Selected gravel material PI≯12 CBR≮25

Notes

1 Design based on 10 (Class IA) and 5 (Class IB) m.s.a

2 Prime coat (where required) to be MC30 cutback @ 1.0 l/m2

3 Tack coat (where required) to be K1-40 emulsion @ 0.6 l/m2

4 Cross section widths according to design standard.

John Burrow & Partners CONSULTING ENGINEERS	KAFUE LUSAKA ROAD FEASIBILITY STUDY/COST ANALYSIS	OVERLAY DETAILS / WIDENING DETAILS	Fig. 12·5

The overlay system proposed will provide a smooth riding surface with good load spreading properties and excellent resistance to enviromental deterioration. As a side benefit, it will provide a wearing surface that the Roads Department is well able to maintain by resealing in the future.

Alternative Overlay Materials

Other overlay materials have been considered as an alternative to the proposed dense bitumen macadam.

Crushed stone would have the advantage that reflection cracking would not arise but the overall load spreading properties of this material are not good (by comparison, that is, with hot-mix asphalt). A considerable additional thickness of overlay could, therefore, be required to meet the same design life criteria. Although sufficent aggregate sources could be located near Lusaka or Kafue, considerable haulage of materials would be involved and crushed stone would prove an expensive overlay material in haulage terms.

Penetration macadam has been rejected as an overlay material because the nature of its construction is such that a good riding quality is almost impossible to obtain. it is considered unlikely that the Roads Department would be satisfied with the riding quality that would result from the use of penetration macadam on a road of this importance and the economic consequences in terms of increased vehicle operating costs would be prohibitive. A further factor to remember is that penetration macadam has poor load spreading properties as compared with hotmix asphalt. Considering the amount of bitumen used in its production it does not constitute a cost effective use of the resources involved.

Gap graded and semi-gap graded hot-mix asphalt would have the advantage of increased resistance to fatigue cracking and better flexibility than continuously graded asphalts like dense bitumen macadam. But these improved qualities are obtained at the expense of increased potential deformation. It is doubtful that suitable gap graded or semi-gap graded mix could be obtained, having regard to the exceptional temperature and loading conditions on the road, without rsorting to the use of 40/50 pen bitumen, a grade never produced at the Ndola refinery.

Notwithstanding the above comments, should it be decided at the detailed design stage to investigate this alternative further, creep testing must be undertaken to prove the potential performance of these mixes using locally available aggregates. Any gap graded mix selected from such testing would be likely to have a bitumen content significantly higher than the dense bitumen macadam option proposed. This of course infers additional cost.

12.4.5 Reflection Cracking

Considerable research attention has been concentrated in recent years on the problem of reflection cracking and possible ways to avoid this phenomenon. The general conclusion reached is that total freedom from reflection cracking can only be guaranteed by complete removal or reprocessing of the existing pavement, or by application of very thick overlays the cost effectiveness of which is dubious.

Notwithstanding the above comments, it is generally accepted that the initiation and progression of reflection cracks can be delayed by providing a partial shear break between the existing surface and the overlay. Two ways of achieving this are worth discussing, geotextile and surface dressing.

Geotextiles are now extensively used in the break systems under overlays. The geotextiles, usually a non woven (needle punched) polyethylene, is laid over a thick layer of penetration grade bitumen which acts both as the break medium and as a water proofing treatment for the existing surface. The costs are generally quite high approximately US $5.00/sq.m., mostly in foreign exchange.

A single spray and chip surface dressing, performs very largely the same function as the geotextile and evidence from some quarters suggests this system is as succesful. In the Zambian context, surface dressing is more attractive from a cost point of view with a significant lcoal cost element and is a familiar process to contractors. On balance it is preferred as the measure of retarding reflection cracks and has been recommended for use on this project.

12.5 JUNCTIONS, ACCESSES AND RAILWAY CROSSING

12.5.1 Turning Circle at km 0 + 000

During the study the severe traffic congestion, particularly at peak hours but generally lasting throughout the business hours, was noted at the Kafue Road/Benbella Drive/Cairo Road/Independence Avenue turning circle at km 0 + 000. The congestion at the turning circle leads to congestion of traffic on the approaches resulting in delays of well over 20 minutes. The cumulative Cost of these delays, quite apart from the inconvenience to the road user, will be highly significant in national terms particularly as the delays occur during business/work hours leading to considerable loss of production.

It is quite clear that the size of the turning circle is insufficient for the traffic levels the intersection is required to cater for. As this intersection was outside the terms of Reference of this study no detail work was undertaken. However, from a cursory examination of the problem the following solutions have been considered and are discussed below.

- Increase the size of the turning circle and significantly modify the approach arrangements.
- Do away with a turning circle and introduce a four way, at grade, signalised junction.
- Introduce signalisation on the turning circle to operate at peak periods.

The first option, while being technically the most ideal is not practical as no additional land is available. The second option, while being functional and implementable will result in the removal of the central island which provides an attractive feature with lush vegetation on the entrance to the inner city area.

The third option combines functionality and aesthetics. Furthermore, the turning circle could be used normally during off peak hours without the signals. This type of intersection is used in many cities and functions very well.

Whilst this item of work has not been included in the scope of works under this rehabilitation project, it is recommended that in the event the Lusaka - Makeni section of the road is upgraded as part of the Kafue - Lusaka Road rehabilitation project then consideration be given to include improvement works on this intersection.

12.5.2 Other Junctions, Accesses and Railway Crossing

The junction and access design standards as defined in the Zambian Road Design Manual (Ref 6) is considered adequate. It is proposed that the existing junction layouts are largely retained with minor width modification.

The only Railway crossing on this Road is at km 40 + 000 near Kafue. This crossing is on a left hand bend on approach from Lusaka and on a right hand bend past the crossing. The bend in itself reduces vehicle speeds on approach to the crossing. This together with rumble strips on the road currently provide the control at this crossing. However, as this is a major trunk road and the rail line is a main trunk line, a signal operated by the railways is recommended.

This is a perfectly feasible option particularly as the Kafue Railway Station is only a very short distance from the crossing.

12.6 ROAD FURNITURE

12.6.1 Traffic Signs

It is recommended that all existing signs should be replaced and upgraded to international standards.

12.6.2 Lighting

On the dual carriageway section, based on the widening proposals, it is proposed to erect the existing lighting in the central reserve.

12.6.3 Crash Barriers and Guard Rails

Crash barriers are to be provided at the edge of shoulder where the height of fill above, adjacent ground level exceeds 2 metres. In addition, crash barriers shall be provided on the outside of bends through the escarpment section (km 37.0 - km 38.0) Guard rails shall also be provided at bridges.

12.6.4 Bus-bays and Lay-bys

It is recommended that bus-bays and shelters should be provided at all bus stops along the study route. This will involve extra widening and includes bus stops on the dual carriageway section. Buses should not stop on the carriageway to pickup or discharge passengers. It is also recommended that lay-bys should be provided at approximately km 28.0 and 50.0.

13. IMPLEMENTATION OF IMPROVEMENT WORKS

13.1 Phasing of Construction

13.2 Availability of Construction Material

13.3 Accommodation of Traffic During Construction

13. IMPLEMENTATION OF IMPROVEMENT WORKS

13.1 Phasing of Construction

A project implementation schedule is indicated in Figure 13.1. The schedule has been built up on the following assumptions.

- An invitation for the consultancy proposals for the detailed Engineering Design and Construction Supervision will be published in November 1989 and a two month period will be allowed for preparation of proposals.

- The consultancy proposal will be evaluated within a two month period and a consultancy contract will be awarded by early March 1990.

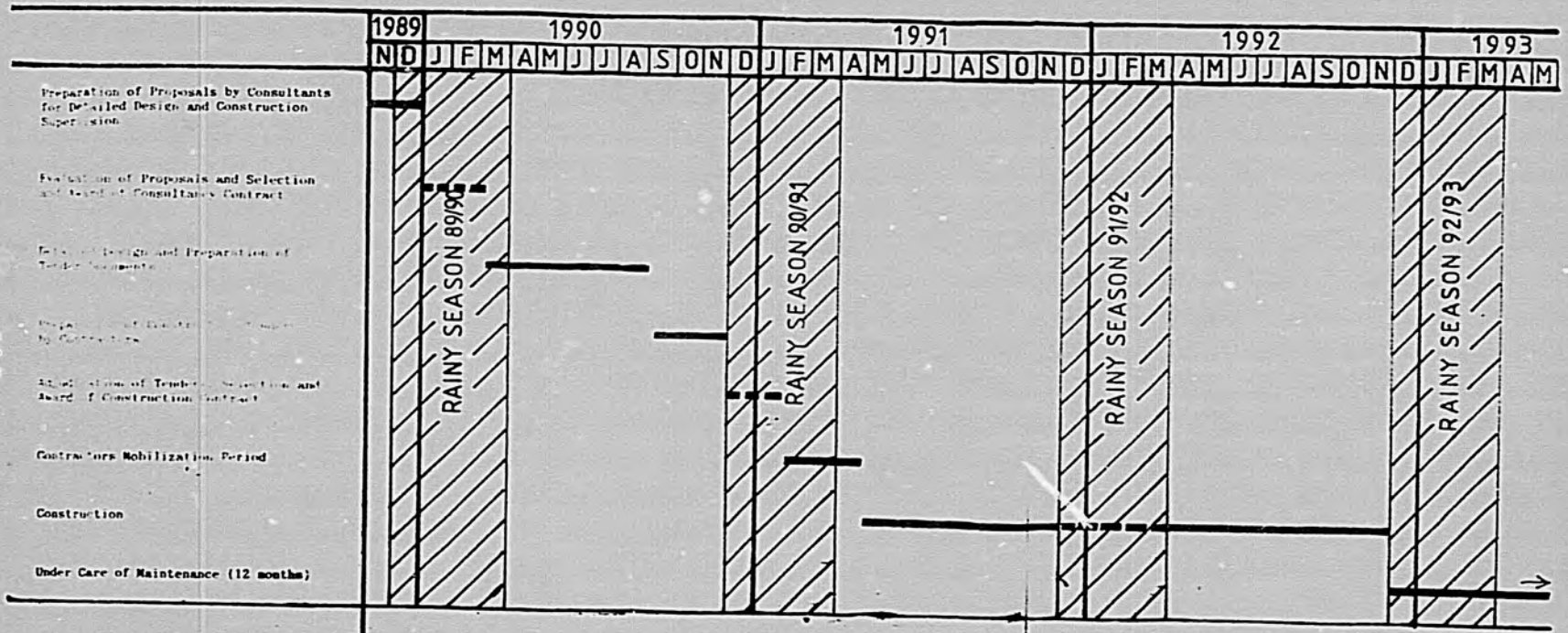
- The detailed engineering design and tender documentation can be completed within six months and prequalified contractors can be invited to prepare tenders by September 1990. Allowing for a three months tender preparation period tender evaluation can begin in December 1990.

- The tender adjudication, pre award negotiations and final selection of a contractor can be completed within a three month period and an award of construction contract can be made by early February 1991

- Allowing for a three months mobilization period bulk of which has been designed to fall in the 1990/91 rainy season and allowing for a 15 full dry months construction period, construction works would be completed by the end of November 1992.

- Allowing for a further 12 month period under the care of maintenance which will take the road through the 1992/93 rainy season, the project can be expected to be completed by the end of year 1993.

The time schedule proposed, while not having too much float time built into it, does have sufficient time allowed for the realistic completion of the various activities. Contractor mobilization during a rainy season and the opportunity to commence construction work on a relatively long dry season is an optimum opportunity. Furthermore, there is sufficient time



Project Implementation Schedule Fig. 13.1

periods of dry weather on either side of the 1991/92 rainy season, which will occur during the middle of the construction activity, to sensibly programme the works to make sure that the traffic does not need to run on deviations or detours during the rainy season.

Recognising that the rehabilitation works are not likely to commence till early 1991, in designing the rehabilitation works, the likely condition of the road pavement in mid 1991 has been estimated using observed rates of pavement deterioration. However, for the sections of the road which are to be either only patched and sealed or overlaid and sealed, it is important to preserve the residual life of the pavement. Therefore it is recommended that the contractor completes the work on the sections requiring patch and seal as early on in the construction period as possible followed by the sections requiring overlay and seal. The sections which are to be reconstructed can be worked on at anytime as the residual strength of the pavements is of very little importance to the reconstructed pavement. However, in the case of these sections of road it should be noted that the vehicle operating cost will be the highest and as such, the whole life cost of the road will rapidly increase with time as the road exponentially deteriorates with time.

13.2 Availability of Construction Material

The Construction works will require the following materials in large quantities. Base and sub-base quality gravels, crushed stone for base course, overlay layer and surfacing, Bitumen for overlays and surfacing, cement for stabilization, drainage works and bridge widening, sand for concrete works and water for construction and maintenance of deviations. The timely availability of the required materials at an economic cost is very critical to the implementation of the project. Therefore, the availability of these materials has been examined albe it at preliminary level within study. The output of this cursory examination must be developed to the required level of detail during the detailed design and tender documentation phase.

Base and sub-base quality gravels are obtained wherever possible from borrow pits along the alignment of the road. In opening up borrow pits a certain cost will be associated with bush clearing, stripping of topsoil and overburden and reinstating the pits at the end of the project. This cost should be compared with the cost associated with hauling the material to the position where it is required on the road.

Taking an ideal case, if suitable material is available adjacent to the road throughout the road alingnment numerous borrow pits could be opened up and the material haul distances would be very small. Depending on the depth of overburden and the thickness of the gravel layer the balance distance between borrow pits can be estimated. From experience, this economic spacing is in the order of 10 km. Therefore the feasibility of establishing borrow pits at this spacing has been examined.

The following existing borrow pits were visually examined and found to have suitable material.

- At km 5 + 000 approximately 1.5 km from the Left Hand Side of the road large borrow pits.
- At km 8 + 000 on the Right hand side of the road a large borrow pit adjacent to the road on land belonging to the Lusaka City Council.
- At km 15 + 000 on the Right hand side of the road approximately 1 km from the road.
- At km 20 + 000 on the Left hand side of the road very close to the road corridor.
- At km 27 + 800 on the Right hand side off the road.
- Between km 35 + 000 and 45 + 000 the is sufficient visual evidence to suggest suitable gravels are available.
- At km 51 + 800 within half a kilometre of the road on the Left hand side.
- At km 54 + 300 just past the Kafue River Bridge on the Left hand side of the road.

From the above it would appear that sufficient quantity of sub-base and base quality gravels will be available at economic haul distance throughout the road alignment.

Crushed stone will be required for the base layers of sections which are to be reconstructed, for overlays of sections which are to be sealed and for surfacing of the road throughout. The cost of the establishment required to quarry rock and crush the rock to produce stone of the required sizes is such that only one crushing site can

be economically justified for a road project of this lengths. i.e. It would be more economic to haul stone for 50 km than establish, dismantle and re-establish large scale crushing plants at several locations. The availability of single good source of stone was examined at a preliminary level during this study.

At km 38 + 000 the roads department owns and operates a quarry which provides crushed stone for their periodic and routine maintenance operations. This quarry also supplied the stone requirements of the Kafue Chirundu road rehabilitation project. It would appear that there is sufficient quantity of rock available at this site and the crusher has sufficient capacity to produce adequate quantities of stone to supply the Lusaka - Kafue road rehabilitation contract in addition to meeting the Roads Department periodic and routine maintenance requirements.

However it should be noted that the current layout of the quarry itself will grossly limit the amount of rock that can be taken out to crush. The current layout and method of working is in effect digging a progressively smaller and deeper hole on the side of a hill. The area available to work on at any given time limits the amount of rock that can be blasted and transported out. If the present production capacity is to be increased, which will have to be done if this quarry is to supply the stone requirements of the Kafue - Lusaka Road rehabilitation project, then the quarry layout and methods of working must be improved. The overburden on the upper part of the hill must be removed and a large working layer must be exposed and benched.

The Roads Department have expressed their reluctance to supply stone to the contractor for this project, one of the leading arguments is that this in effect makes them a subcontractor in addition to being the client and such a contractually conflicting situation is not desirable. One alternative that was briefly discussed and discarded due to the numerous institutional problems is the Contractor taking over the quarry for the duration of the road rehabilitation project and selling stone to the Roads Department at a pre agreed price to meet the departments periodic and routine maintenance requirements. One of the significant problems would be the legal implications of the temporary use of machinery and equipment belonging to the Government and the other would be the reluctance on the contractors part to use the staff currently employed by the Roads Department at this quarry.

In the event that the Roads Department is not prepared to supply stone then the other alternative would be for the Contractor to establish a new working at a different face of this hill or indeed at the adjacent hill. The contractor will then have to set up his own crushing plant as well at this site. If the Contractor is to crush his own stone, one alternative that is worth consideration is the establishment of the crusher at the site of the old Chilanga Cement Quarry on the Right hand side of the road at km 26 + 000. It would mean that rock blasted at km 38 + 000 will have to be hauled to km 26 + 000 for crushing. The significant advantage with the site at km 26 + 000 is that three phase power is readily available with all the step down transformer facility to hook up to a crusher. The power for crushing at km 38 + 000 will have to be supplied by diesel generators.

During detailed design another source of stone that is worth investigating further is an old disused quarry approximately 3.5 km from the Road or the Right hand side at km 27 + 800. The quality and quantity will need to be established but this location would be ideal as it is closer to the above crusher site and is approximately at the middle of the Lusaka - Kafue Road project i.e. the longest crushed stone haul distance would be in the order of 27 km from this site.

Bitumen is produced at the oil refinery in Ndola as a by product of the oil refinery process. However the capacity of the refinery is limited and if a few major road projects are underway simultaneously in Zambia the supply from this source will not be adequate. In estimating the foreign currency requirement for this project the eventuality of having to import bitumen has been allowed for. Cement is produced at a plant with an annual capacity of 200,000 tonnes operated by Chilanga cement at Chilanga which is at km 15 + 000 on the Kafue - Lusaka Road. No problems are envisaged with adequate supply of cement. No problems are also envisaged with the availability of adequate quantities of sand from existing sand quarries at Lusaka and Kafue.

Large quantities of water will be required for construction works. In addition even larger quantities will be be required to control the dust on the detours and deviations. It is anticipated that a fleet of three 10 000 l capacity tankers will carry on average

10 loads a day giving rise to a total water demand of 300 000 l/day or 300 cu.m per day. The following watering point have been identified from a preliminary investigation.

- At km 3 + 000 water can be pumped out of the quarry operated by Crushed Stone Sales.
- At km 7 + 000 a 6000 gallons/hour borehole supply is available.
- At km 15 + 000 a large quantity of water is available at the old Chilanga Cement quarry at the present Chilanga Golf course.
- At km 27 + 000 a very large quantity of water is available from the small lake formed by the disused Chilanga Cement quarry.
- At km 31 + 000 there is water available on the proposed deviations from a fast flowing stream.
- At km 36 + 000 on the proposed deviation water can be drawn from a stream.
- From km 40 + 000 to km 50 + 000 water can be abstracted from small streams or with approval from the Lusaka Urban District Council from hydrants off the Kafue - Lusaka bulk water pipeline.
- At km 53 + 300 water can be abstracted from the Kafue River.

During detailed design and contract documentation these sources should be confirmed. In addition it would be beneficial to indicate the potential sources in the tender documents to minimise the uncertainties that the contractor would have to price for.

13.3 Accommodation of Traffic During Construction

The Kafue - Lusaka road is one of the most heavily trafficked section of the Zambian paved road network system. The section between the turning circle at Independence Avenue in Lusaka and Makeni at km 3 + 900 currently carries over 15000 vehicles per day. The section between km 3 + 900 and 15 + 100 carries

approximately 5000 vehicles per day and the remainder of the road on average carries over 2000 vehicles per day. A significant proportion of this traffic comprises large multi axle vehicles.

Throughout the construction period, which is likely to be over 15 months, this traffic must be catered for. The traffic must be allowed to move in a safe manner with minimal inconvenience to the road user. The type of rehabilitation works propcsed will require total possesion of sections of the road for periods of several months. Therefore significant attention should be given to making adequate provision for the existing traffic at all stages of the project.

Having examined the options available the following solutions appears to be the most favourable at this stage. However the suggested deviations and detours must be examined in grater detail during the detailed design and tender documentation phase.

- km 0 + 000 to km 3 + 900 Construct single gravel strips within the road reserve but on the extremities of the road reserves on both sides and restrict cross over within that 3.9 km section.

- Km 3 + 900 to km 15 + 000 construct a 6.1 m gravel strip on the Right hand side of the road on the outer extremity of the road reserve.

- Km 15 + 000 to km 27 + 000 use the existing disused Chilanga Cement haul road which runs close to the main Road on the kight hand side. Only minor improvement required but considerable watering and some grader work will be required during use.

- Km 27 + 000 to km 28 + 200 Construct a 6.1 m gravel strip on the Right hand side of the road on the outer extremity of the road reserve.

- Km 28 + 200 to km 30 + 000 widen the existing Yacht Club road on the left hand side of the main road which then connects the old Kafue - Lusaka Road.

- Km 30 + 100 to km 40 + 500 widen and improve the old Kafue - Lusaka Road on the left hand side of the main road running close to the road till km 35 and then swinging to the left of the hills and coming back to the Road at Km 40 + 500.

- Km 40 + 500 to 41 + 200 widen and improve the old Kafue - Lusaka Road on the Right hand side of the main road.

- Km 41 + 200 to km 45 + 000 widen and improve the old Kafue - Lusaka Road on the left hand side of the Road close to the main road.

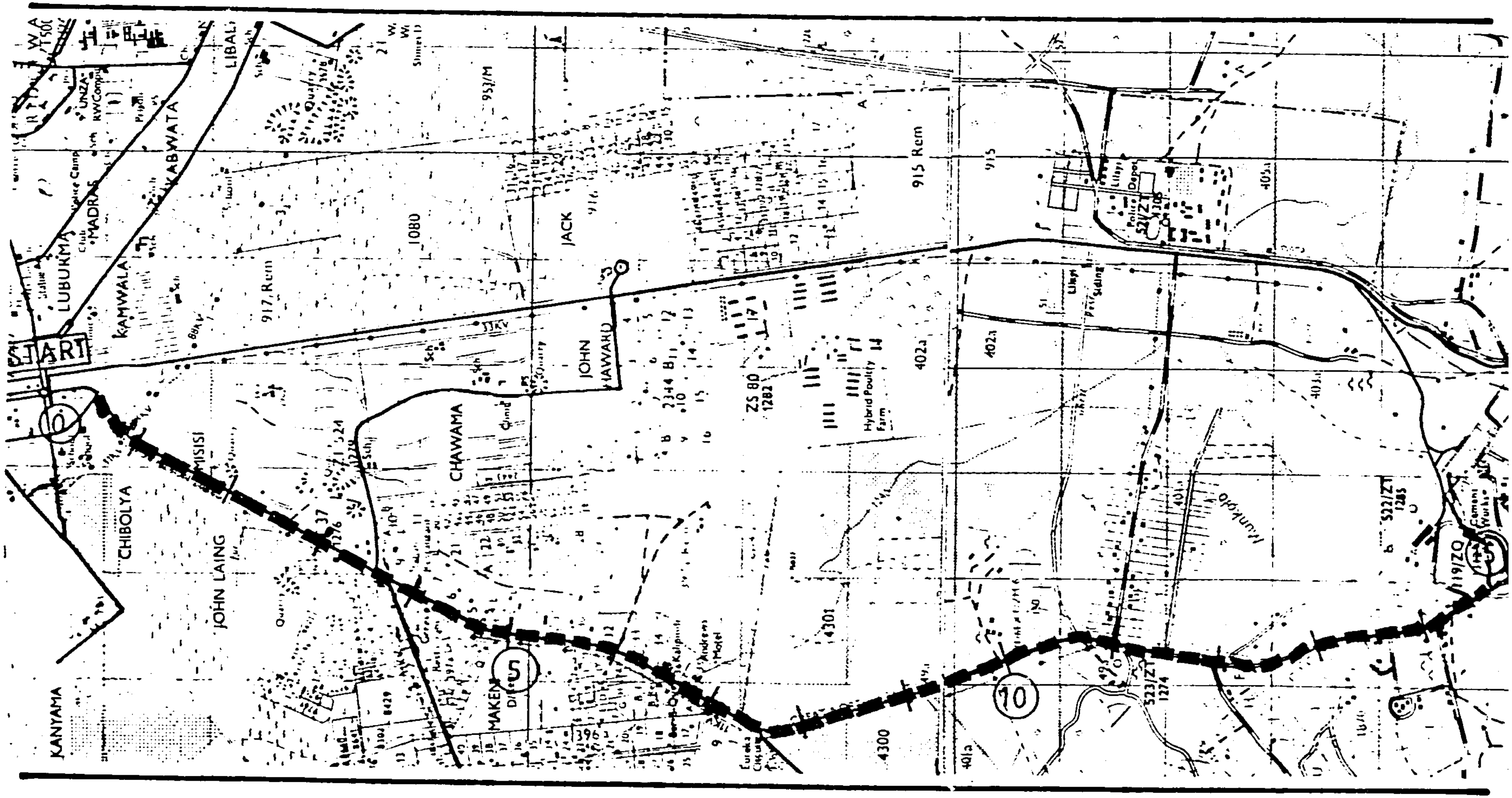
- Km 45 + 000 to km 46 + 200 construct a 6.1 gravel strip on the left hand side of the Road on the outer extremity of the road reserve.

- Km 46 + 200 to 53 + 300 widen and improve the old Kafue - Lusaka Road on the left hand side and well away from the main road.

In summary, approximately 16 km of new deviation roads will have to be constructed. However for the remainder, 37 Km, a combination of the disused Chilanga Cement quarry road and the old Kafue - Lusaka road can be used as deviations. In the case of the Chilanga Cement haul road, which is approximately 12 km in length the road was constructed to carry heavy offhighway dump trucks hauling rock and as such is in a very good condition and only requires little improvement. The old Kafue - Lusaka Road on the other hand will require widening and improvement. Fortunately bulk of the deviation is close to the main road and as such access to individual farms is not likely to be a problem.

However at two locations, km 35 + 000 to km 40 + 000 and km 46 + 000 and 53 + 000, the detours are well away from the main road and provisions will have to be made for any individual access. In both cases it would be feasible to let the individual users use the track by the side of the main road that will have to be maintained by the Contractor for use by his own construction traffic.

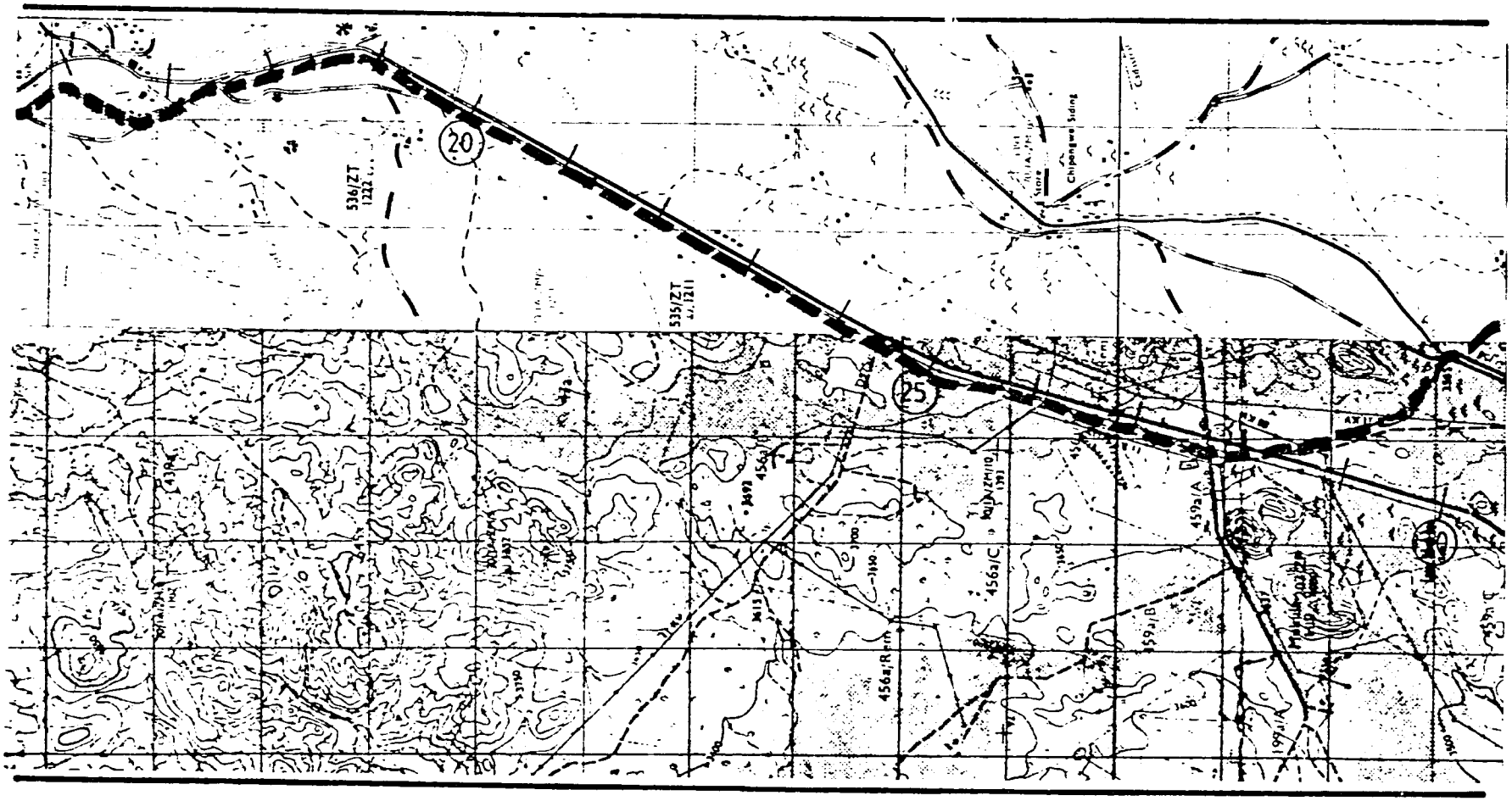
The possible detours are all indicated on 1:50 000 scale maps presented on Figure 13.2, 13.3, 13.4 and 13.5.



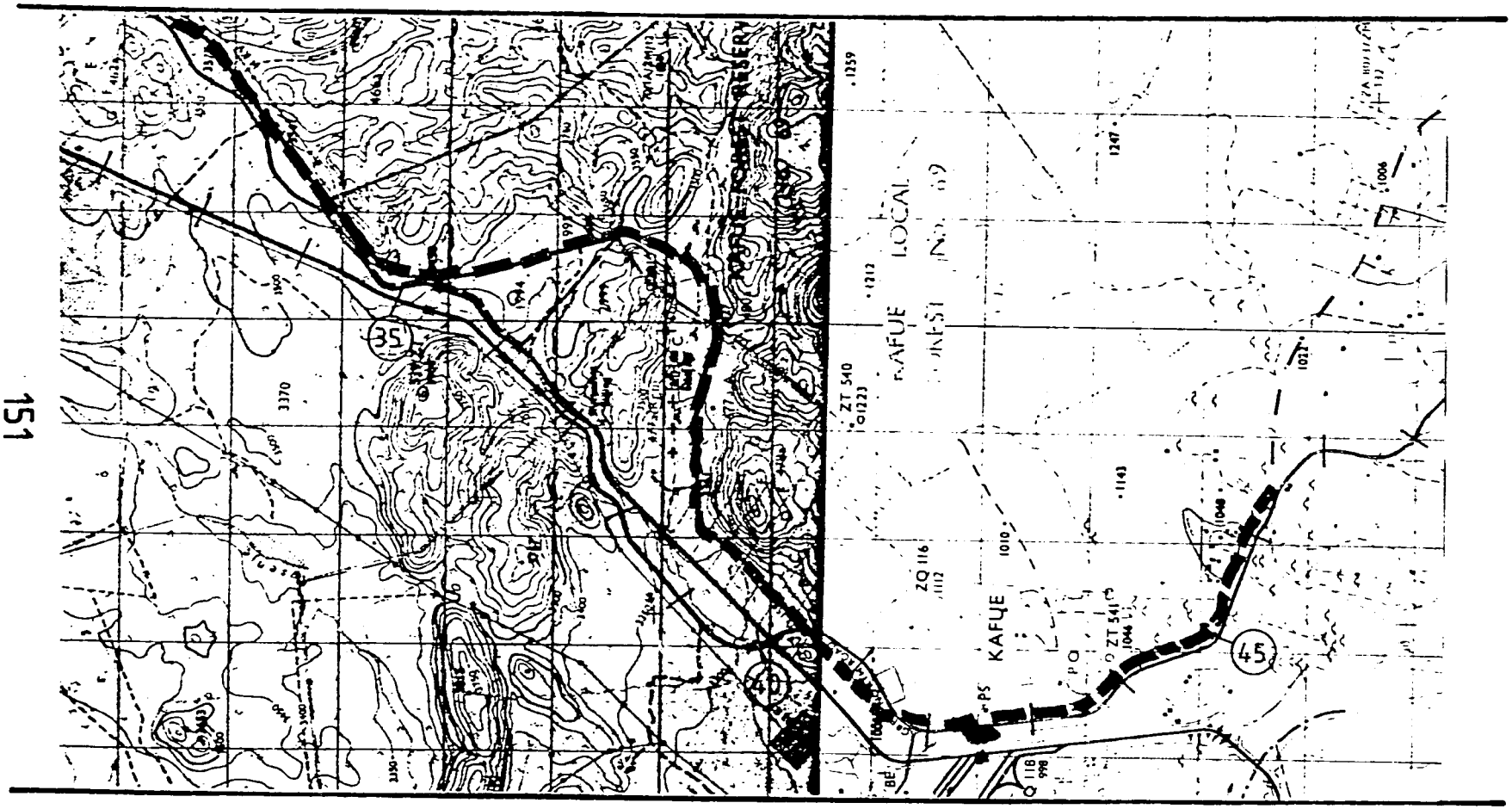
KAFUE LUSAKA ROAD
FEASIBILITY STUDY / COST ANALYSIS

ACCOMODATION OF TRAFFIC
KM 0+000 TO KM 15+000

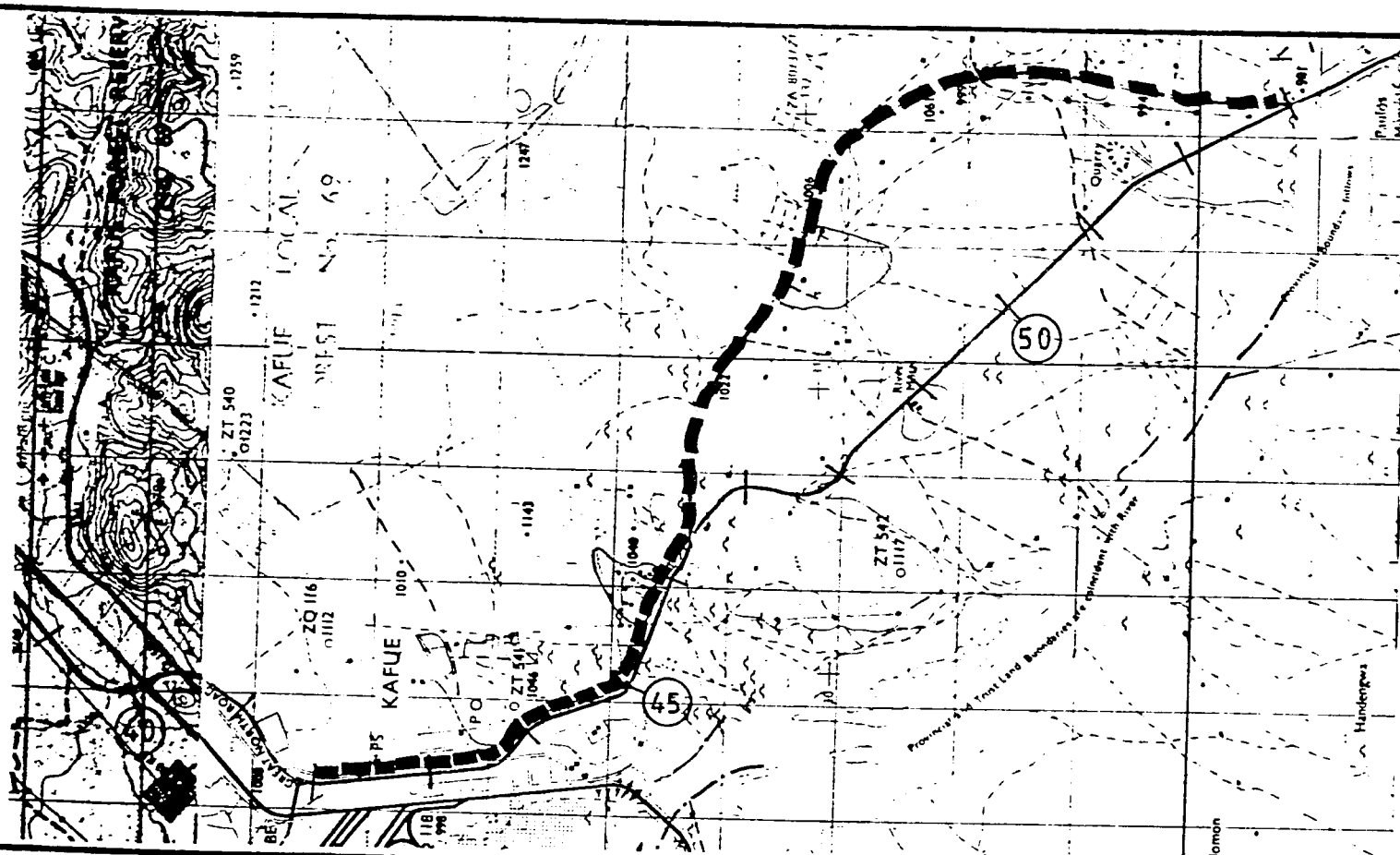
FIG 13-2



	KAFUE LUSAKA ROAD FEASIBILITY STUDY/COST ANALYSIS	ACCOMODATION OF TRAFFIC KM 15+000 TO KM 30+000	FIG 13.3



	KAFUE LUSAKA ROAD FEASIBILITY STUDY / COST ANALYSIS	ACCOMODATION OF TRAFFIC KM 30+000 TO KM 45+000	FIG 13.4



KAFUE LUSAKA ROAD
FEASIBILITY STUDY/COST ANALYSIS

ACCOMODATION OF TRAFFIC
KM 45+000 TO KM 53+300

FIG
13.5

14. COST ESTIMATE

14.1 Bills of Quantities

14.2 Unit Rates

14.3 Project Cost Estimate

14.4 Foreign/Local Currency Split

14.0 COST ESTIMATE

14.1 Bills of Quantities

The rehabilitation proposals recommended in Section 12 has been built up into work items and preliminary Bills of Quantities have been prepared. These preliminary Bills are presented as a separate document. While the Bills of Quantities are detailed with respect to the work items covered, it should be noted that the quantities themselves are only a first order estimate.

The work items have been grouped into the following Bills.

1.0 Preliminaries - which provides for all Contractor's preliminary and general expenses, cost of providing Surety and third party insurance, provision and maintenance for the contract period of offices, a laboratory and residential facilities for the supervisory Engineer and his staff, provision and maintenance of vehicles and telephone communication for the Resident Engineer's staff and provision to pay compensation to any land owners or settlers whose developments or properties may be affected by this project.

2.0 Accommodation of traffic - As discussed in Section 13 the traffic currently using the road should be provided for throughout the construction period. A network of detours and diversions will be provided and maintained during the construction period. The bill items provide for the improvement of existing detours or construction of new deviations to an acceptable standard, provision of 150 mm layer of gravel wearing course and the maintenance of these detours and deviations for an average seven month period when they will be required.

3.0 Borrow Pits - For pavement and shoulder widening, for sections where the road level needs to be raised, for sections where the road is to be reconstructed and for gravel wearing course for (detours) and deviations, lateritic gravels to varying standards will be required. In this Bill provision is made to clear vegetation, topsoil and overburden at approximately five suitable borrow pit sites and reinstate the pits at the end of construction.

4.0 **Dual carriageway section** - In accordance with the Terms of Reference, the section of the road between the turning circle at Lusaka and Makeni has been billed separately. In this bill all detailed work items pertaining to this section such as site clearance, removal of existing bituminous layer, earthworks to raise the level throughout and new pavement construction has been included. The only items that have been measured elsewhere are accommodation of traffic, drainage improvements and road furniture.

5.0 **Site Clearance and Topsoil Stripping** - virtually throughout the whole of the alignment the carriageway and the shoulders have to be widened. In this bill the work items related to clearing the strip of land either side of the existing roadway and stripping topsoil has been allowed for.

6.0 **Earthworks** - which comprises earthworks to raise the level of the road where required for drainage purposes and to build up widened shoulders.

7.0 **Shoulders to Existing Pavement** - on sections of the road where the existing pavement is only to be overlaid, or patched and sealed, as discussed before, the existing shoulders have to be widened. The proposed method of widening is to saw through the existing surfacing and base on the edge of the carriageway, remove the existing shoulder material and build up new shoulder to the full width from the lower layers. In this bill these work items up to road bed level have been allowed for.

8.0 **Existing Pavement Preparation** - In the case where the road is to be reconstructed it is recommended that the existing bituminous material be removed and disposed of and the existing base course be scarified and recompacted prior to placing any reconstruction layers.

9.0 **Subbase and base** - for widened shoulders and reconstructed pavements the sub-base and base shoulder material, stabilisation of sub-base gravel material, and crushed stone base for new construction have been allowed for. This bill also includes all overhaul of material. In the case of lateritic gravel materials it has been assumed that borrow pits will be established at

five locations along the alignment. In the case of crushed stone for new construction, carriageway and shoulder base, it has been assumed that the existing quarry at approximately km 36+500 will be the source.

10.0 **Dense Bituminous Macadams Base and Regulating Course** - This bill allows for the bituminous overlays and regulating course which will be used in sections of the road where the existing pavement is to be strengthened.

11.0 **Surface treatment and surface dressing crack sealing layer** - the whole length of the road is to be provided with a surface dressed seal layer. In addition the shoulders are to be sand sealed throughout. In the case of sections of the pavement which are to be overlaid the existing pavement must be sealed prior to placing the overlay. Items of work relating to these tasks are included in this bill.

12.0 **Pavement repair work** - Even where the existing pavement is to be overlaid or patched and sealed the pavement must be repaired prior to the placing of the next layer. All existing poor quality patchwork must be removed and redone and where pot-holes exist they must be repaired.

13.0 **Drainage improvement and Rehabilitation** - As discussed before the most important aspect of road design is the provision of adequate drainage works to protect the road from surfacing and ground water. In this Bill, all items relating to the repair or replacement of existing culverts provision of new culverts, as required, provision of mitre drains, catchment drains, subsoil drains and protection of drainage works has been allowed for.

14.0 **Miscellaneous Items** - Road furniture, signage, road markings and such items as guard rails and kerbing has been included. In addition a lump sum has been provided for constructing walkways on either side of the existing road-over-road bridge at Chilanga.

The style and format of the Bills are similar to that used for other Road Construction and Rehabilitation works in Zambia and are in genaral used with international standard conditions of contract. The method of measurement is also that used on similar contracts.

14.2 Unit Rates

The cost estimate have been prepared by extending these Bills of Quantities with unit rates of construction work for the various items. Unit Rates of construction in any environment comprises the various costs, of the constituent items, the overheads and supervisory costs; an allowance for the risk of undertaking this work and the projectas a whole. Such factors as, the current state of the construction industry, the level of activity in the construction sector, the economic climate of the country etc. all have a significant influence.

In addition each and every contractor will price an item of work in a different way, arriving at a different number. Further, in the case of Zambia where prices change at a fast pace it makes the exercise of establishing unit rates a very difficult task. In order to compare the alternative construction strategies available from a whole life cost point of view and select the most attractive option a brief economic analysis was undertaken using the World Bank model HDM III. This analysis required the establishment of financial, foreign exchange and economic cost of the key work items in pavement construction. For this execise a dozen major items were examined in detail and unit rates were built up from the basic constituents of labour, material and plant inputs. As these costings were undertaken largely for economic analysis where the benefit to the country as a whole is considered the items were priced in Kwacha and adjusted using parallel market foreign exchange conversion rates to arrive at economic costs.

However the projects is to be financed largely in foreign currency and in order to minimise currency fluctuation risks and protect the estimate from a contantly changing cost escalation factor, it was decided in agreement with the client to prepare the cost estimate in United States Dollars. In order to arrive at a meaningful cost estimate the bills of quantities were given to two large road contractors based in Zambia who are both familiar with this type of work and both of whom are considered capable of handling this scale of construction. They were both asked to price the bills on the basis that the project would be largely funded in foreign currency.

The difficulties in establishing unit rates can be seen by the following spread of rates on a selection of the major items.

		Contractor A	Contractor B
1.	Excavate in approved borrow pits load, haul, place and compact to form fills to 95% MDD (AASHO TEST)/cum	$8.50	$15
2.	Overhaul of pavement material per cum per kilometre	$0.40	$0.55
3.	Provide, spread and compact approved graded crushed stone to base including watering slushing and rolling required to provide a surface mosaic of tightly bound stone per cum	30.00	$45.00
4.	Provide, lay and compact bituminous premix pre cum	$120	$175
5.	Provide heat and spray 80/100 pen straight run bitumen at nominal rate of 1.3 l/sq.m unit price per litre	$0.90	$1.20

Establishment of unit rates is further complicated by the fact that the only comparable road project in Zambia was awarded in 1986 and the rates contained in that tender are out of date. Recently a tender for rehabilitation of the Kafue - Monze Road which is similar in nature to this project has been invited. The tender is not due in till early July and the rates are not likely to be available even for confidential use till October or November.

However in order to prepare a first order cost estimate, mean unit rates have been established and whereven possible these have been cross checked against rates built up from basic prices and costs.

In general the Unit Rates have been prepared on the assumption that the Contractor will be responsible for the whole of the works including procurement or winning all material required and transporting all Material from source to the location where they will be incorporated into the works. For example in the case of crushed stone it has been assumed that the Contractor will Quarry stone from his own source and crush it. As discussed in Section 13, even if the Roads Department Quarry at Kafue is used, due to its current state significant costs similar to that required for opening a new quarry will arise from expanding the current working face. Thus, the rates built up for crushed stone on the basis that the Contractor will do his own Quarrying and crushing will not be that different from the rate the Roads Department will have to charge for supplying the stone from the Quarry at Kafue.

14.3 Project Cost Estimate

Preliminary estimate of the total cost of the rehabilitation of the road from the Independence Avenue/Kafue Road turning circle in Lusaka to the start of the Kafue River Road Bridge past Kafue township using June 1989 prices is twenty two million nine hundred and seventy nine thousand United States Dollars (US$ 22.979 million). This figure includes a ten percent contingency provision and the cost of the design and construction supervision consultancy. It should be noted that a ten percent contingency figure is considered adequate as the quantities have been estimated to a preliminary engineering level.

The cost estimate has been built up as follows:

1.0	Preliminaries	3 234,000
2.0	Accommodation of traffic	1 331 000
3.0	Borrow pits	430 000
4.0	Dual Carriageway section	3 028 000
5.0	Site clearance and topsoil stripping	550 000
6.0	Earthworks	1 030 000
7.0	Shoulders to existing pavement	383 000
8.0	Existing pavement preparation	382 000
9.0	Sub-base and base	2 770 000
10.0	Dense Bituminous Macadam Base and Regulating Course	2 248 000
11.0	Surfacing treatment and surface dressing crack sealing layer	2 033 000

12.0	Pavement repair work	195 000
13.0	Drainage improvement and rehabilitation	1 404 000
14.0	Miscellaneous items	417 000
		19 435 000
Contigency @ 10 %		1 944 000
Detailed design and construction Supervision Consultancy cost		1 600 000
Total Estimated Project Cost at June 1989 prices:		22 979 000

In accordance with the Terms of Reference the cost of the section of the road from the turning circle at Lusaka to Makeni, the dual carriageway section, has been recosted separately. This sector of the road is expected to cost in the order of four million five hundred thousand United States Dollars (US$ 4.5 million).

The mean that the cost of the rehabilitation works on the section between Makeni and Kafue Bridge is eighteen million four hundred and seventy nine thousand United States Dollars (US$ 18 479 000).

It is strongly recommended that these cost estimates be reviewed when information on the tendered rates for the Kafue - Monze project becomes available.

14.4 Foreign /Local Currency Split

As discussed before the cost estimate has been prepared on the basis that sufficient funds will be available to the contractor in Foreign currency. On a project of this type which is largely a plant intensive construction a significant component of the total costs will be incurred in foreign currency.

All the following items for example are plant intensive.

- Site clearance, strip topsoil on required areas of the road alignment and at borrow pits, cart away and dispose or temporarily store and reuse.

- Excavate suitable gravel material for earthworks and pavement construction layers, load, haul, place and compact.

- Cut and fill on road alignment to form correct levels of road bed.

- Strip existing bituminous surfacing for sections of reconstruction and scarify and recompact existing base.

- Drill, blast, crush, screen, load, haul, place and compact crushed stone for use as base course, bituminous layers and surfacing.

- Heat, mix, load, haul, lay and compact Dense Bituminous Macadam overlay and regulating course.

- Pavement repair works such as saw through existing pavement, patch and seal.

- Drainage improvement works such as excavate side drain mitre drains catchwater drains and cross drains.

It is virtually impossible to buy construction plant or spares for plants in the local currency. Even on the rare occasion when it is available locally the price of the item in general in equivalent to converting the foreign cost at well over US$ 40 to the Kwacha. The only items that could be classified as local components are the cost of the plant operator, fuel and oils. Therefore, the cost of owning and operating plant is largely in foreign currency.

The total hourly cost of owning and operating at 1987 prices separated into the local and foreign currency components for a Cater pillar D7 Bulldozer has been shown in Table 14.1 and summarised below as an example.

	$/Hr	K/Hr
Total Hourly owning cost Total Hourly operating cost	41.00 38.40	95.61 141.12
Total cost	79.40	236.73
% of cost as component of Total cost	77%	23%

Construction activity requires the input of plant, materials and labour. In the case of materials, other than the materials that will be obtained or produced on site such as crushed stone, sand and gravel, this project will require significant quantities of bitumen and cement. While both these items are available for purchase in the local currency is should be noted that in the case of bitumen the capacity of the oil refinery at Ndola is limited and in the event that a few major road projects are underway simultaneously the local bitumen supply will not be adequate. Therefore in any estimation of foreign currency requirement, provision should be made for this eventuality.

The only item of cost which will have no component of foreign currency will be the cost of labour. However even the Contractor's cost of superintendence and management of the contract will have a significant component of foreign currency expenditure attached to it.

While the exact split of foreign and local currency components will be difficult to establish, it is estimated that the local currency component for this type of work will be in the region of 20 to 25 percent and the foreign currency component will be in the region of 75 to 80 percent of the total cost. These components will also be significantly affected by the type of Contractor selected to do the work.

	Variable	Info	Total $	Total K	$/Hr	K/Hr
New Purchase Price FOB	220000					
Freight to Dar	6000					
C & F		226000				
Insurance (C&F+25%*1,4%)		3955				
CIF			229955			
Dar Port Charges %	5		11498			
Transport Dar/Lus	7000		7000			
Exchange Rate	8					
Kwacha Equivalent CIF LUS		1987622				
Duty %	15			298143		
Sales Tax on K Value	20			556534		
Import Levy % CIF	5			99381		
Total Purchase Cost TPC			248453	954059		
Equiv. TPC K		2941681				
No of vehicle tyres /U Ca	1					
Unit price Tyres/U Carr	25000					
Deduct cost of Tyres			-25000			
Deduct Cost of GET	1000		- 1000			
Value to be Recovered new			222453	954059		
Actual value recovered	1		222453	954059		
Deduct res Val %	0		222453	954059		
Depreciation Period	5					
Working Hrs/Yr	2000					
Cost per Hr					22.25	95.41
Original Purchase Price $	367710					
Int on OP Price %	12				13.24	
Current Value $	367710					
Ins Cost (% Current Value)	3				5 52	
Licence K	400					0.20
TOTAL HOURLY OWNING COSTS					41.00	95.61
Fuel consumption /Hr	37					
Unit Price Fuel	2.55					
Fuel Cost/hr						94.35
Oil/Grease %	10					9.44
Repair Cost % TPC	20					
Labour %	10					
Spares %	90					
Labour Cost per Hr						27.34
Spares Cost per Hr					30.75	
Tyre Life Hrs	3500					

Table 14.1 - Part I - Breakdown of plant owning and operating cost for a caterpillar D7 Bull dozer (1987 prices)

	Variable	Info	Total $	Total K	$/Hr	K/Hr
No of Tyres/annum		0.57143				
Tyre Cost					7.14	
G.E.T Cost	1000				0.50	
Operators Cost per Hr	10					10.00
TOTAL HOURLY RUNNING COST					38.40	141.12
TOTAL HOURLY OWNING COSTS					41.00	95.61
TOTAL HOURLY RUNNING COST					38.40	141.12
TOTAL HOURLY COST					79.40	236.73

Table 14.1 - Part II - Breakdown of plant owning and operating cost for a caterpillar D7 Bulldozer

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   (c) Appendix A : Pavement Strength and Condition.

   (d) Appendix B - 1 : Traffic Surveys
   Appendix B - 2 : Population
   Appendix B - 3 : The Zambian Economy
   Appendix B - 4 : Unit Construction and Maintenance Costs
   Appendix B - 5 : The HDM III Model

   (e) Appendix c : Summary of Economic Evaluation

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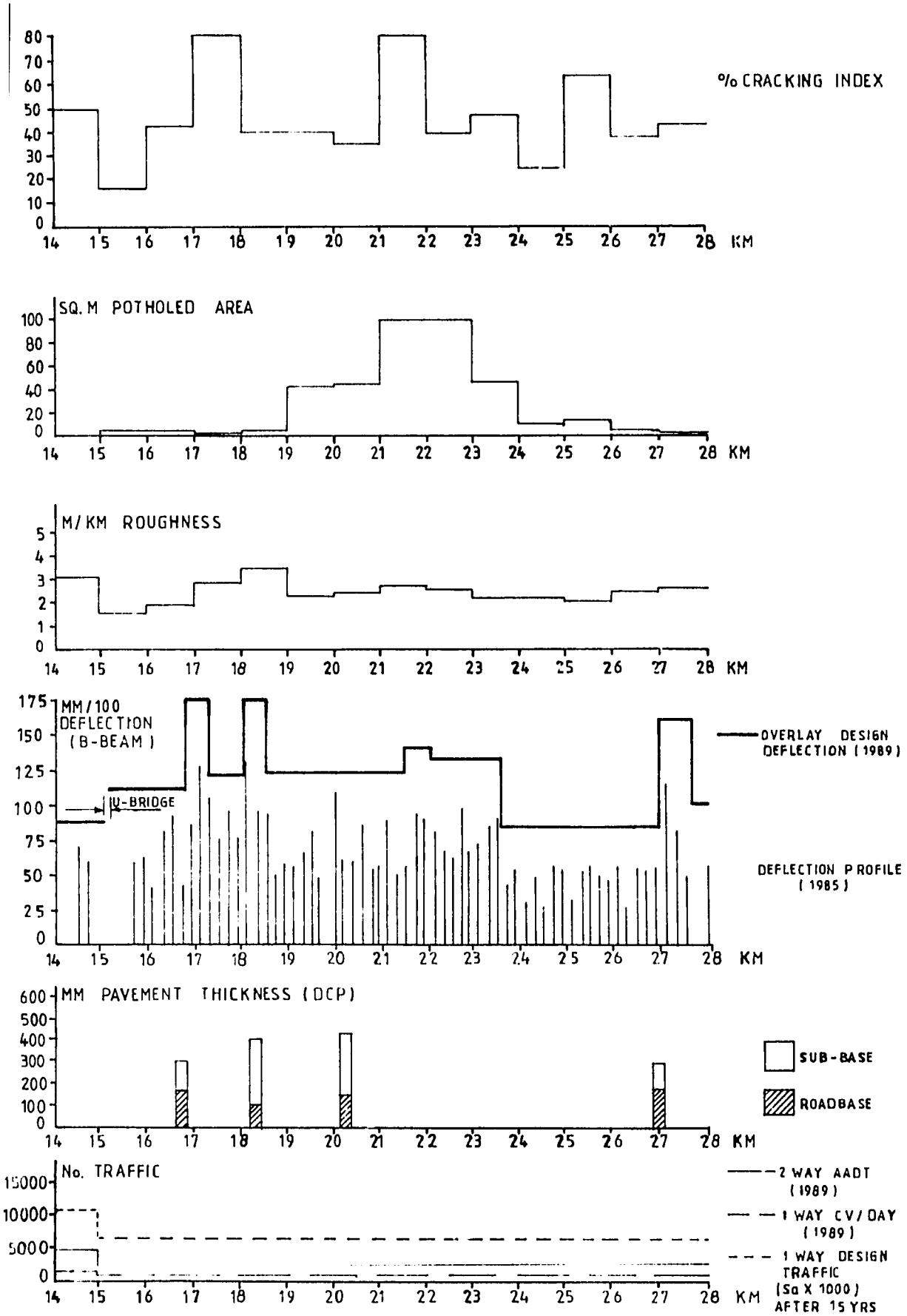
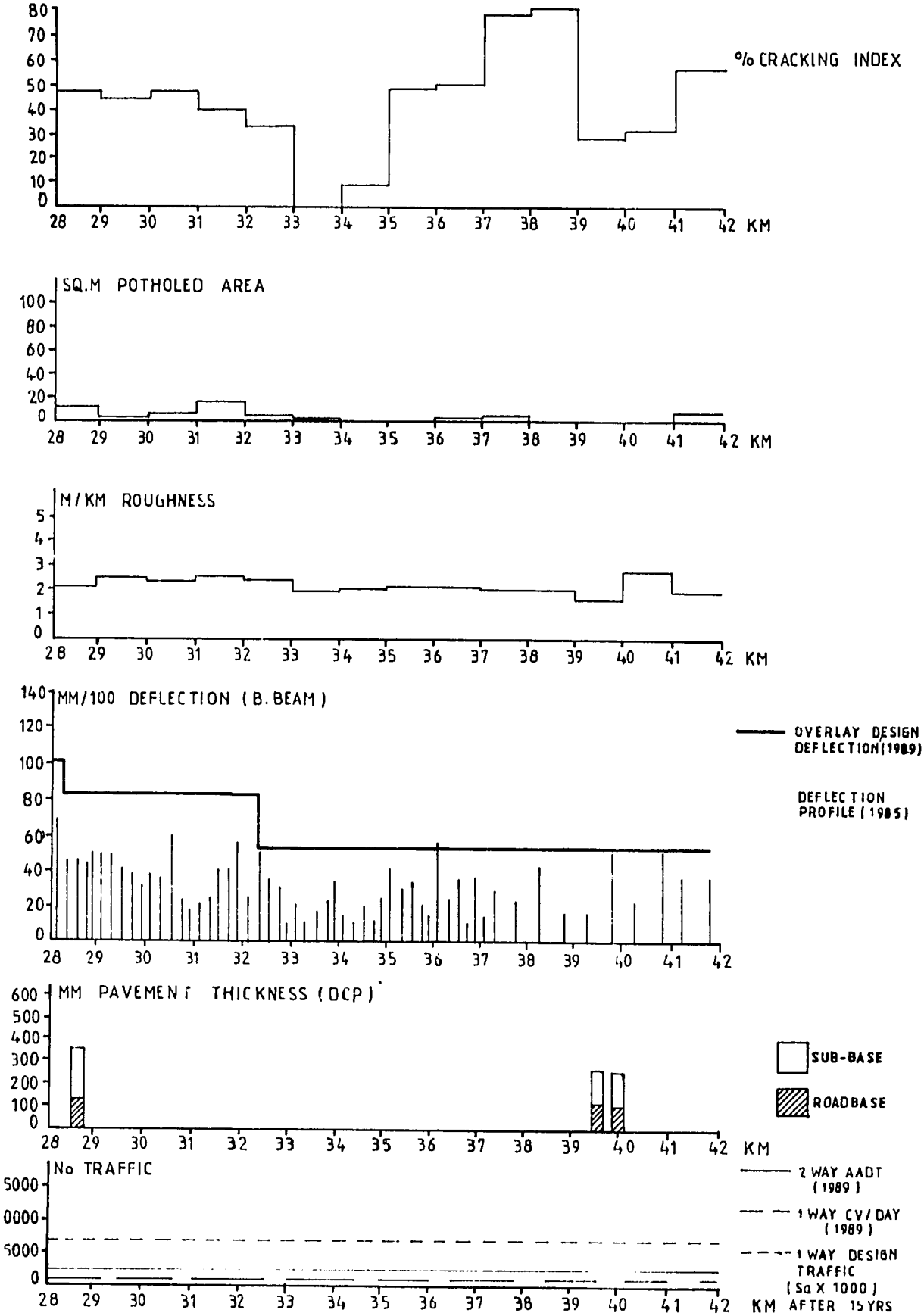
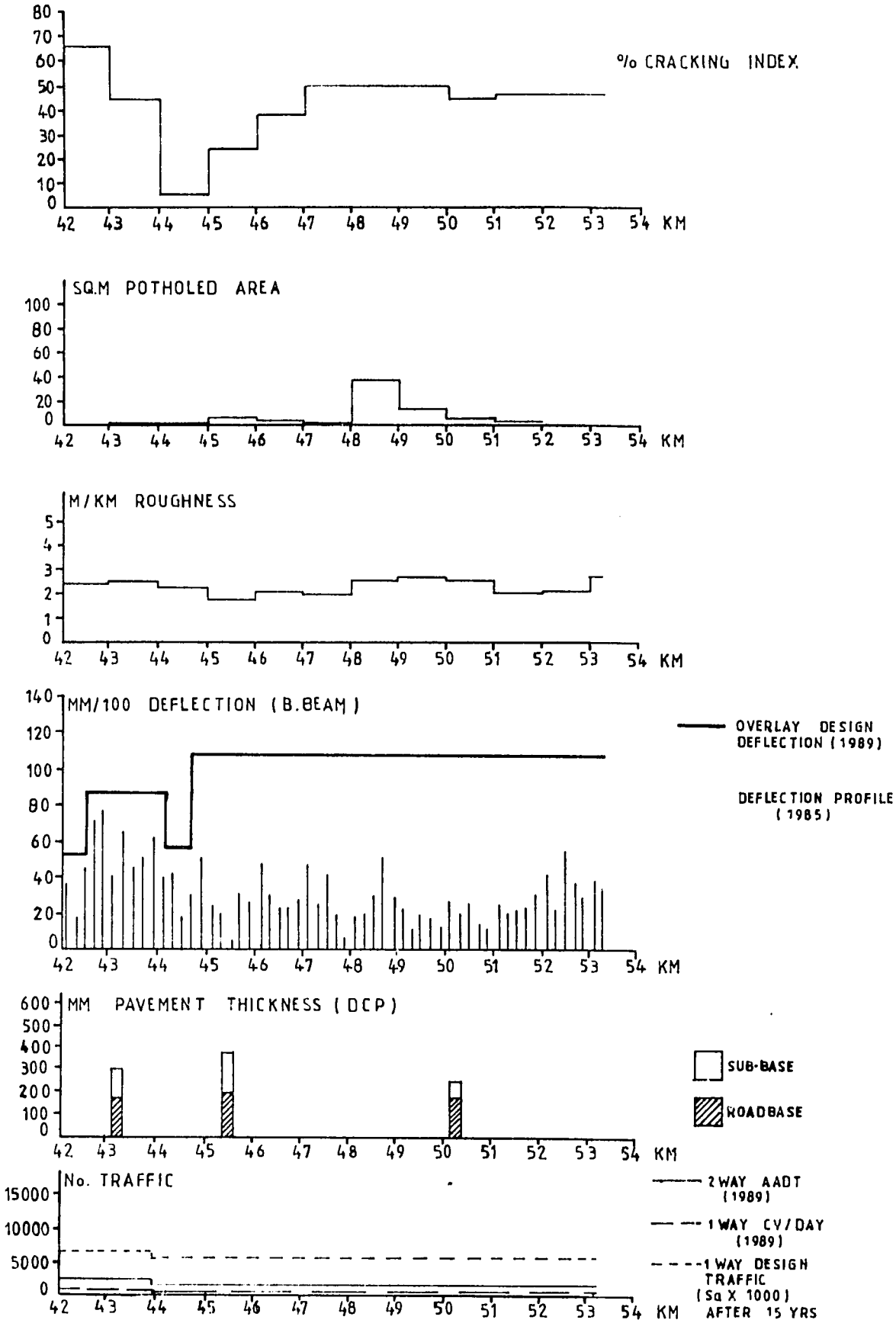
APPENDIX A

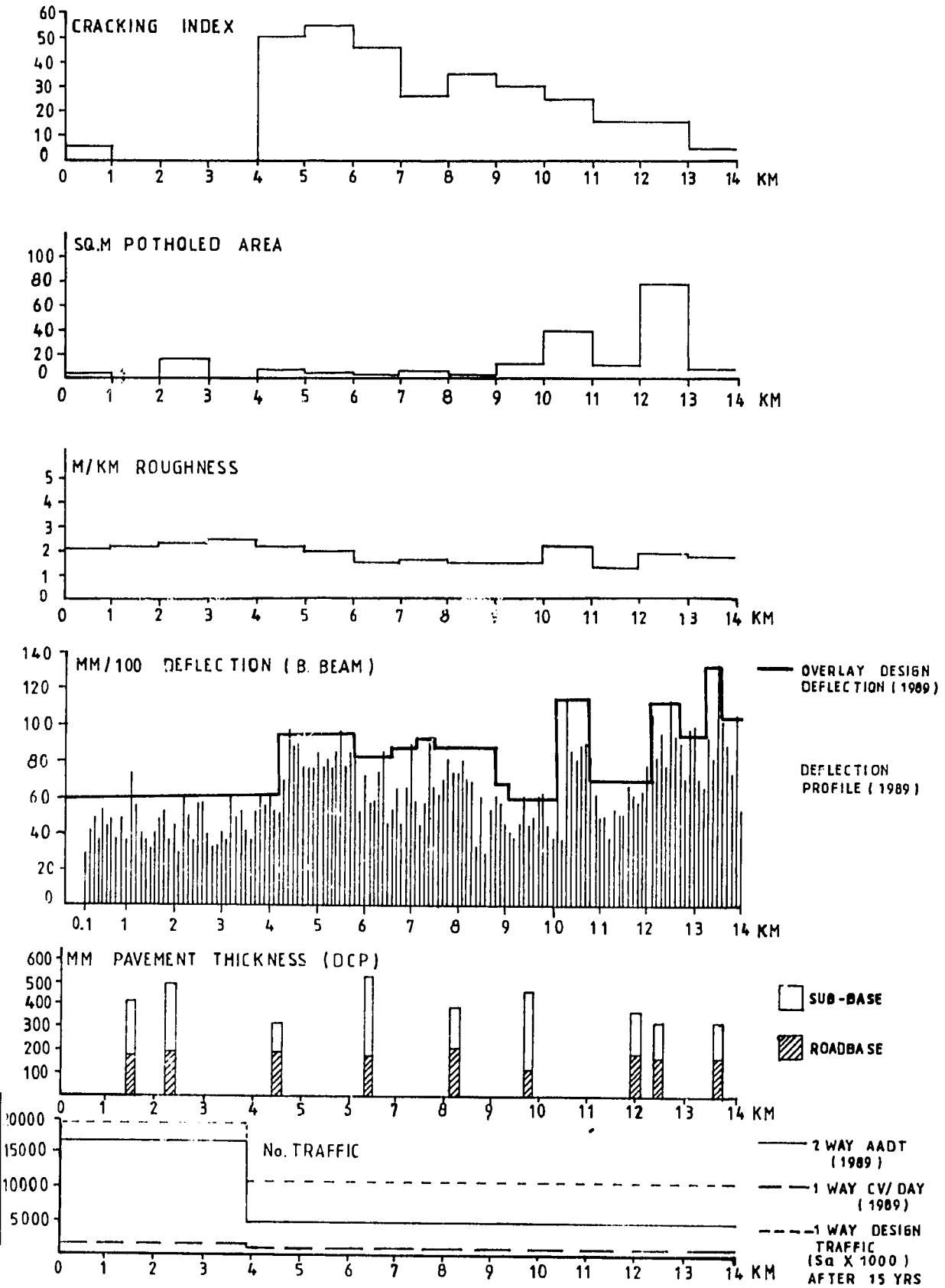
Road Survey Results

Appendix A	:	Summary of Survey Results
Cracking Index	:	Percentage of sampled points which exhibit a Crack Intensity greater than 2 m/square.
Potholed Area	:	Area of Potholes expressed in square metres.
Roughness	:	Apparent Roughness value (in m/km) deduced from vehicle mounted bump integrator data. Value displayed are the average value from two runs, one in each direction.
Deflection	:	Transient deflection (mm/100) measured using the Benkelman Beam. The value shown is the higher value from the outside wheeltrack in each direction.
Pavement Thickness	:	Layer thickness deduced from Dynamic Cone penetrometer tests.
Traffic	:	Data from Traffic counts in 1989 with projected MSA after 15 years.


% CRACKING INDEX
SQ. M POTHOLED AREA
M/KM ROUGHNESS
MM/100 DEFLECTION (B-BEAM)
U-BRIDGE
OVERLAY DESIGN DEFLECTION (1989)
DEFLECTION PROFILE (1985)
MM PAVEMENT THICKNESS (DCP)
SUB-BASE
ROADBASE
No. TRAFFIC
2 WAY AADT (1989)
1 WAY CV/DAY (1989)
1 WAY DESIGN TRAFFIC (Sa X 1000) AFTER 15 YRS
KM


% CRACKING INDEX
SQ.M POTHOLED AREA
M/KM ROUGHNESS
MM/100 DEFLECTION (B.BEAM)
OVERLAY DESIGN DEFLECTION(1989)
DEFLECTION PROFILE (1985)
MM PAVEMENT THICKNESS (DCP)
SUB-BASE
ROADBASE
No TRAFFIC
2 WAY AADT (1989)
1 WAY CV/DAY (1989)
1 WAY DESIGN TRAFFIC (Sa X 1000) AFTER 15 YRS
KM


% CRACKING INDEX
SQ.M POTHOLED AREA
M/KM ROUGHNESS
MM/100 DEFLECTION (B.BEAM)
OVERLAY DESIGN DEFLECTION (1989)
DEFLECTION PROFILE (1985)
MM PAVEMENT THICKNESS (DCP)
SUB-BASE
ROADBASE
No. TRAFFIC
2 WAY AADT (1989)
1 WAY CV/DAY (1989)
1 WAY DESIGN TRAFFIC (Sa X 1000) AFTER 15 YRS
KM


CRACKING INDEX
SQ.M POTHOLED AREA
M/KM ROUGHNESS
MM/100 DEFLECTION (B. BEAM)
OVERLAY DESIGN DEFLECTION (1989)
DEFLECTION PROFILE (1989)
MM PAVEMENT THICKNESS (DCP)
SUB-BASE
ROADBASE
No. TRAFFIC
2 WAY AADT (1989)
1 WAY CV/DAY (1989)
1 WAY DESIGN TRAFFIC (Sa X 1000) AFTER 15 YRS
KM

APPENDIX B

D C P Profiles

DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test No : DCP 17

Chainage (km) : 1.5

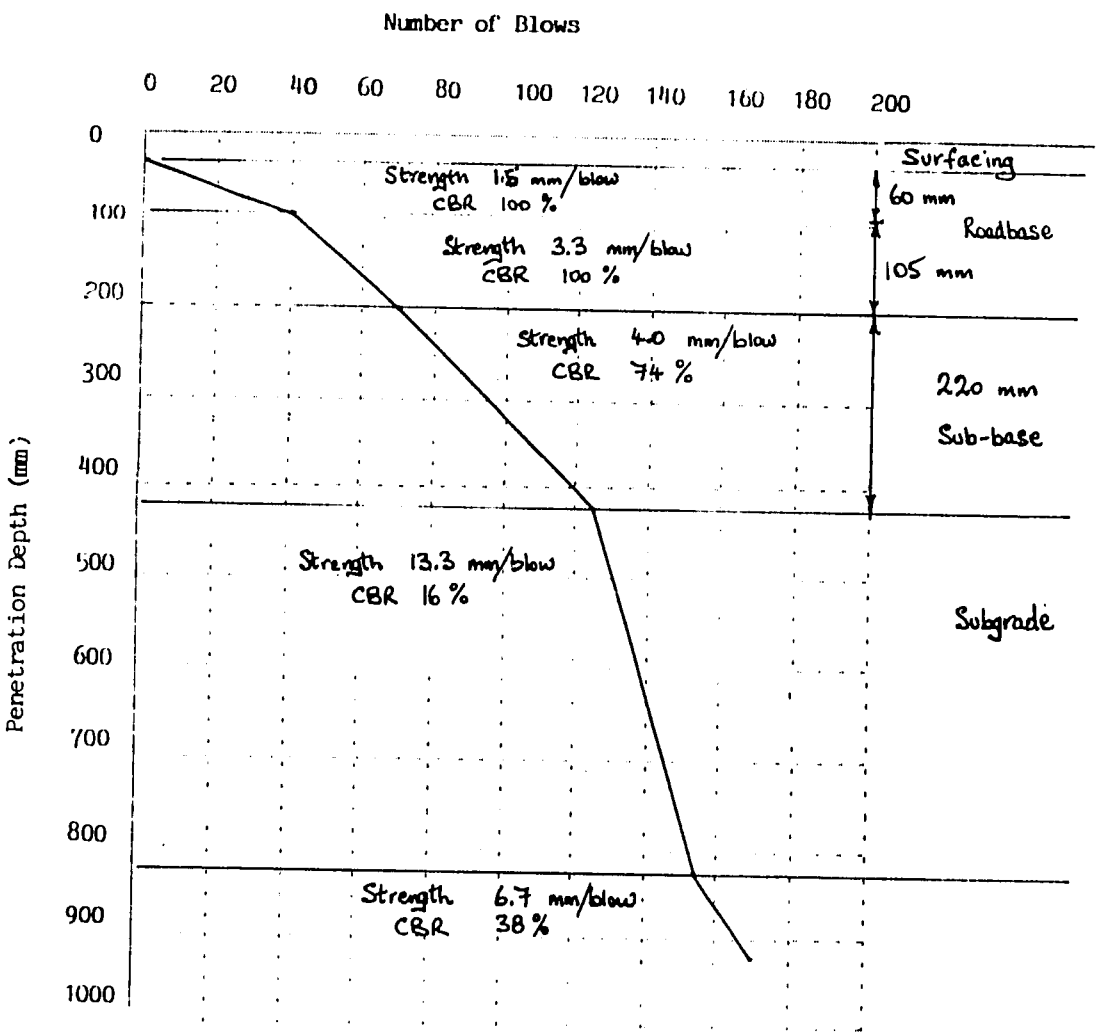
Date : 20-5-89

Carriageway : Northbound

Surface Thickness: 35 mm

Position : Outside Wheeltrack

Surface Condition: Good



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 1

Chainage (km) : 2.2

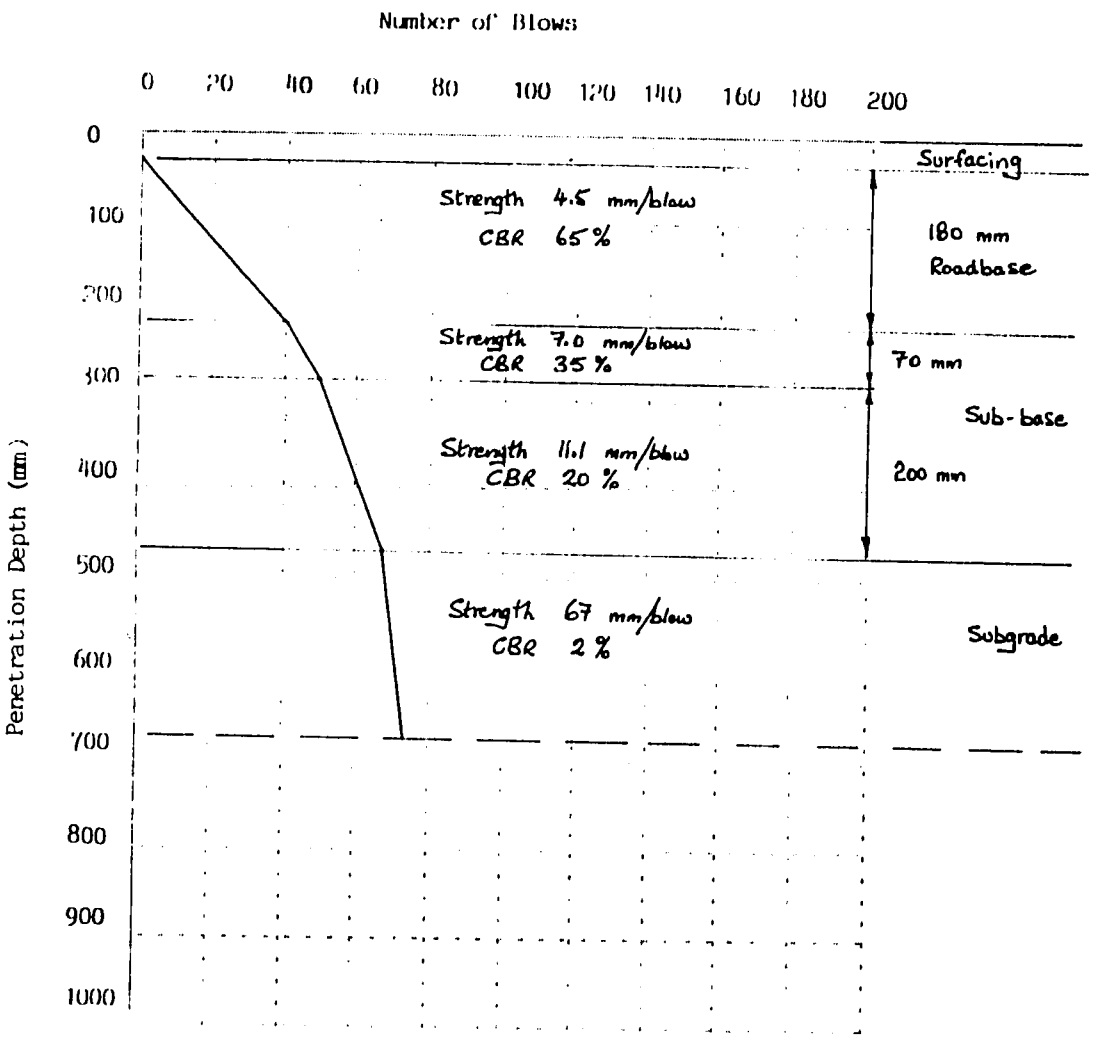
Date : 3-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Good



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 2

Chainage (km) : 4.5

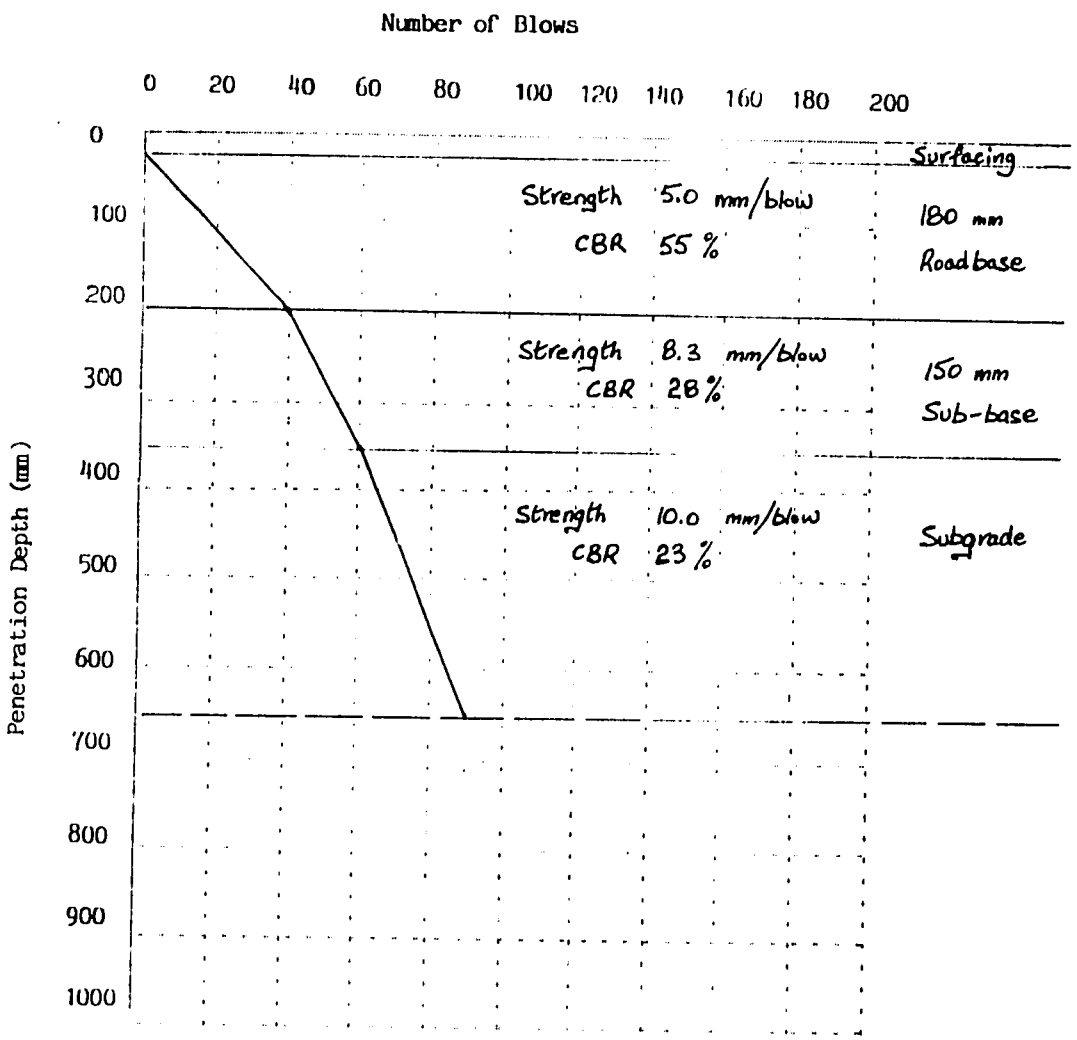
Date : 3-5-89

Carriageway : Southbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected Cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 3

Chainage (km) : 6.4

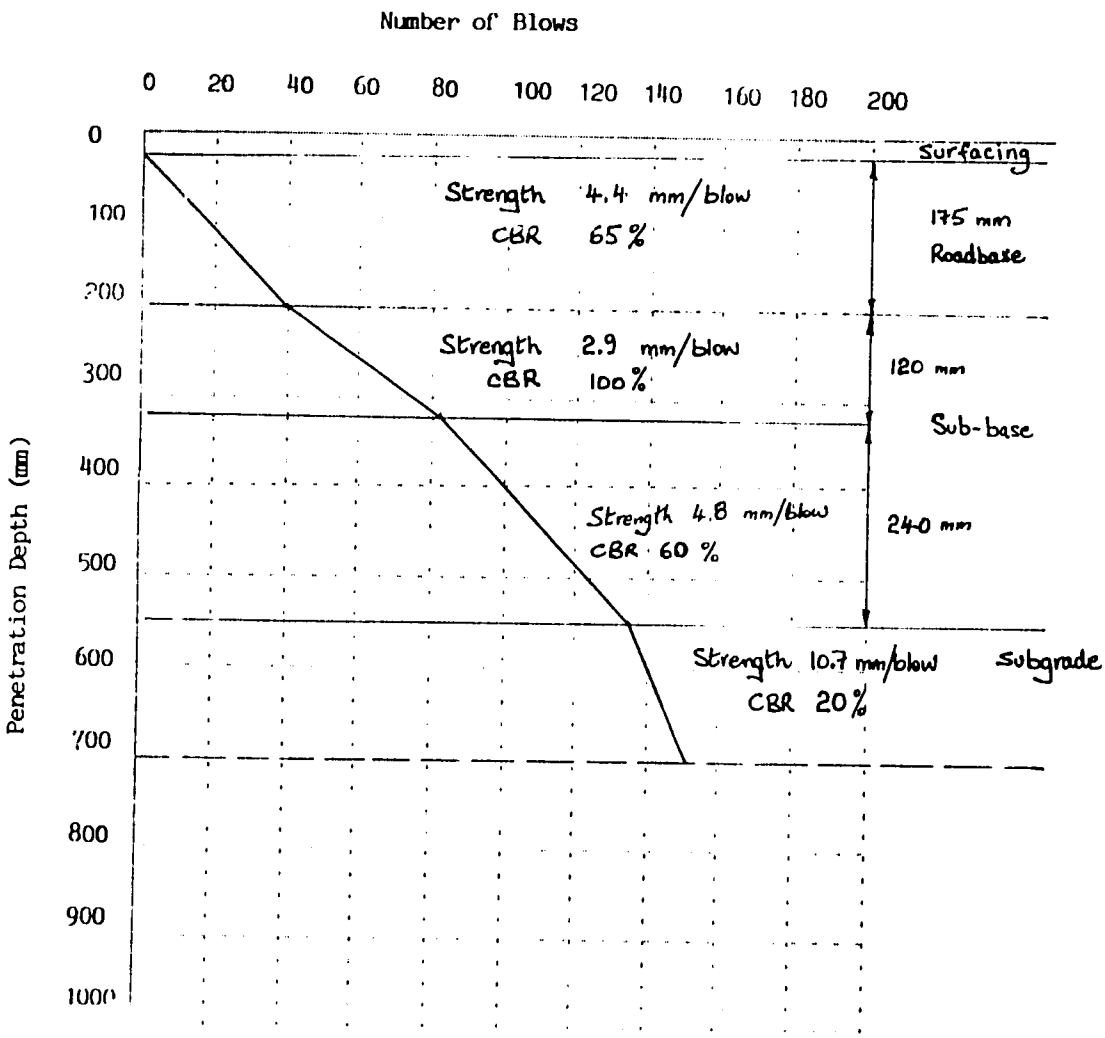
Date : 3 - 5 - 89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected Cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 4

Chainage (km) : 8.1

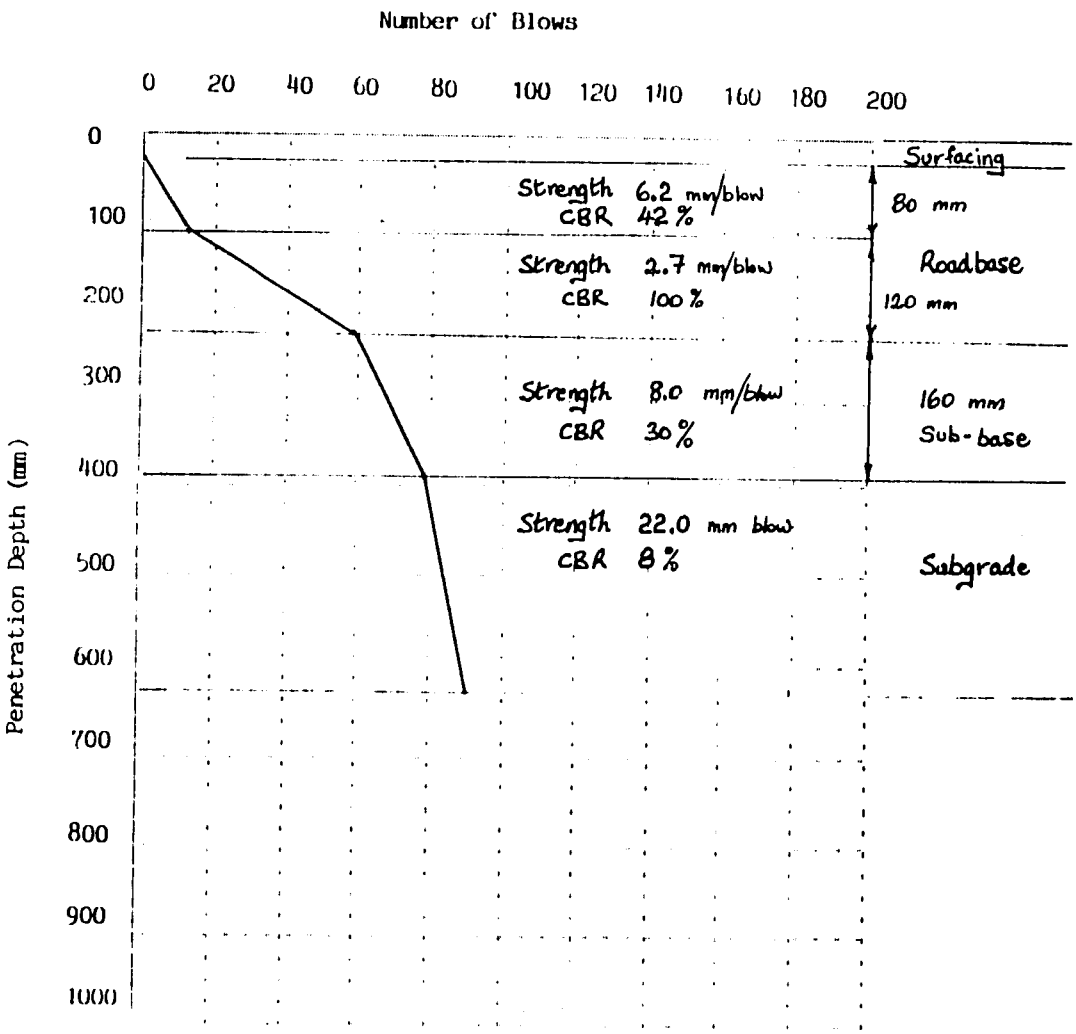
Date : 3-5-89

Carriageway : Southbound

Surface Thickness: 30 mm

Position : Outside Wheeltrack

Surface Condition: Single Crack



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 5

Chainage (km) : 9.9

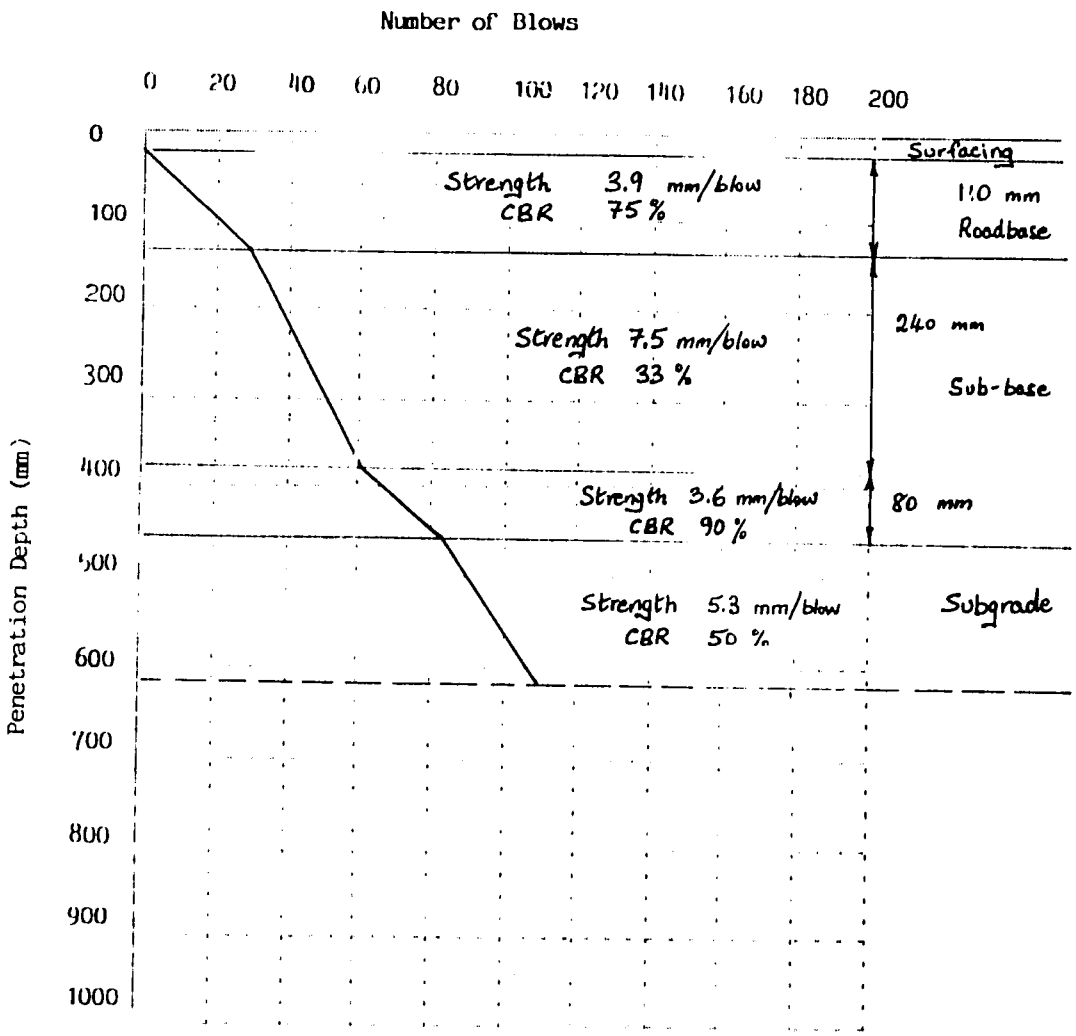
Date : 3-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Fine cracks



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA- KAFUE

Test N° : DCP 19

Chainage (km) : 10.5

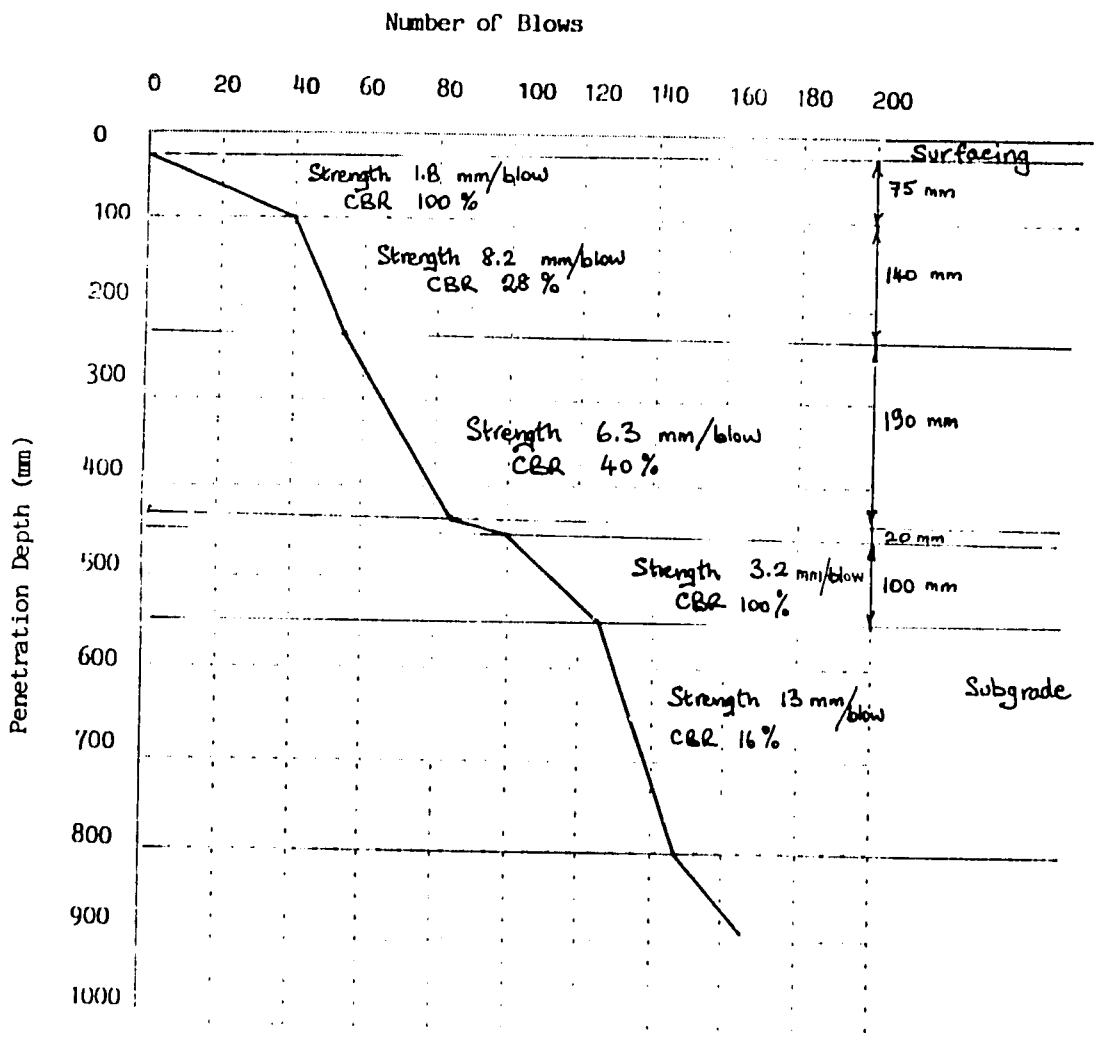
Date : 20-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected Cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 6

Chainage (km) : 12.0

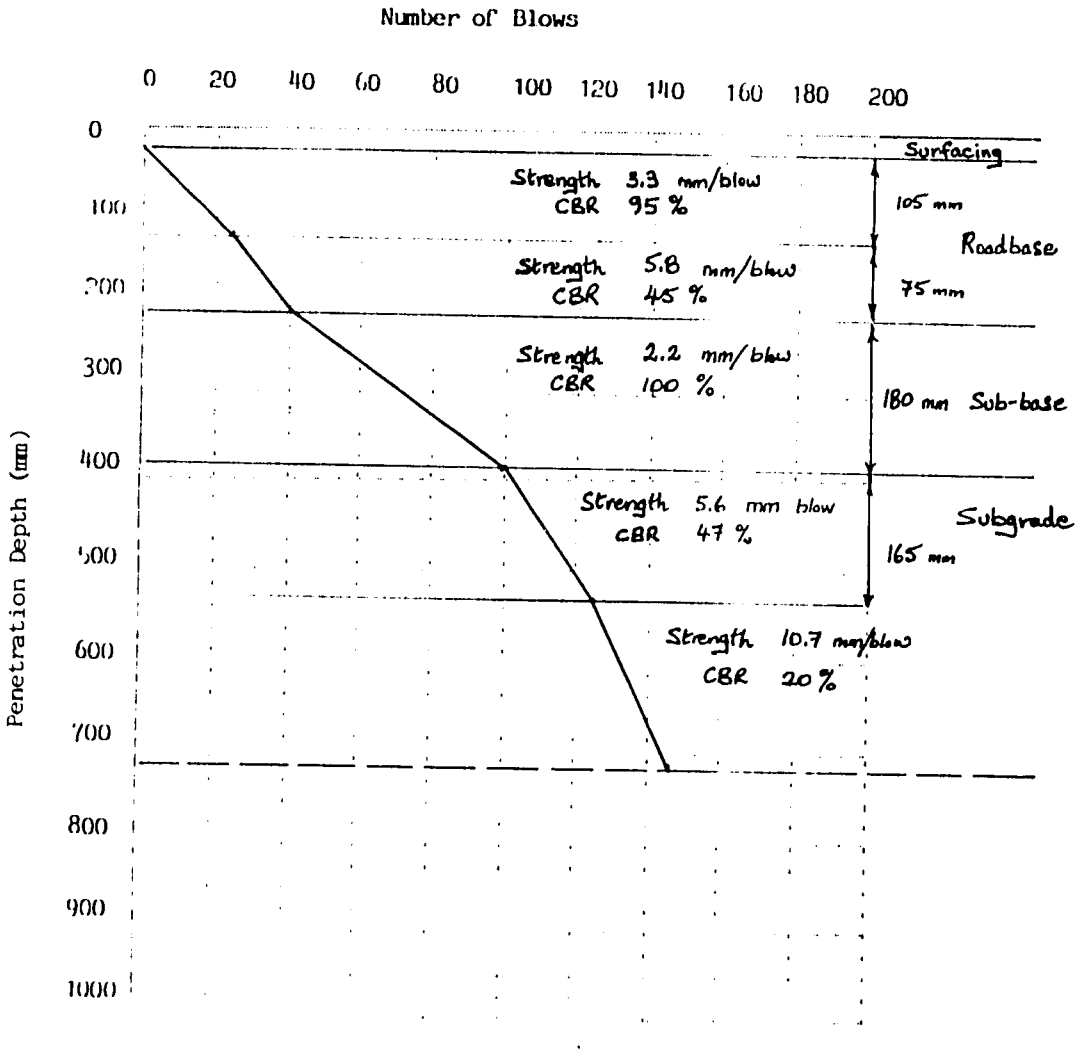
Date : 3 - 5 - 89

Carriageway : Southbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Fine cracks



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA- KAFUE

Test N° : DCP 7

Chainage (km) : 12.5

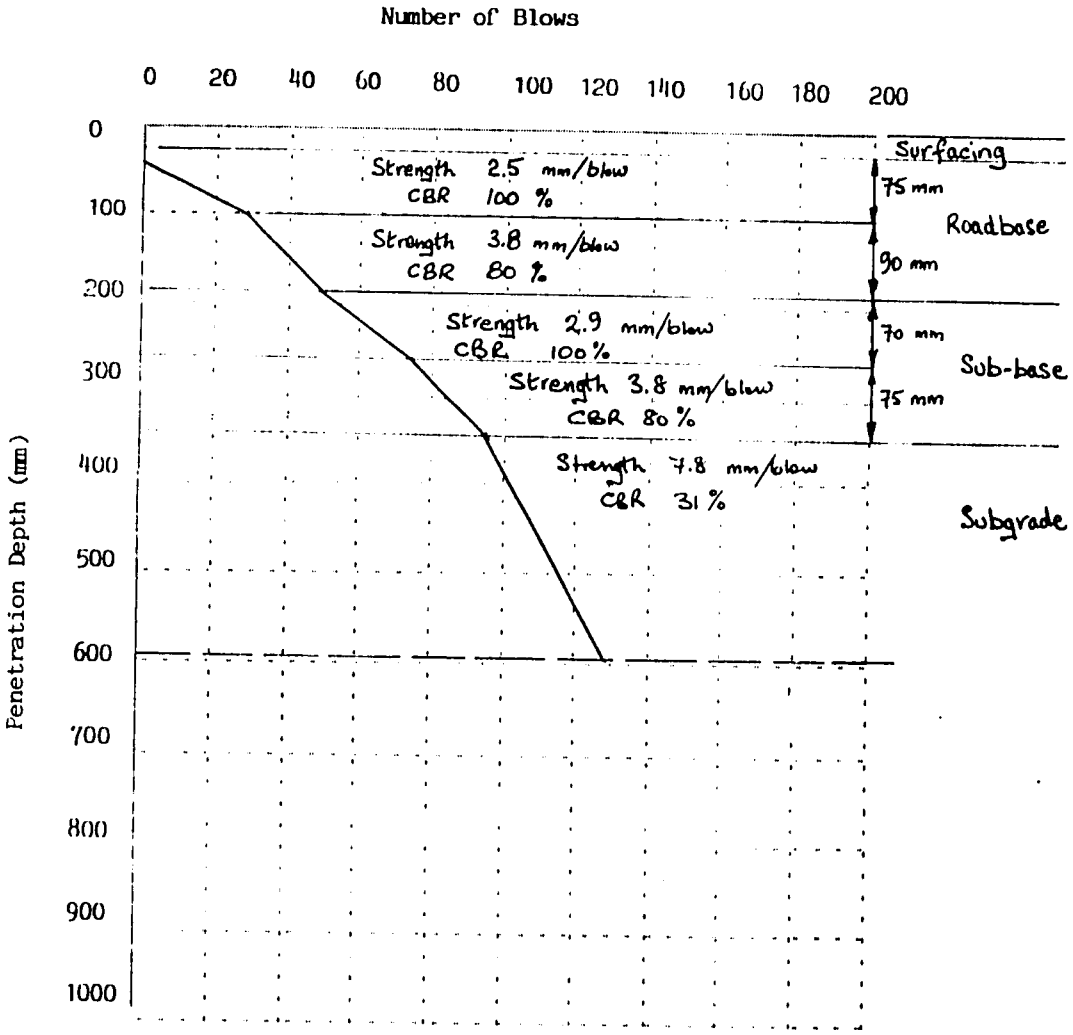
Date : 3-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Potholes



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA- KAFUE

Test N° : DCP 8

Chainage (km) : 13.7

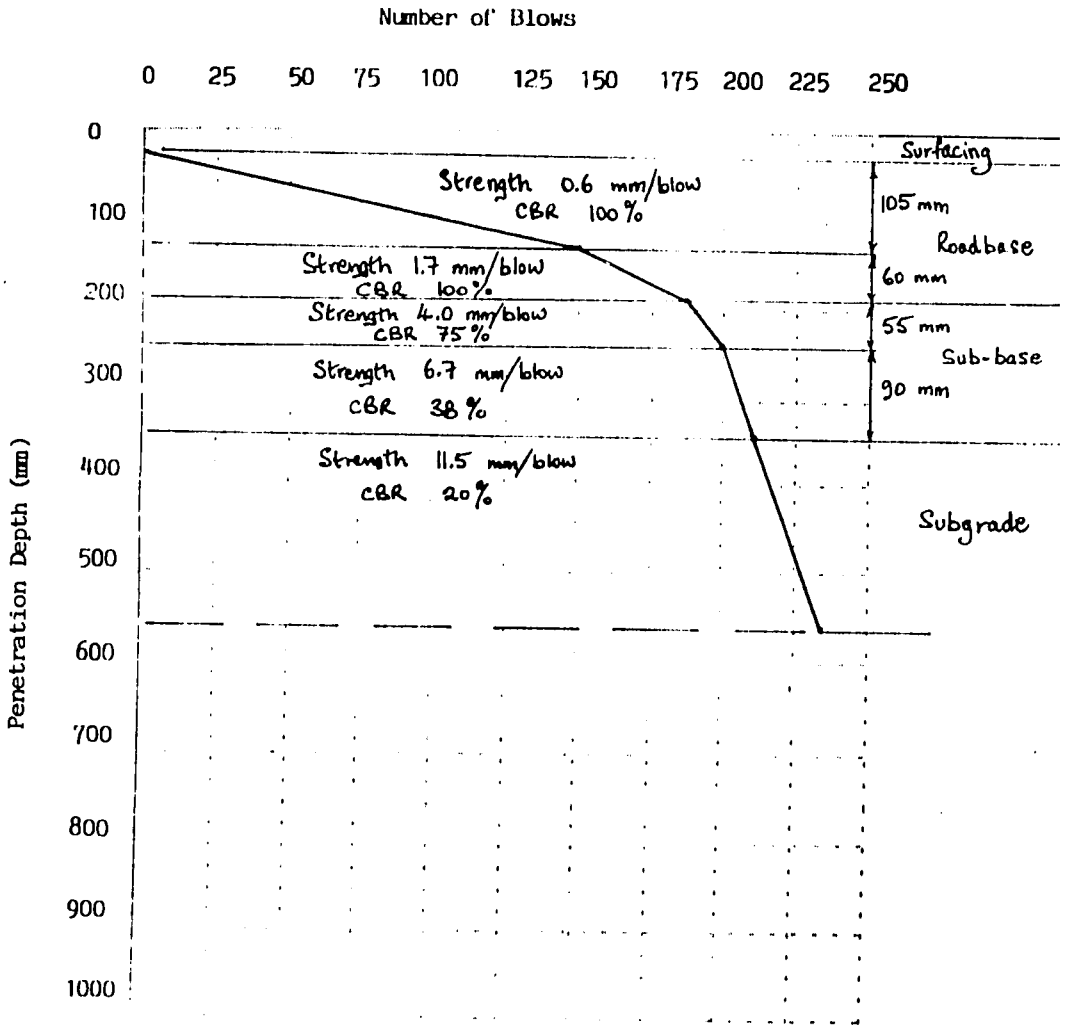
Date : 3-5-89

Carriageway : Southbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Good



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test No : DCP9

Chainage (km) : 16.9

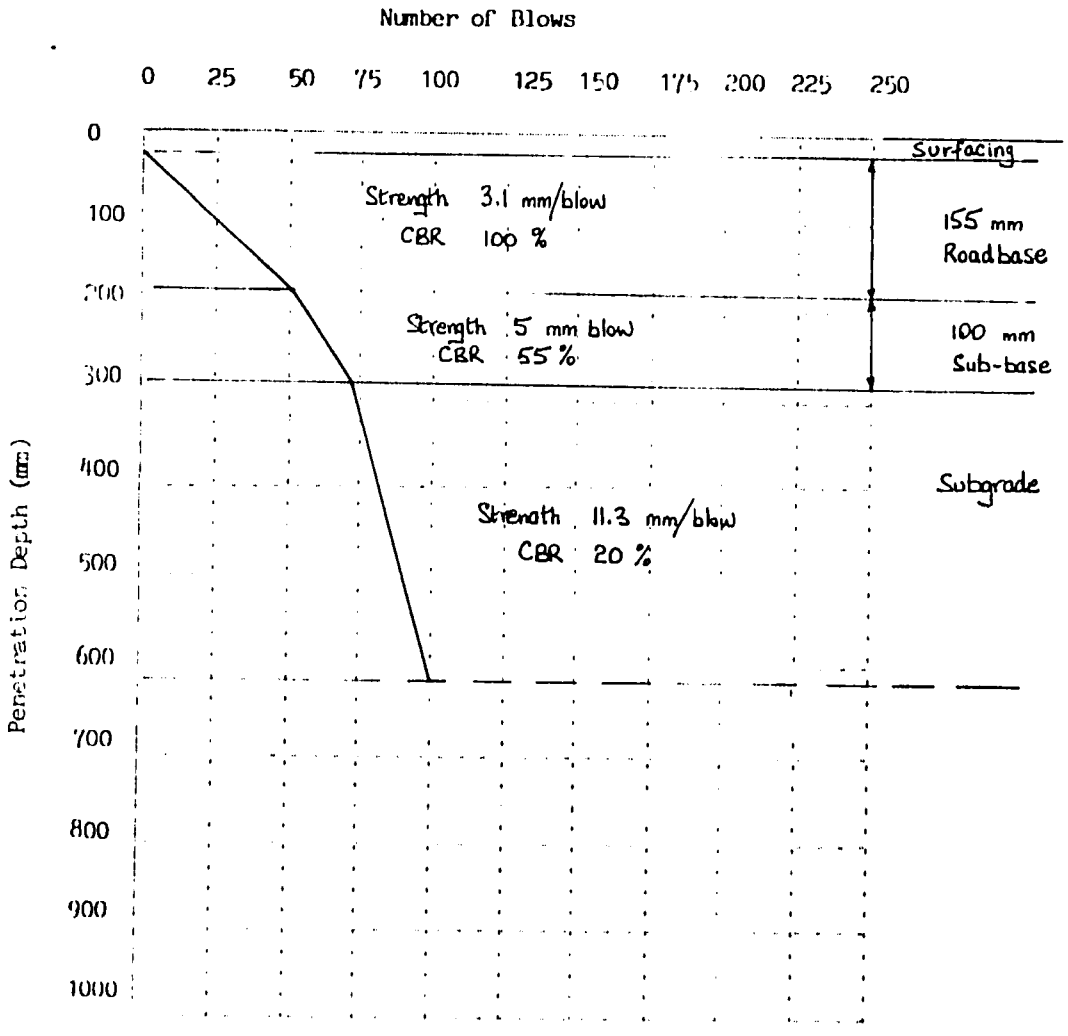
Date : 3-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Potholes



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test No : DCP 18

Chainage (km) : 18.2

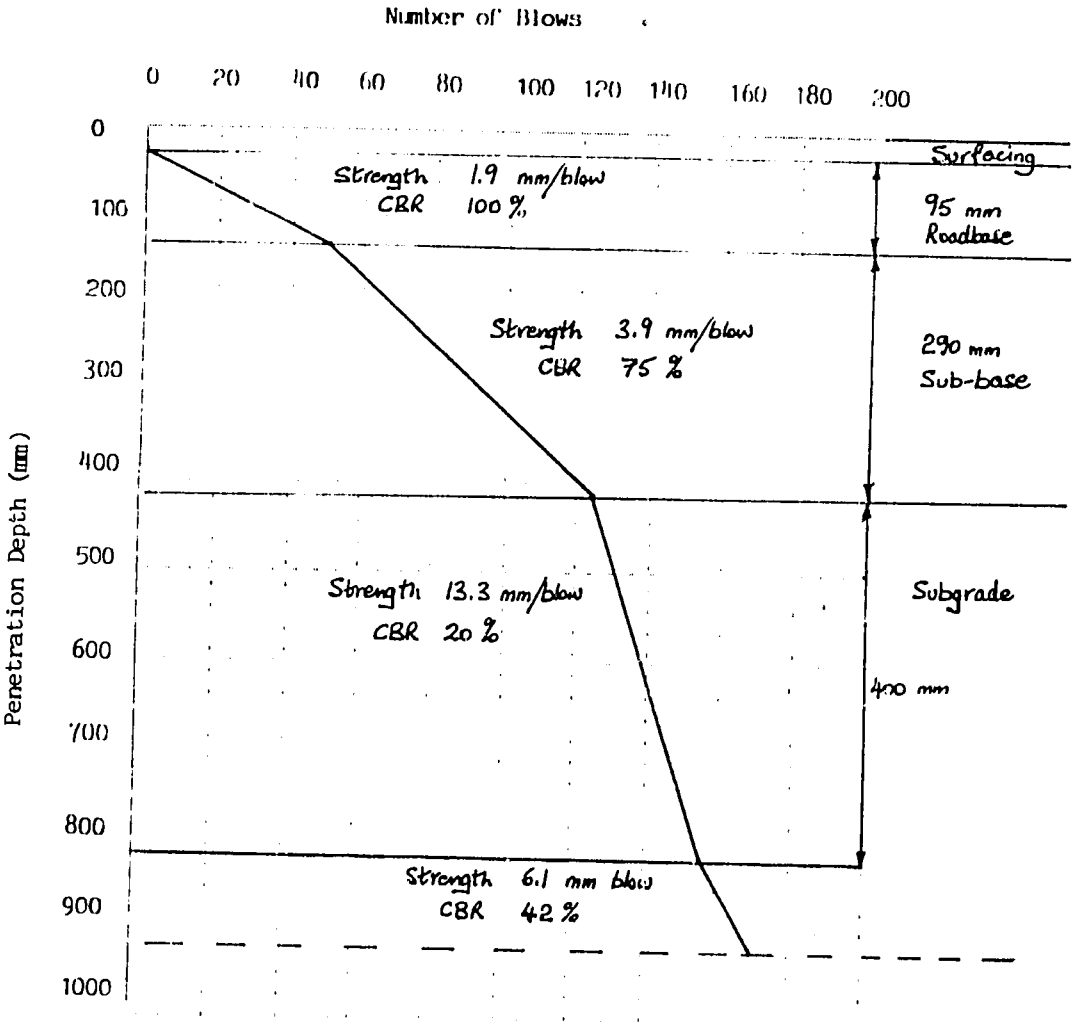
Date : 20-5-89

Carriageway : Northbound

Surface Thickness: 35 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected cracking.



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 13

Chainage (km) : 20.2

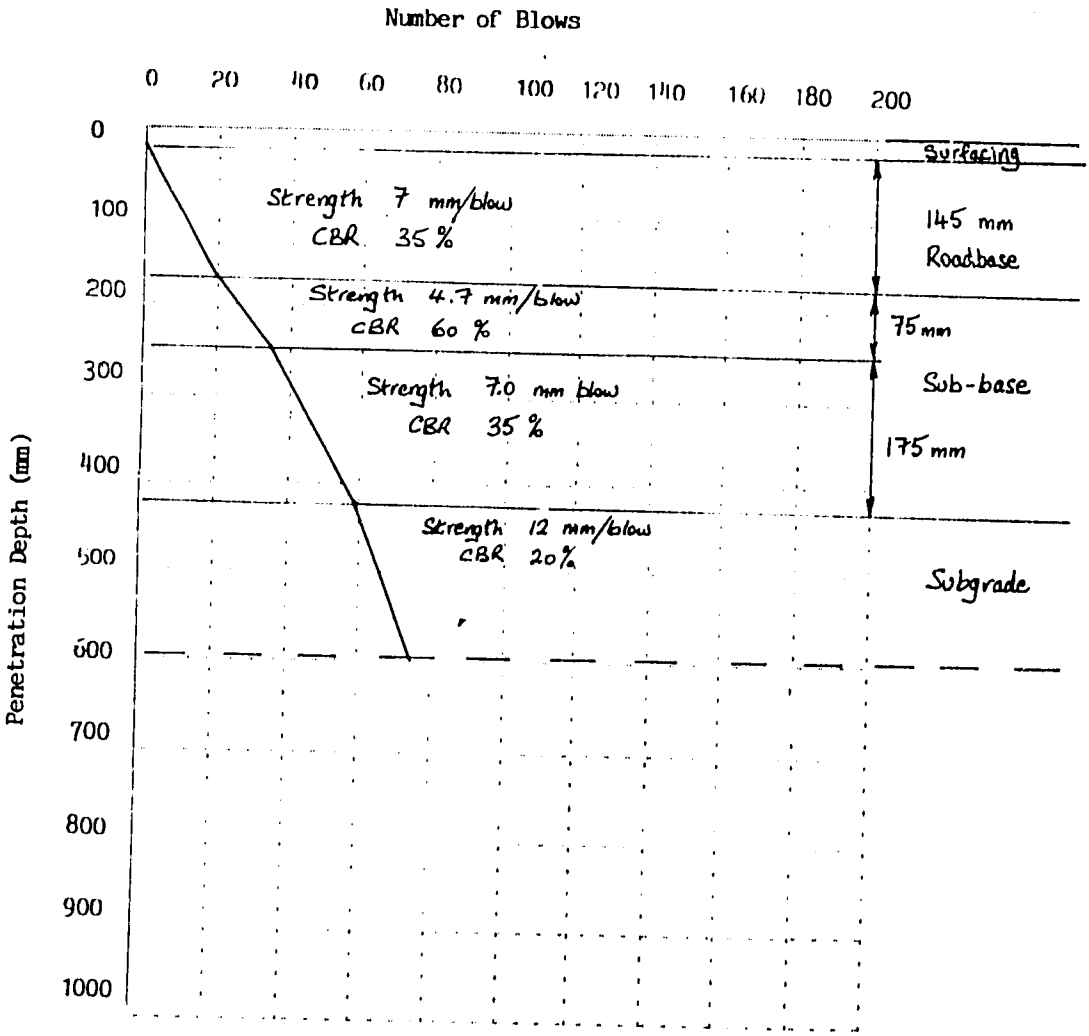
Date : 4-5-89

Carriageway : Southbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected Cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 14

Chainage (km) : 27.0

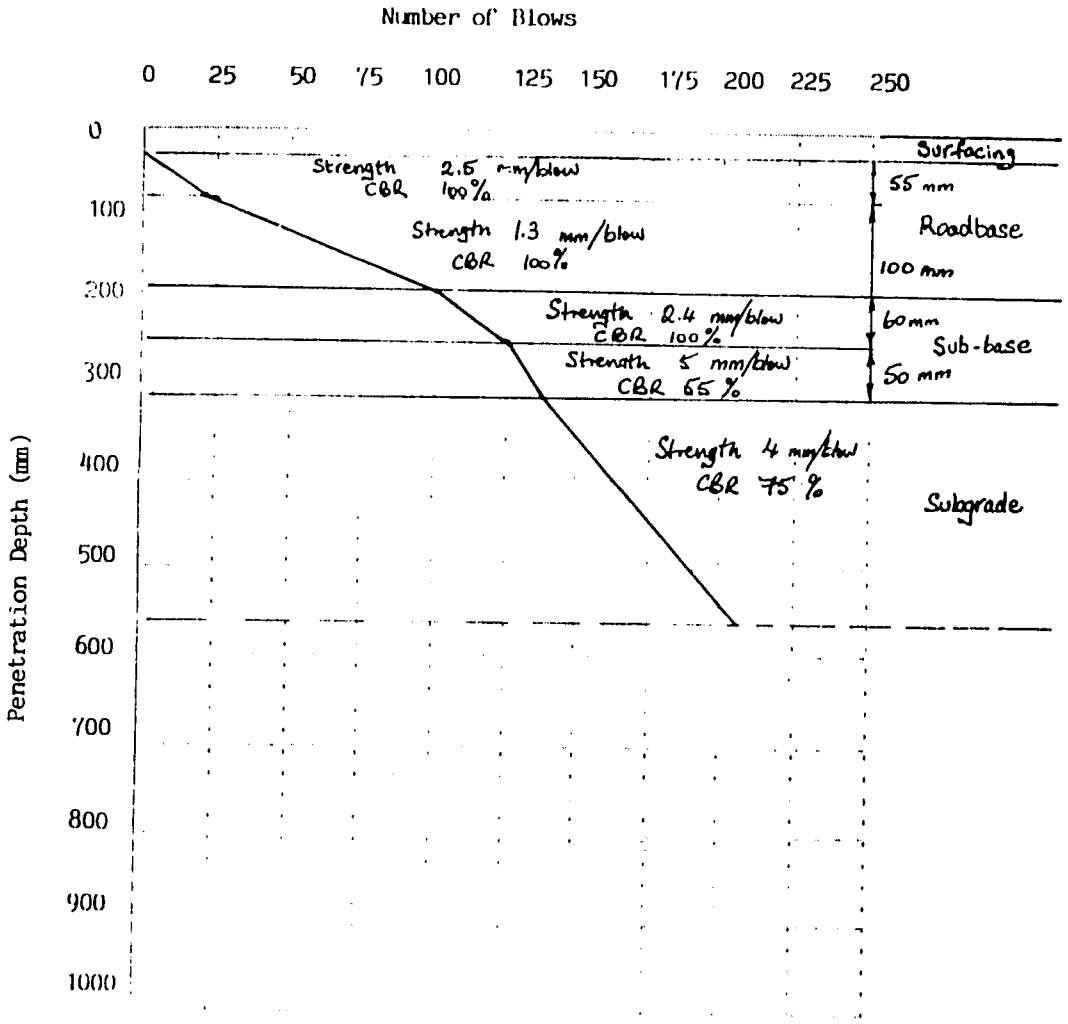
Date : 4-5-89

Carriageway : Northbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected Cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA- KAFUE

Test NO : DCP 20

Chainage (km) : 28.4

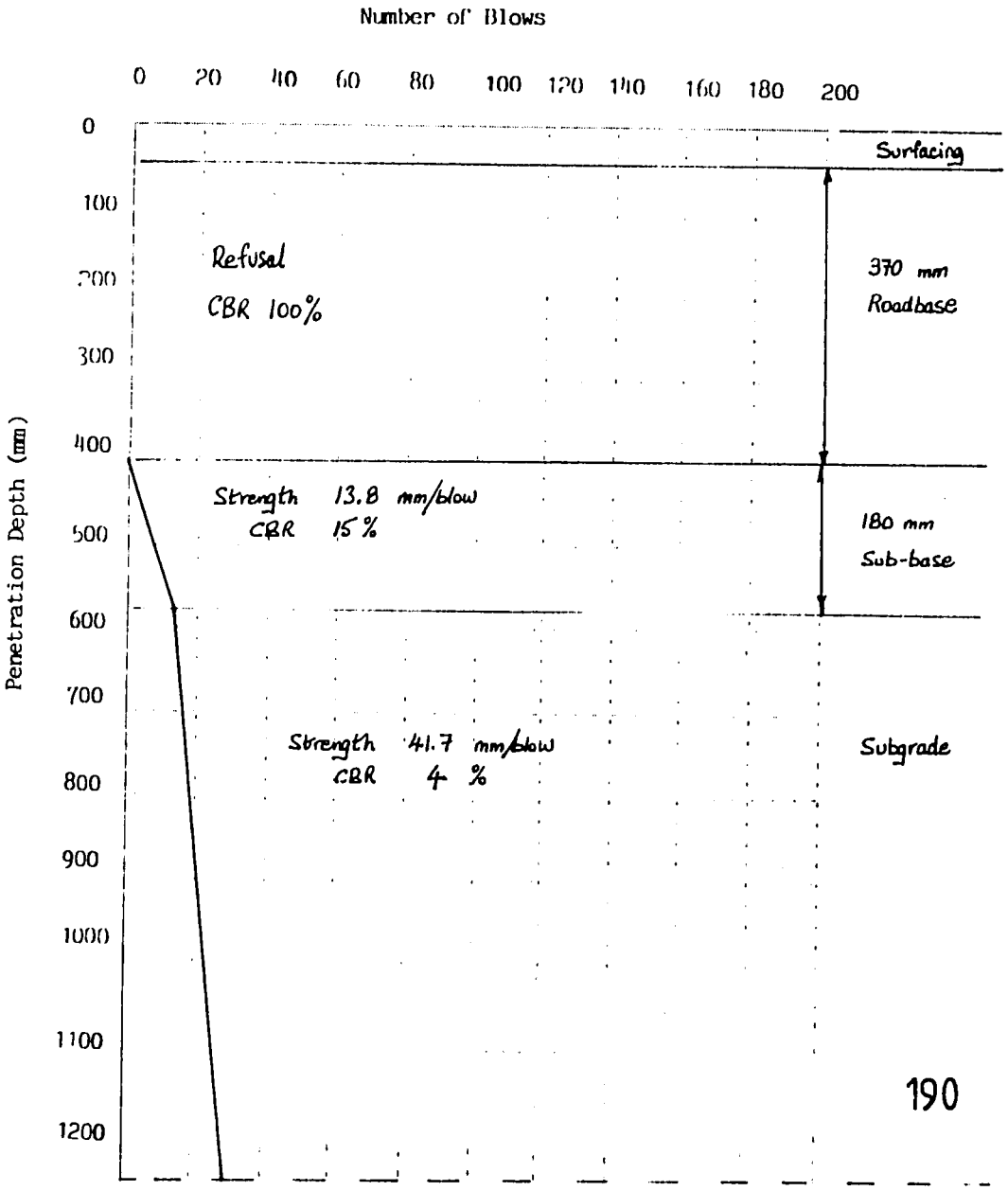
Date : 20-5-89

Carriageway : Northbound

Surface Thickness: 30 mm

Position : Outside Wheeltrack

Surface Condition: Interconnected cracking



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test NO : DCP 10

Chainage (km) : 28.8

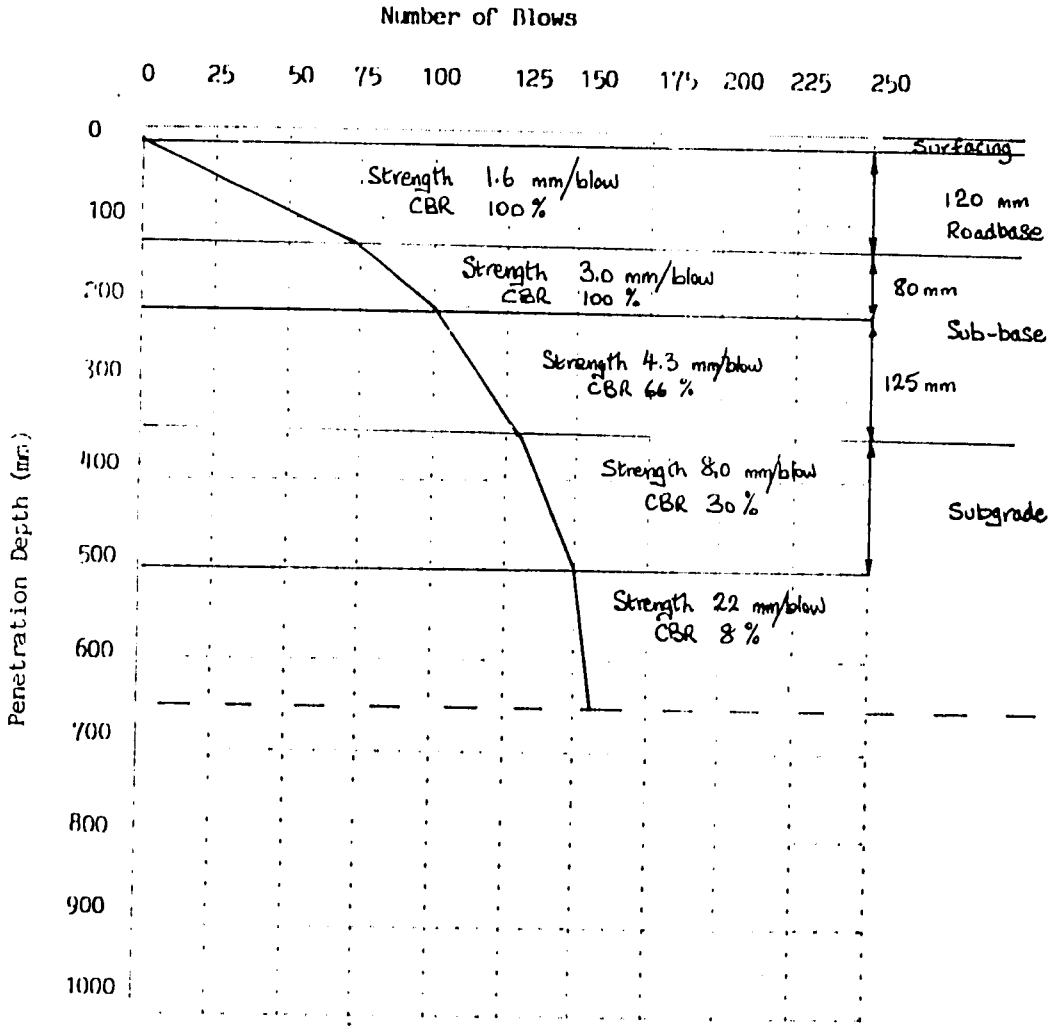
Date : 3-5-89

Carriageway : Southbound

Surface Thickness: 15 mm

Position : Outside Wheeltrack

Surface Condition: Good



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test NO : DCP 21

Chainage (km) : 39.5

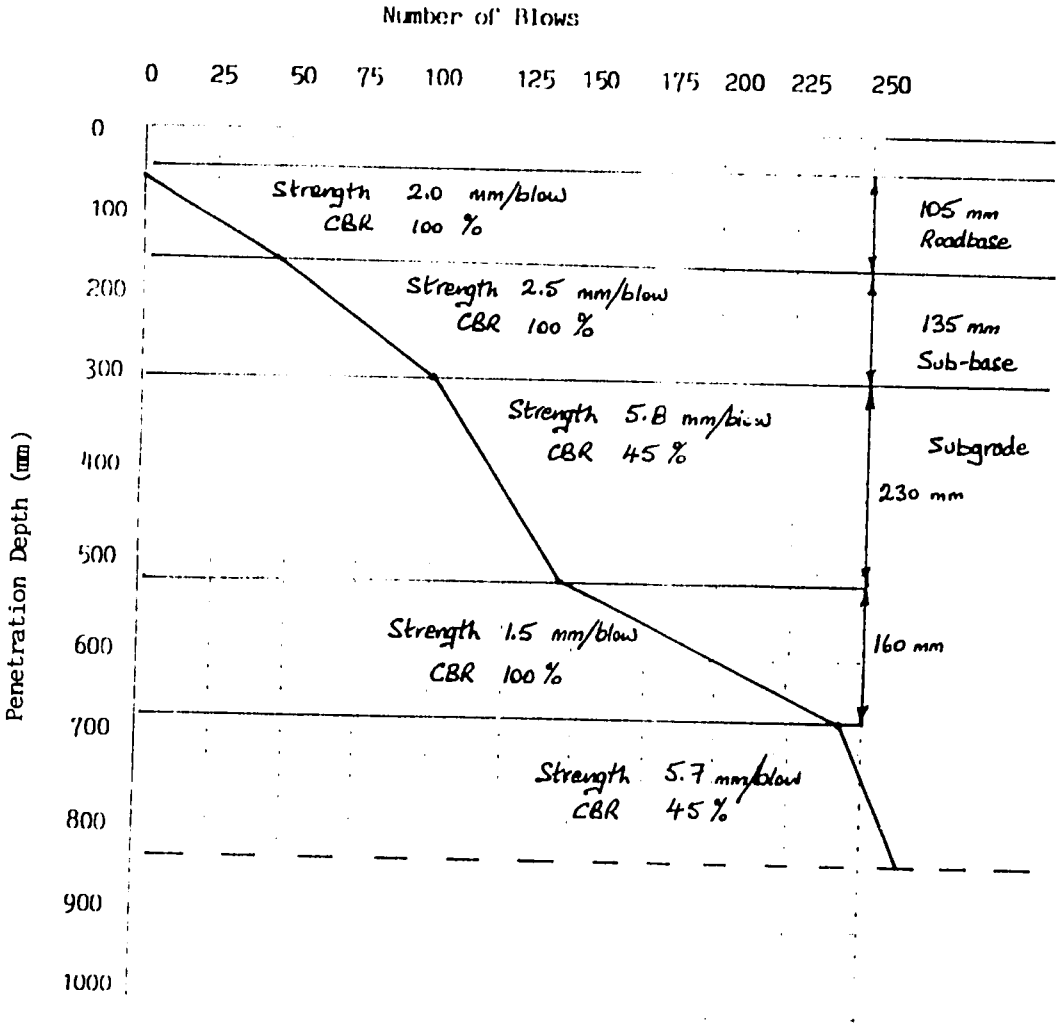
Date : 20-5-89

Carriageway : Northbound

Surface Thickness: 40 mm

Position : Outside Wheeltrack

Surface Condition: Good



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 12

Chainage (km) : 43.1

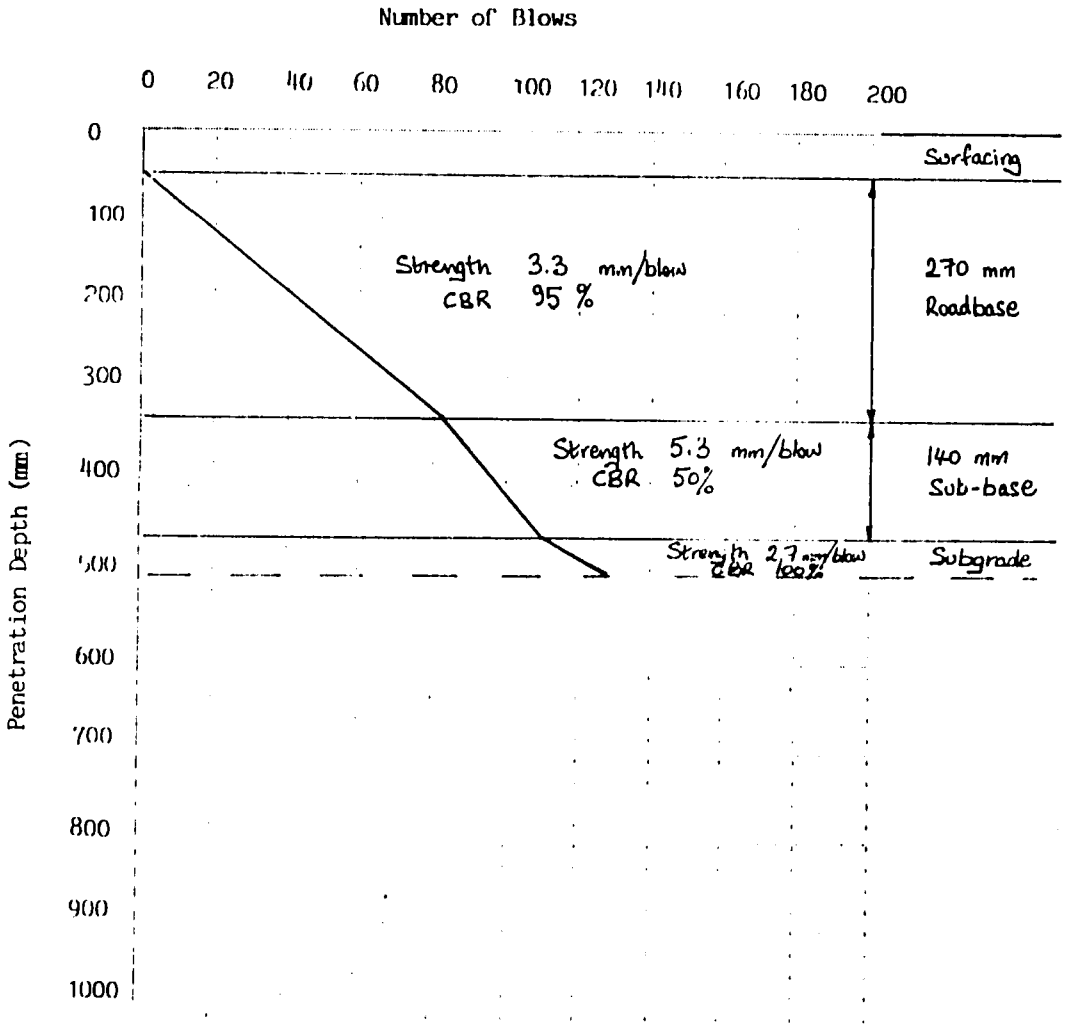
Date : 3-5-89

Carriageway : Southbound

Surface Thickness: 50 mm

Position : Outside Wheeltrack

Surface Condition: Potholes



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA – KAFUE

Test No : DCP 15

Chainage (km) : 45.5

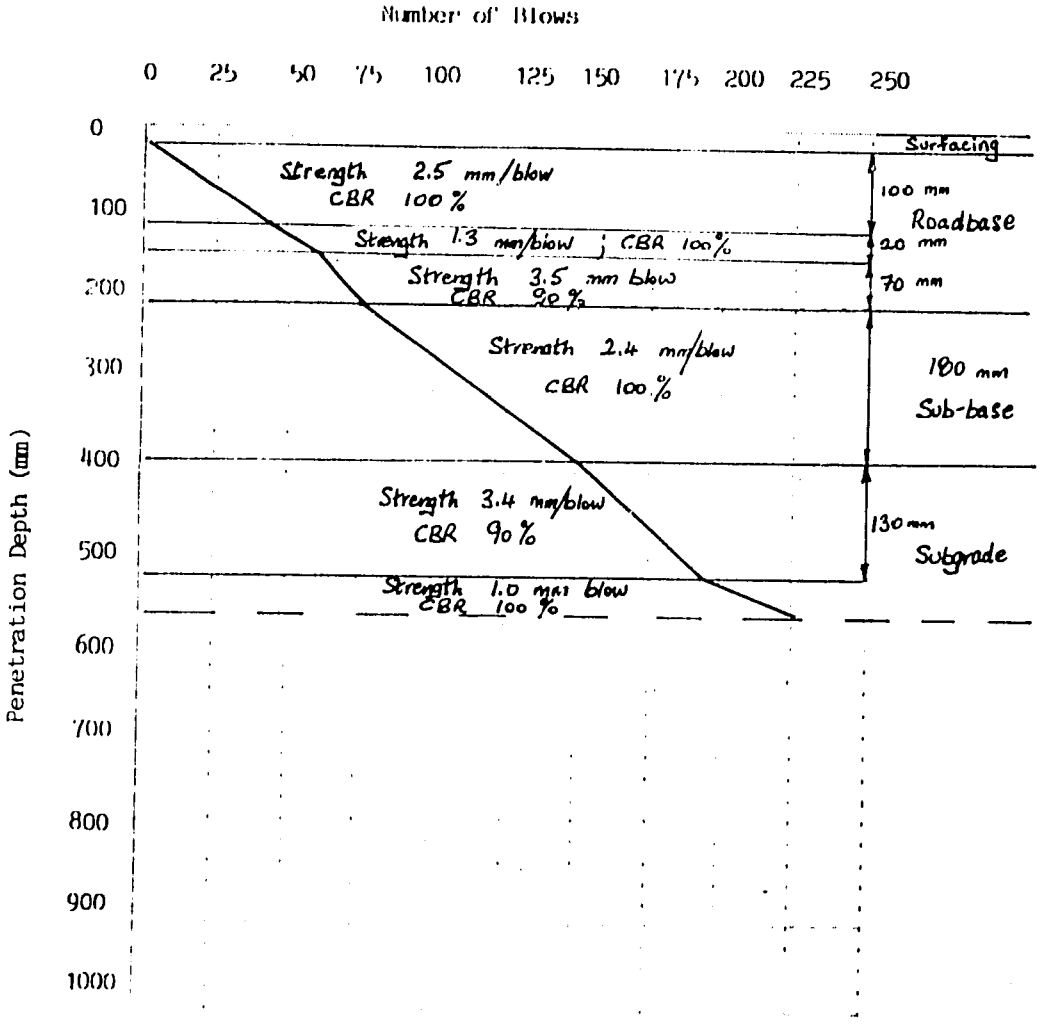
Date : 4-5-89

Carriageway : Northbound

Surface Thickness: 10 mm

Position : Outside Wheeltrack

Surface Condition: Potholes



DYNAMIC CONE PENETROMETER TEST

Road : LUSAKA - KAFUE

Test N° : DCP 16

Chainage (km) : 50.1

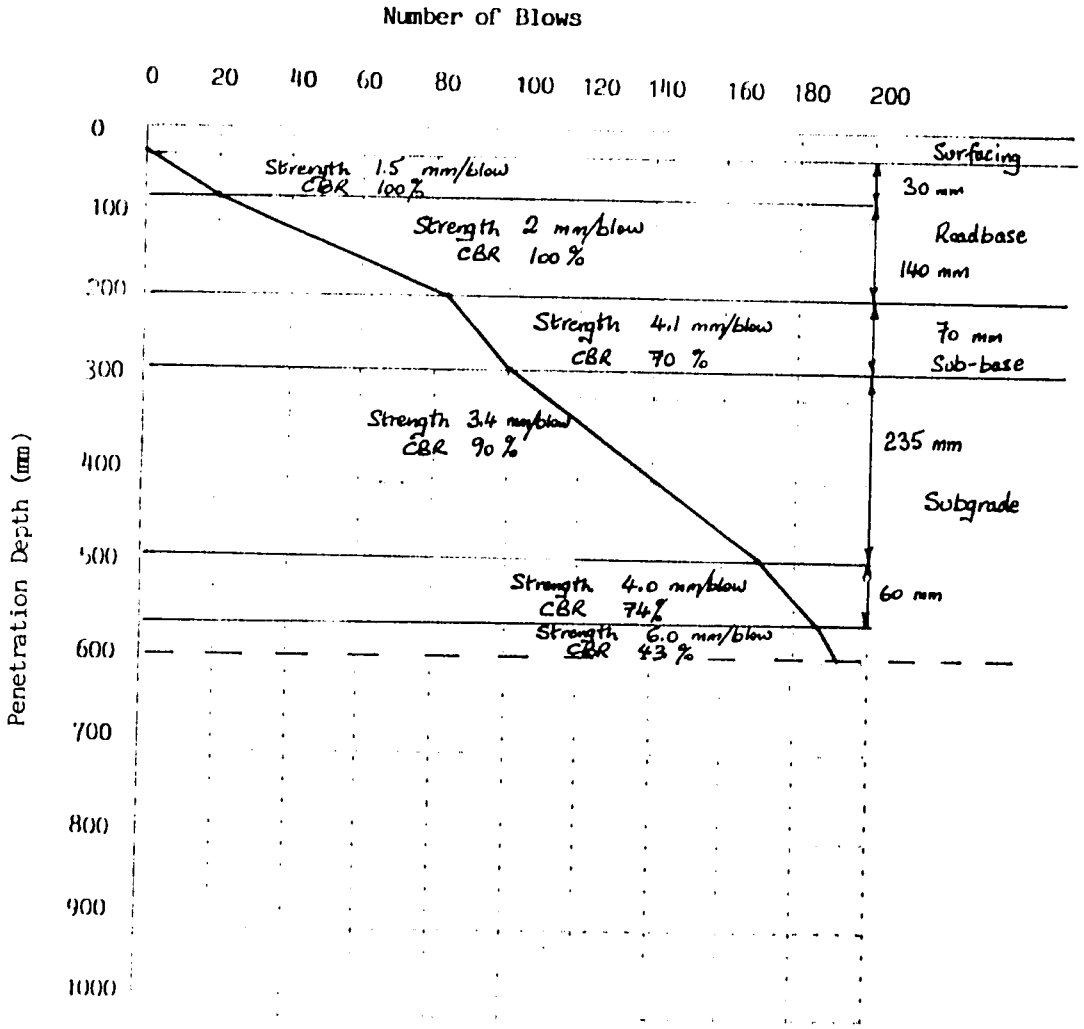
Date : 4-5-89

Carriageway : Southbound

Surface Thickness: 25 mm

Position : Outside Wheeltrack

Surface Condition: Potholes



BACKGROUND INFORMATION

The DCP programs discussed here are based on original work done by Mr E G Kleyn of the Transvaal Roads Department (TRD). (See list of references at the back of this document). Later work, using some of these original work, as well as adding additional calculations and new concepts was undertaken by Mr M De Beer of the Division of Roads and Transport Technology, CSIR, Pretoria. The most important aspect of the latest work is the development of a DCP-classification system. This system is used to classify DCP data for relatively lightly cementitious and granular base thin surfaced flexible pavements. See De Beer et al., 1992, for a description of the classification system.

These computer programs automatically classify the DCP data using the calculated relevant parameters, A and B, and also calculates:

- average penetration rates in mm/blow

- pavement structural number, DSN_{800}, in blows, which is the actual number of blows to penetrate 800 mm (approximately 31,5 inches) from the surface of the pavement.

- CBR values derived from the following formula :

  If average penetration rate (DN) $\geq$ 2 mm/blow :

  then $CBR = 405,3 \times DN^{(-1,259)}$

  and if DN $\leq$ 2 mm/blow :

  then $CBR = (66,66 \times DN^{2}) - (330 \times DN + 563,33)$

- UCS values derived from the following formula :

  $UCS = 15 \times CBR^{0,88}$

- Structural capacity in million E80s derived from the following formula :

  Million Standard Axles (MISA) = $Cm \times (DSN_{800})^{3,5} \times 10^{-9}$
  where Cm = moisture condition = 6,5 for wet
  = 14,0 for moist
  = 30,0 for optimum (omc)
  = 64,0 for dry conditions

  These Cm values are automatically taken into account when selecting the general moisture condition during data input phase, see Section 4.1.

- Balance Number at 100 mm depth, BN_{100}, which is the DSN percentage at a depth of 100 mm of the pavement.

- Standard Pavement Balance Curve (SPBC) B-value for the data, which is the B-value describing the best fit standard pavement balance curve for the data, as is indicated in the second graph on Figures 3 and 6.

- Graphical output of penetration rates

- A Normalised curve, which is a summary of the deviation in terms of area versus depth of the DCP-data from the best fit Standard Pavement Balance Curve (SPBC). Maximum and minimum peaks on this curve are indicative of layer interfaces and give the depths where layers of different penetration rates are encountered. Basically the peaks are indicative of the interface between a relatively softer layer (high penetration rate) and a relatively harder layer (low penetration rate) or vice versa. These peaks are then used to redefine the depth and thickness of the different layers on the basis of relative strength in terms of penetration rate.

- Lastly, a layer strength diagram of the redefined layers is given.

MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA17.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TR17
DISTANCE : 1.5
POSITION : L | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS | CROCK | LONG. | OTHER
DATE : [illegible]

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	154	
BALANCE NUMBER (BN100)	22	29
DIFFERENCE IN BN100	-7	

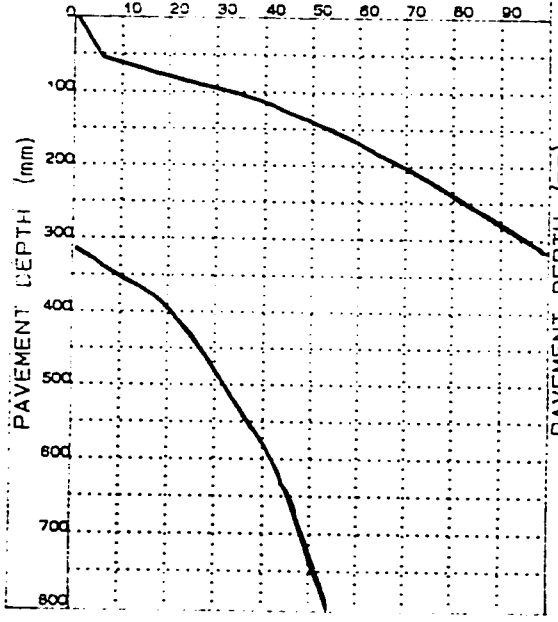
BALANCE CURVE IS WHERE B25 A= 2767
STRUCT. CAP. (E80x10**6) .36*
ROAD CATEGORY A :
TRAFFIC VERY HEAVY TRAFFIC
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0-375	4.1	2.4	8.1	69	614
376-800	13.0	4.3	20.0	16	172

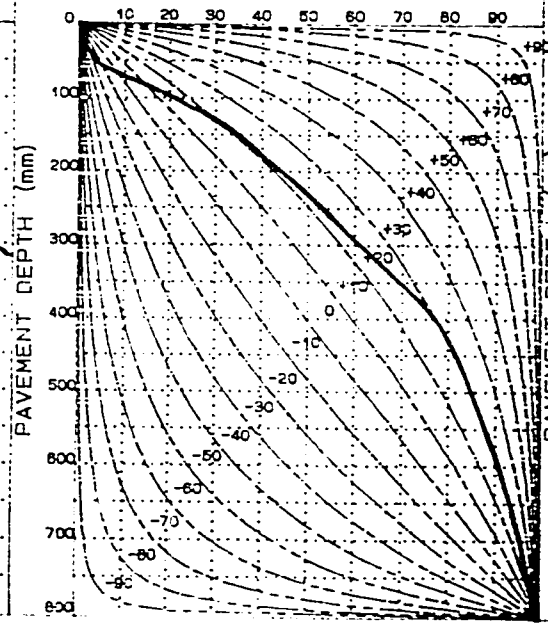
CATEGORY V : AVERAGE BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

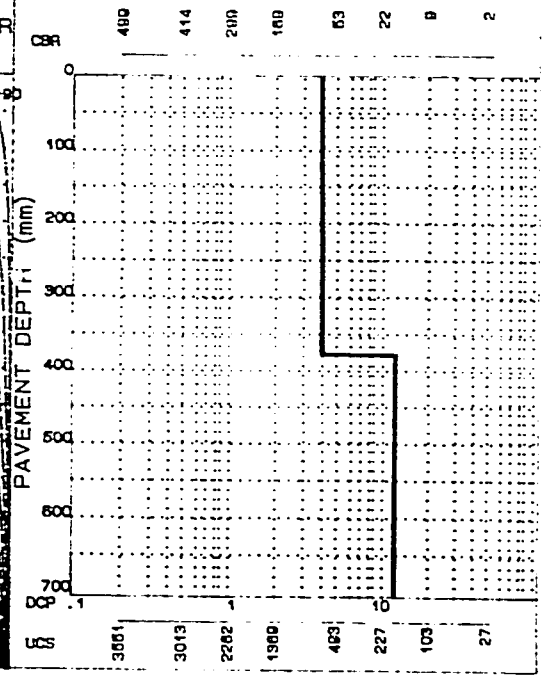


BALANCE CURVE

(-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA3.DAT
REGION : LUSAKA - CHILANGA
ROAD NUMBER : TP1
DISTANCE : 2.2
POSITION : L | > | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B. CURVE
STRUCTURE NUMBER	54	
BALANCE NUMBER (BN100)	17	26
DIFFERENCE IN BN100	9	
BALANCE CURVE IS WHERE B	26	A= 2657
STRUCT. CAP. (E80x10**6)	1.26*	
ROAD CATEGORY	A:	
TRAFFIC	VERY HEAVY TRAFFIC	

* Structural capacity not reliable

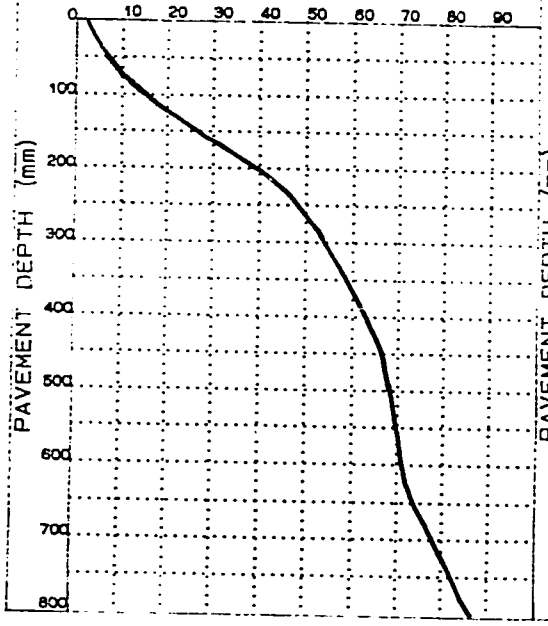
AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	BP	CBR	UCS
0-150	7.1	3.0	12.0	34	334
151-500	12.6	9.3	27.8	16	172
501-800	21.4	10.8	39.2	9	99

CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ASC

DCP CURVE

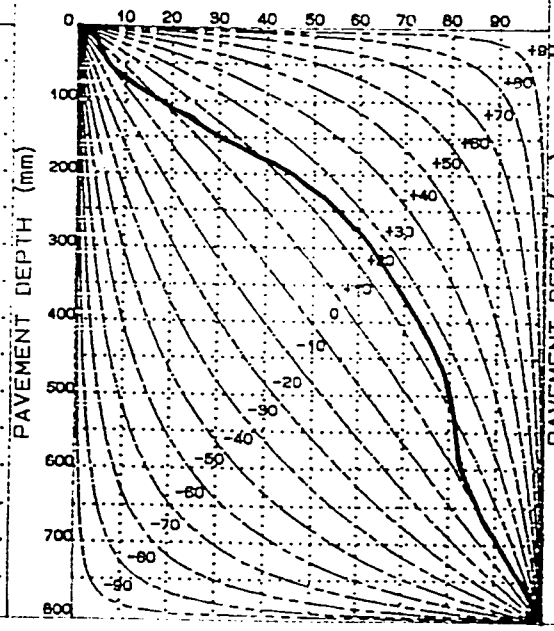
NUMBER OF BLOWS



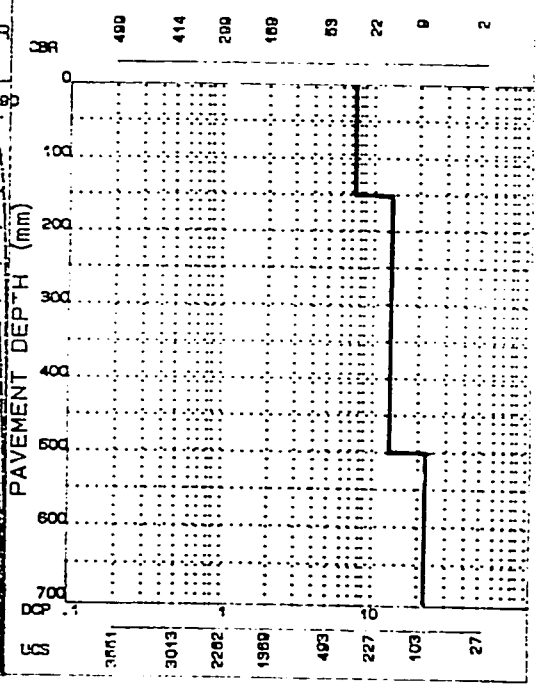
BALANCE CURVE

(DSN 80C) (-90 < B < +90)

% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT. CSIR. SA

DATA FILE : LSKA2.DAT
REGION : LUSAKA - CHILANGA
ROAD NUMBER : TP2
DISTANCE : 4.5
POSITION : L | | | M | | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	101	
BALANCE NUMBER (BN100)	14	21
DIFFERENCE IN BN100	-7	
BALANCE CURVE IS WHERE B	15	A= 1373
STRUCT. CAP. (E80x10**5)	.31*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

* Structural capacity not reliable

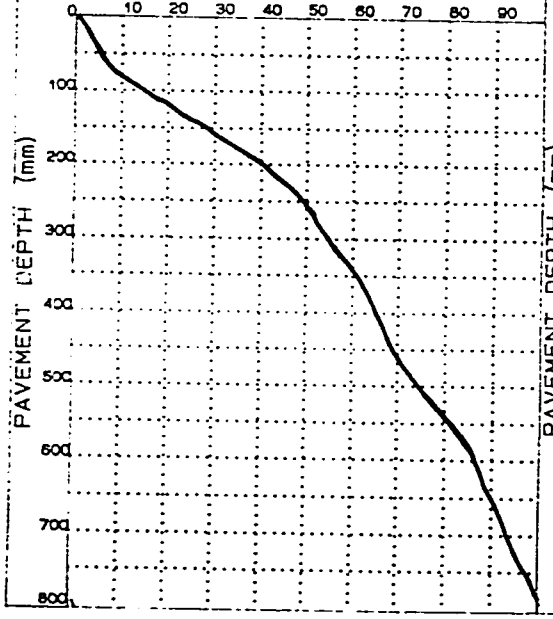
AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0-170	6.4	3.2	11.7	39	376
171-320	7.3	2.6	11.5	33	325
321-600	11.5	2.6	15.8	18	190

CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

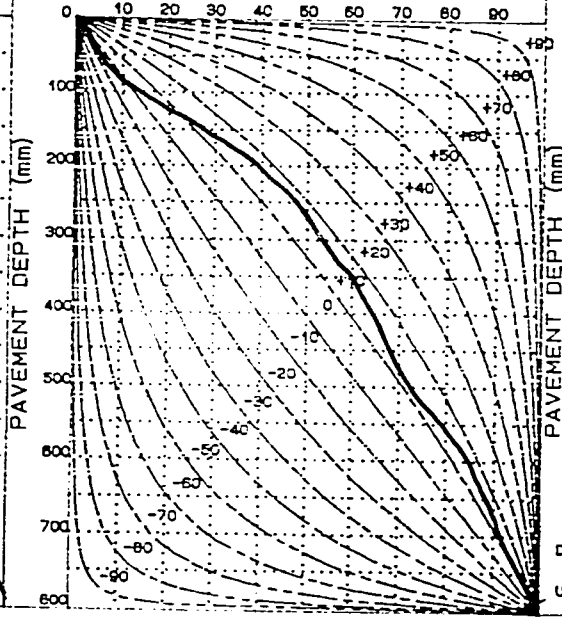
NUMBER OF BLOWS



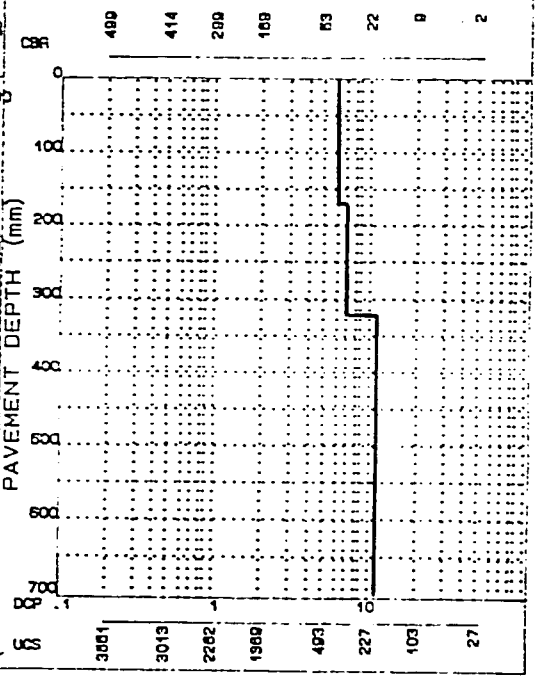
BALANCE CURVE

(DSN 600) (-90 < B < +90)

% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA3.DAT
REGION : LUSAKA - CHILANGA
ROAD NUMBER : TP3
DISTANCE : 6.4
POSITION : L | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	159	
BALANCE NUMBER (BN100)	1	18
DIFFERENCE IN BN100	-7	

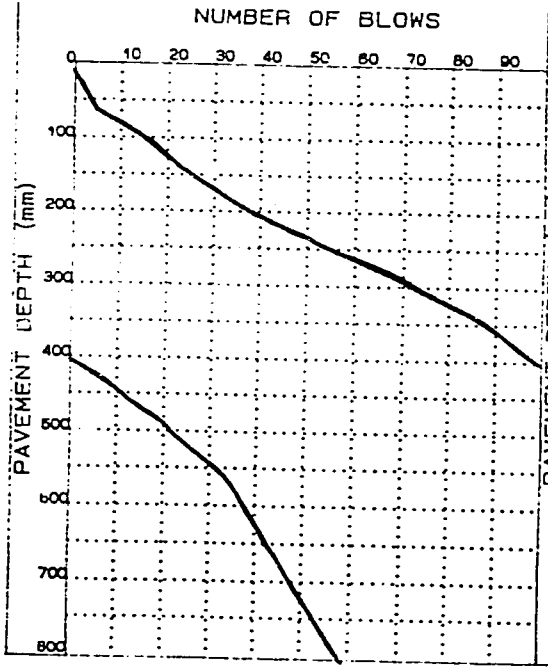
BALANCE CURVE IS WHERE B_1 = A= 3337
STRUCT. CAP. (280x10**6) .52*
ROAD CATEGORY A :
TRAFFIC : VERY HEAVY TRAFFIC
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95%P	CBR	UCS
0-500	4.8	2.2	8.3	58	534
501-540	5.3	0.3	5.7	50	469
541-800	9.7	1.3	11.7	23	236

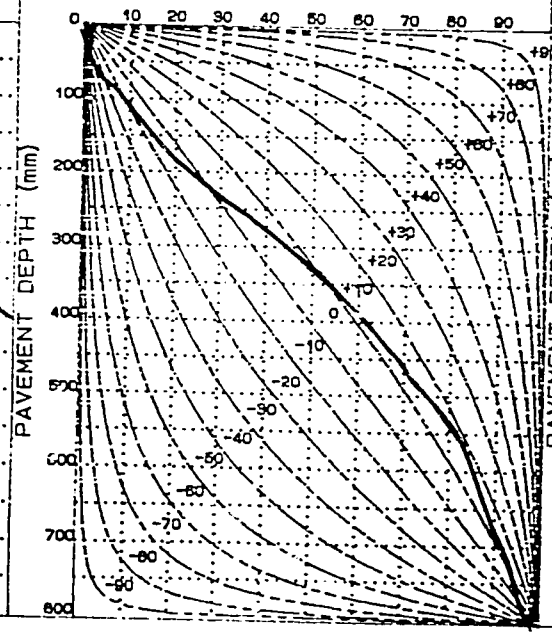
CATEGORY VI : POORLY BALANCED DEEP STRUCTURE (FSD)

DCP CURVE

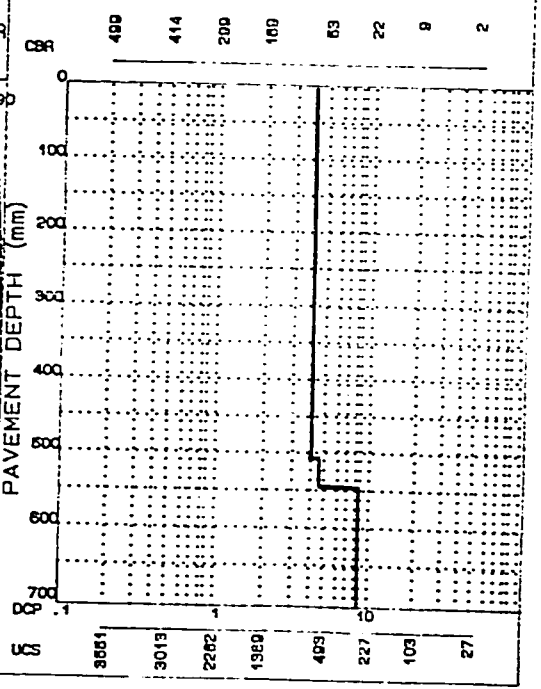


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE :LSKA4.DAT
REGION :LUSAKA - CHILANGA
ROAD NUMBER :TP4
DISTANCE : 6.1
POSITION : L | | | M | | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE :89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	97	
BALANCE NUMBER (BN100)	13	33
DIFFERENCE IN BN100	-20	
BALANCE CURVE IS WHERE B	30	
STRUCT. CAP. (E80x10**6)	0.27*	A= 3674
ROAD CATEGORY	A :	

TRAFFIC VERY HEAVY TRAFFIC

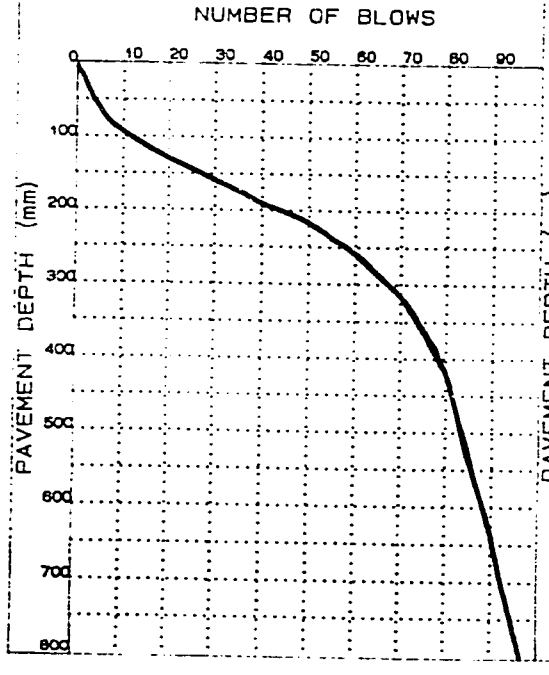
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0-220	5.9	3.8	12.2	43	410
221-310	5.2	0.7	6.4	51	477
311-800	20.1	5.5	29.1	9	103

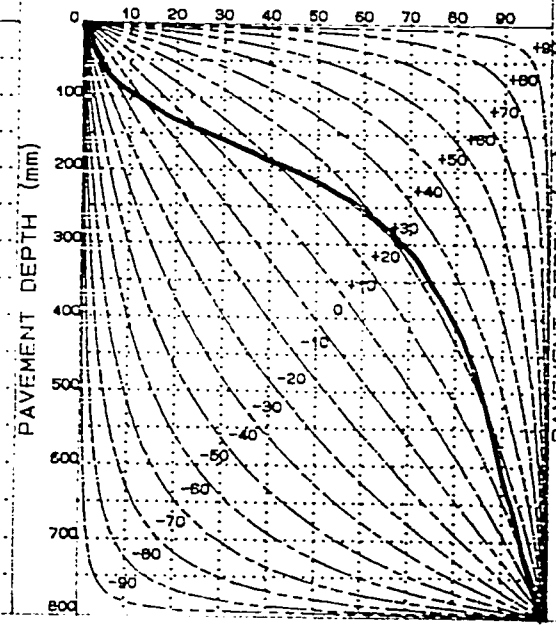
CATEGORY VI : POORLY BALANCED DEEP STRUCTURE (PBD)

DCP CURVE

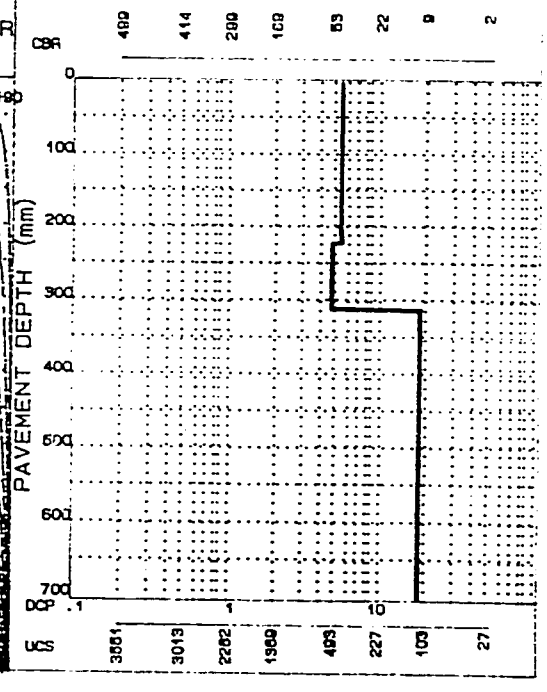


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSNA5.DAT
REGION : LUSAKA CHILANGA
ROAD NUMBER : TP5
DISTANCE : 9.5
POSITION : L [X] M R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	148	
BALANCE NUMBER (BN100)	13	12
DIFFERENCE IN BN100	:1	
BALANCE CURVE IS WHERE B-Z A=	928	
STRUCT. CAP. (E80x10**6)	.18*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

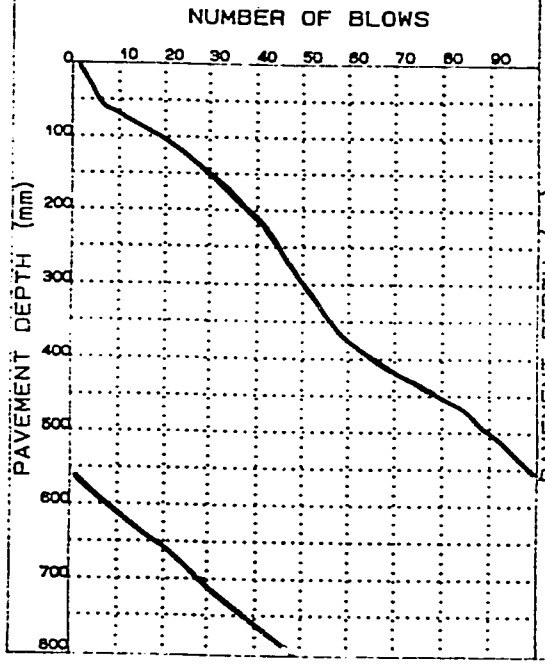
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	σ	CBR	UCS
0- 95	7.0	3.2	12.2	34	334
96-215	5.8	0.8	6.9	46	435
216-315	8.7	0.5	9.5	26	283
316-375	7.0	1.0	8.7	34	334
376-800	5.0	0.7	6.2	54	501

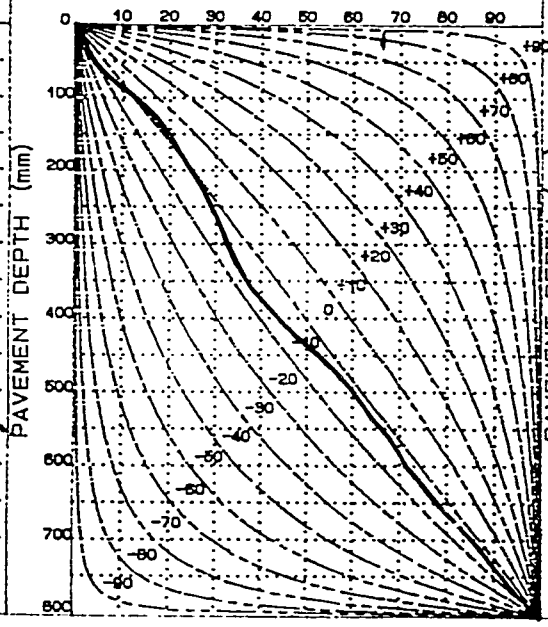
CATEGORY VII : WELL-BALANCED INVERTED STRUCTURE (WBI)

DCP CURVE

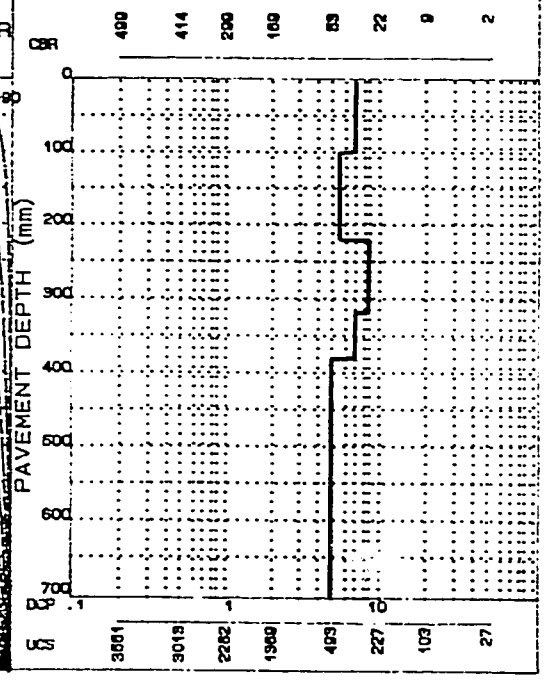


BALANCE CURVE

(DSN 900) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA19.DAT
REGION : NORTHBOUND
ROAD NUMBER : TP19
DISTANCE : 10.5
POSITION : L [X] M R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	145	
BALANCE NUMBER (BN100)	26	20
DIFFERENCE IN BN100	6	
BALANCE CURVE IS WHERE B	19	A= 2837
STRUCT. CAP. (E80x10**6)	.3*	
ROAD CATEGORY	A :	

TRAFFIC VERY HEAVY TRAFFIC

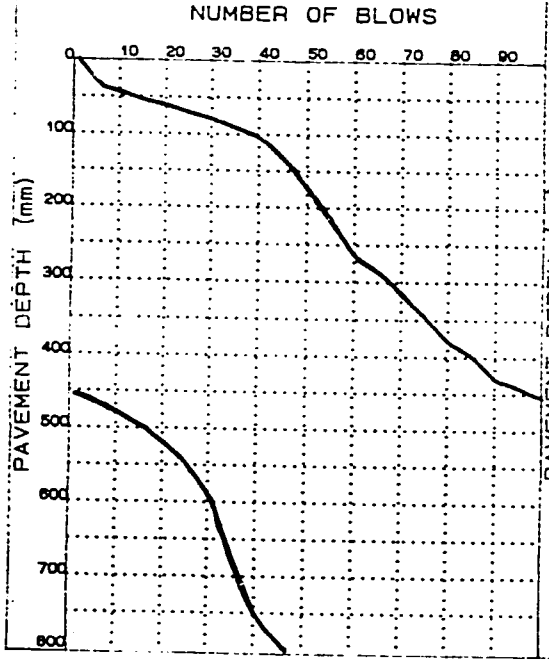
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95%	CBR	UCS
0-150	4.7	2.5	8.8	57	528
151-300	7.9	1.8	10.8	30	299
301-450	5.3	1.5	7.7	49	460
451-600	6.0	3.3	11.4	42	402
601-800	14.4	3.8	20.6	14	152

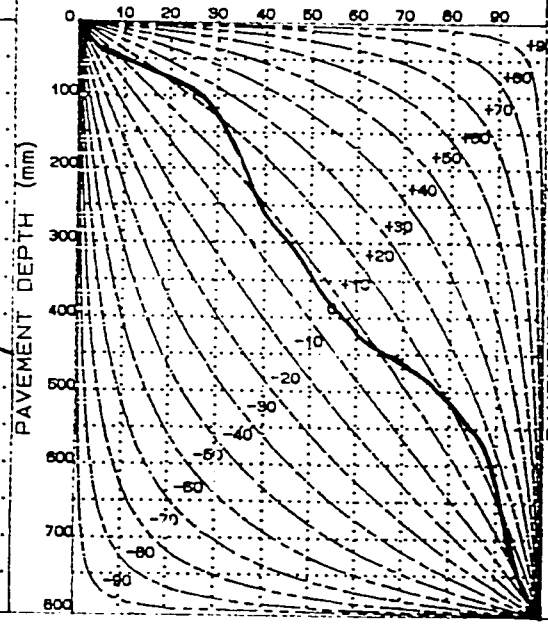
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

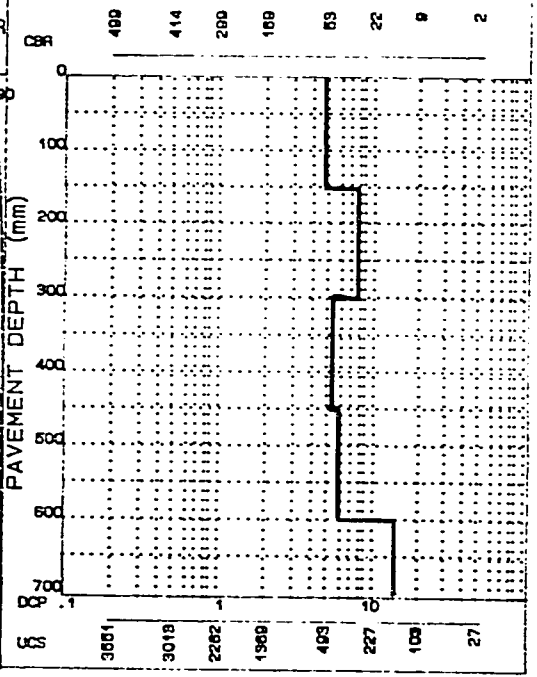


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA6.DAT
REGION : LUSAKA CHILANGA
ROAD NUMBER : TP6
DISTANCE : 12.0
POSITION : L | M | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	156	
BALANCE NUMBER (BN100)	13	20
DIFFERENCE IN BN100	-7	
BALANCE CURVE IS WHERE B	15	A= 3299
STRUCT. CAP. (E80x10**6)	.33#	
ROAD CATEGORY	A :	

TRAFFIC VERY HEAVY TRAFFIC

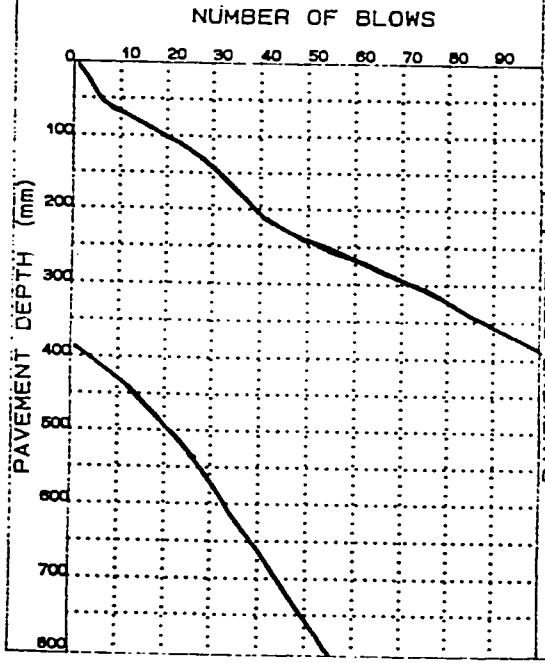
Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	BEP	CBR	UCS
0-175	6.1	2.7	10.5	41	393
176-245	4.7	1.7	7.4	57	528
246-375	3.0	0.8	3.9	102	878
376-800	7.8	1.7	10.8	30	299

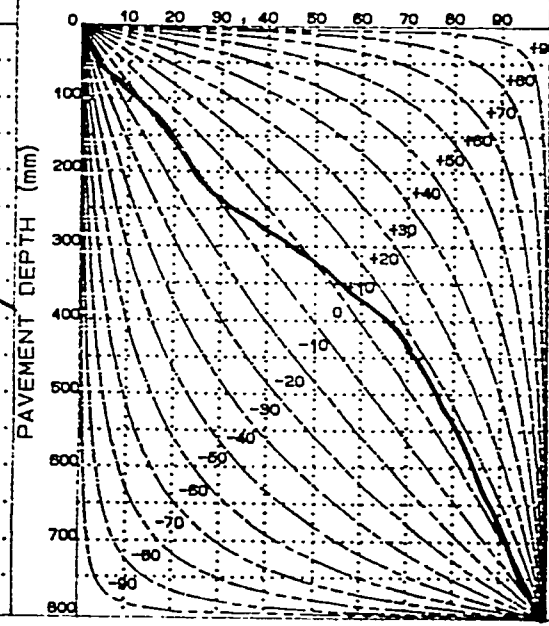
CATEGORY VI : POORLY BALANCED DEEP STRUCTURE (PBD)

DCP CURVE

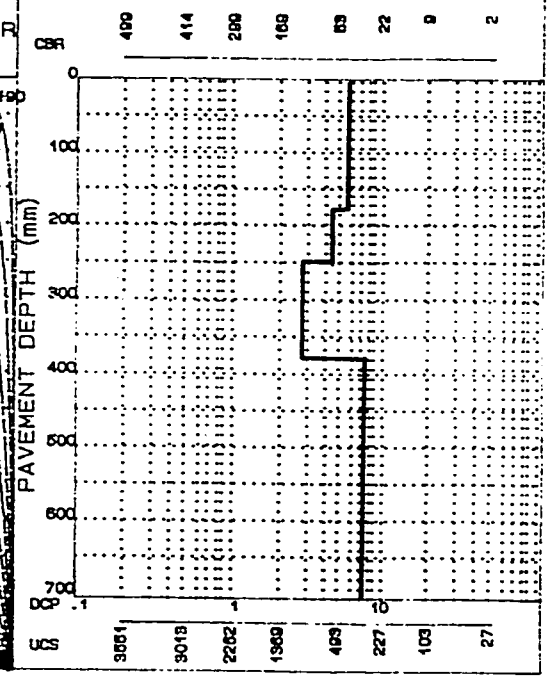


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE :LSKA7.DAT
REGION :LUSAKA CHILANGA
ROAD NUMBER :TP7
DISTANCE : 12.5
POSITION : L | X | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS | CROCK | LONG. | OTHER
DATE :89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	160	
BALANCE NUMBER (BN100)	17	19
DIFFERENCE IN BN100	-2	
BALANCE CURVE IS WHERE B	12	A= 1601
STRUCT. CAP. (E80x10**6)	1.55*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

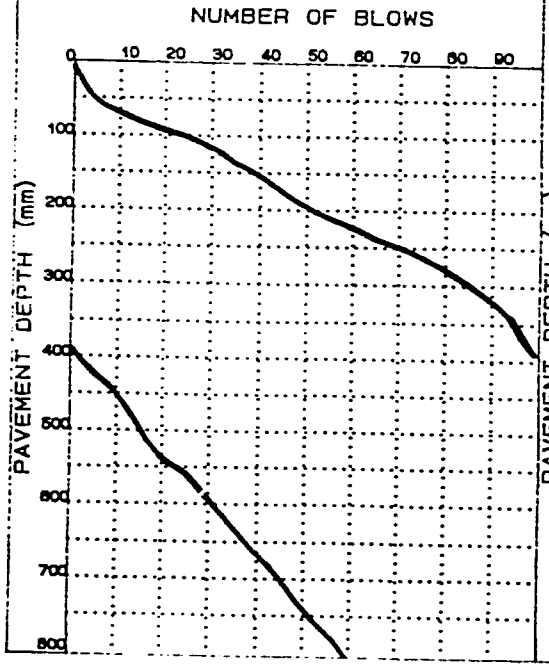
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	$B_{85}P$	CBR	UCS
0-145	5.4	3.4	11.0	48	452
146-800	6.3	2.2	9.9	36	378

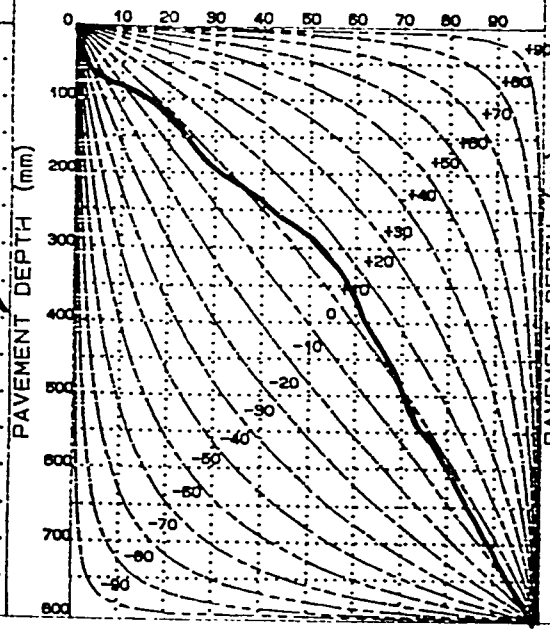
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABO)

DCP CURVE

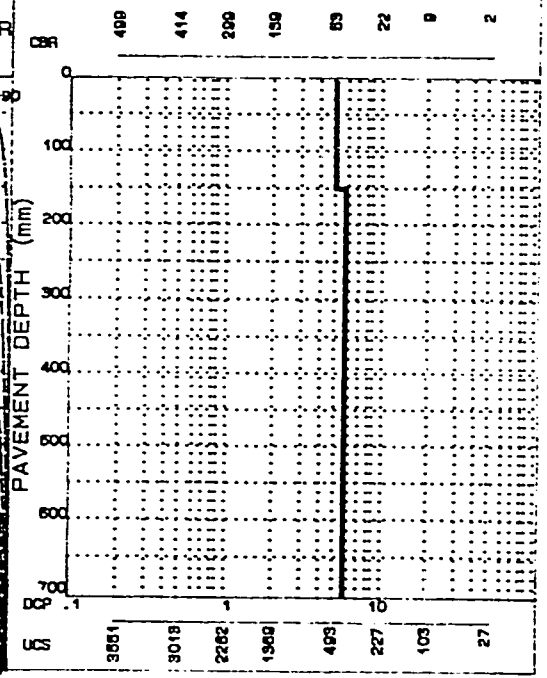


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE :LSKA8.DAT
REGION :LUSAKA CHILANGA
ROAD NUMBER :TPB
DISTANCE : 13.7
POSITION : L | | | M | | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE :89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	256	
BALANCE NUMBER (BN100)	46	51
DIFFERENCE IN BN100	-6	

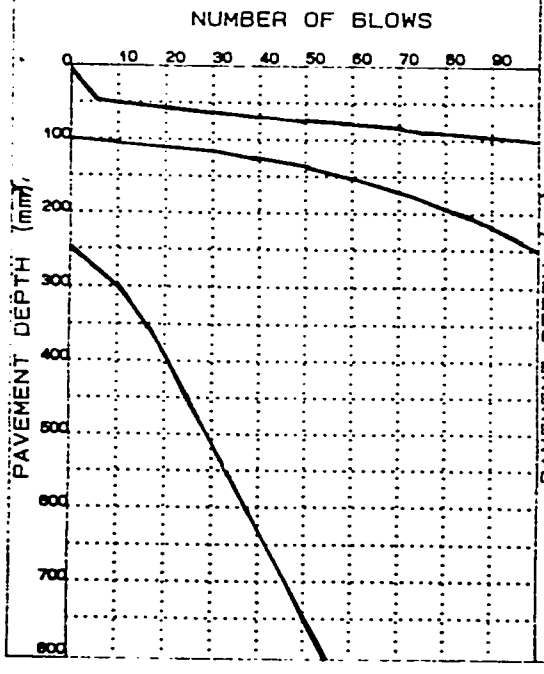
BALANCE CURVE IS WHERE B45 A= 2772
STRUCT. CAP. (E80x10**6) .94*
ROAD CATEGORY A :
TRAFFIC VERY HEAVY TRAFFIC
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	85P	CBR	UCS
0-175	2.7	3.0	7.5	117	991
176-305	4.2	1.6	6.8	68	598
305-800	11.6	1.2	13.5	18	190

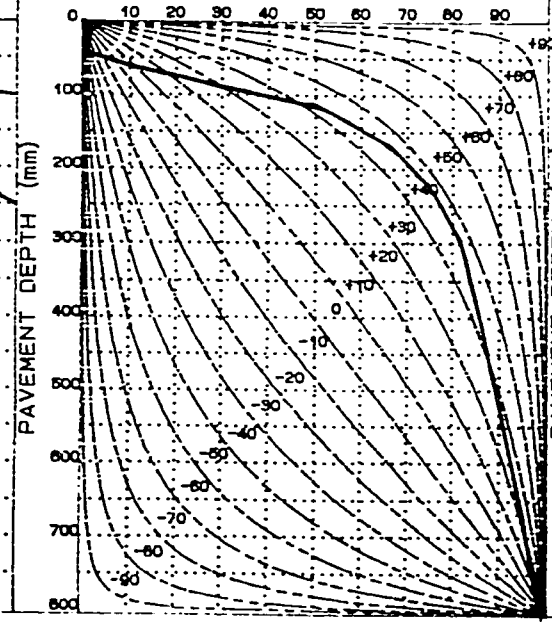
CATEGORY II : AVERAGELY BALANCED SHALLOW STRUCTURE (ABS)

DCP CURVE

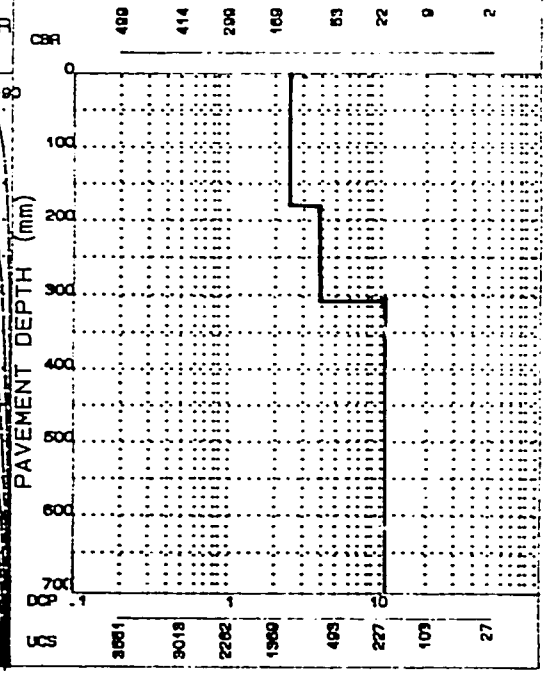


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT. CSIR, SA

DATA FILE : LSKA9.DAT
REGION : LUSAKA CHILANGA
ROAD NUMBER : TP9
DISTANCE : 16.9
POSITION : L [X] M R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	115	
BALANCE NUMBER (BN100)	22	26
DIFFERENCE IN BN100	-4	
BALANCE CURVE IS WHERE B22	A= 1907	
STRUCT. CAP. (E80x10**6)	0.49*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

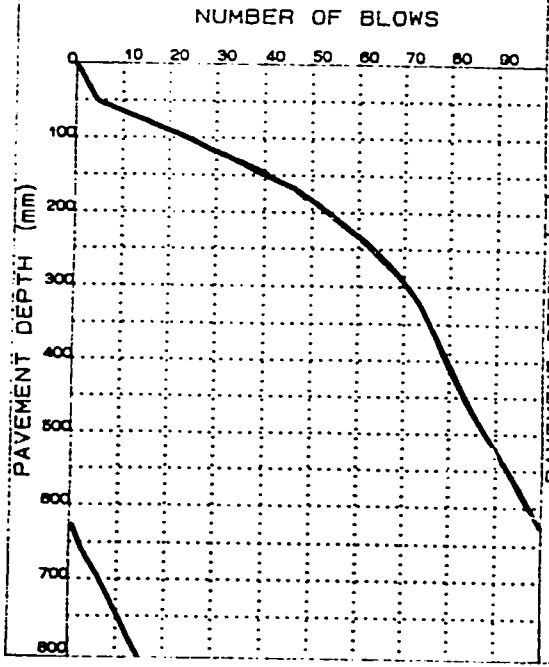
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95%	CBR	UCS
0-230	4.9	2.7	9.4	54	501
231-800	11.0	2.0	14.3	19	200

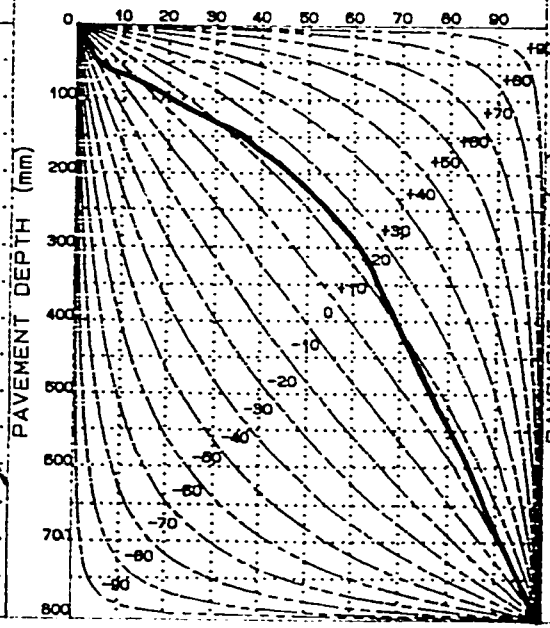
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABC)

DCP CURVE

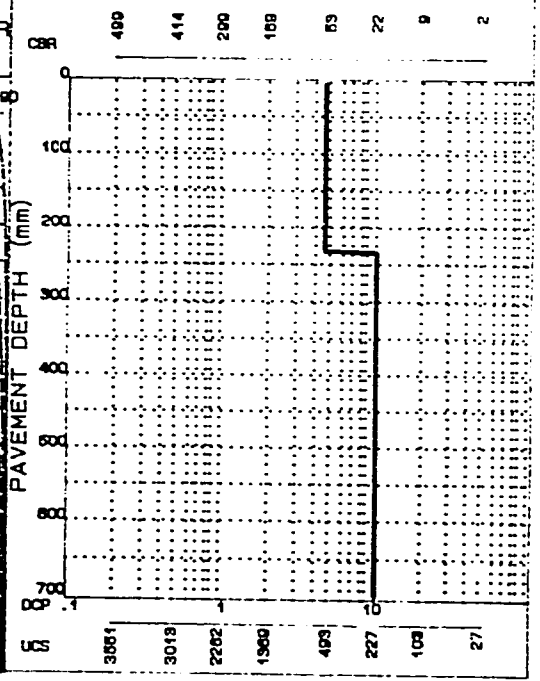


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA18.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TP18
DISTANCE : 18.2
POSITION : L | X | M | | | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	171	
BALANCE NUMBER (BN100)	39	14
DIFFERENCE IN BN100	-5	

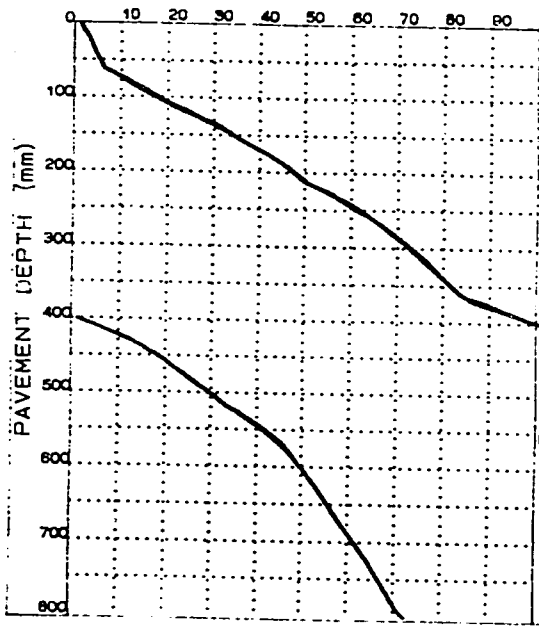
BALANCE CURVE IS WHERE B = A= 3210
STRUCT. CAP. (EBOx10**6) .96*
ROAD CATEGORY A :
TRAFFIC VERY HEAVY TRAFFIC
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	SE	CBR	UCS
0-330	5.4	3.2	10.7	48	452
331-510	3.9	1.3	6.1	73	654
511-800	7.8	2.0	11.1	30	299

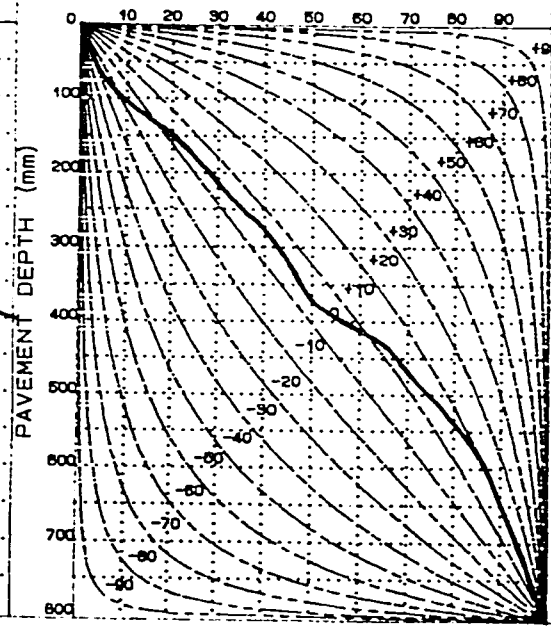
CATEGORY VI : POORLY BALANCED DEEP STRUCTURE (PBD)

DCP CURVE

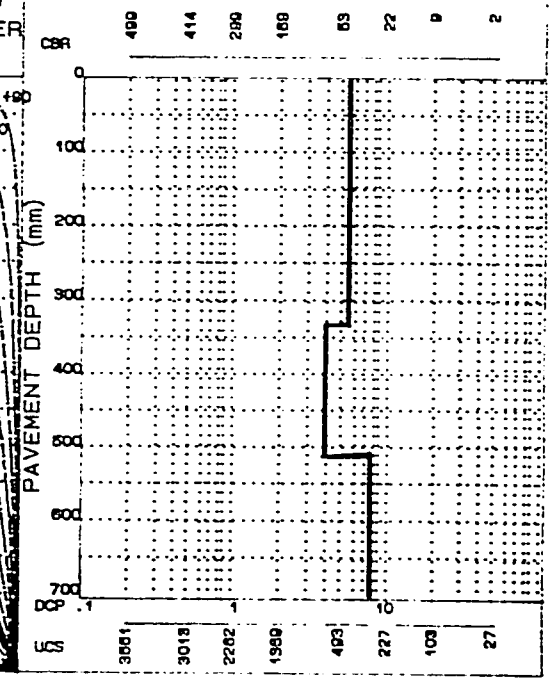


BALANCE CURVE

(DSN 80) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA13.DAT
REGION : CHLANGA KAFUE
ROAD NUMBER : TP13
DISTANCE : 20.2
POSITION : L [X] M R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	101	
BALANCE NUMBER (BN100)	14	13
DIFFERENCE IN BN100	-2	

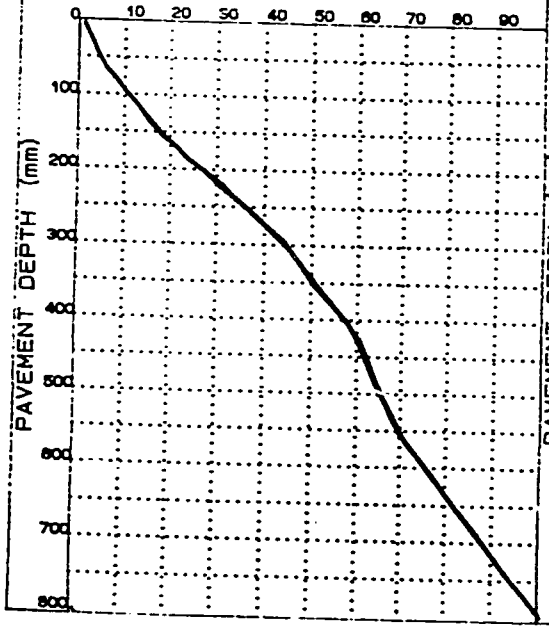
BALANCE CURVE IS WHERE B = A= 1617
STRUCT. CAP. (E80x10**6) 0.31*
ROAD CATEGORY A :
TRAFFIC VERY HEAVY TRAFFIC
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SO	DCP	CBR	UCS
0-115	9.6	2.5	13.7	23	238
115-800	6.5	2.6	13.1	27	272

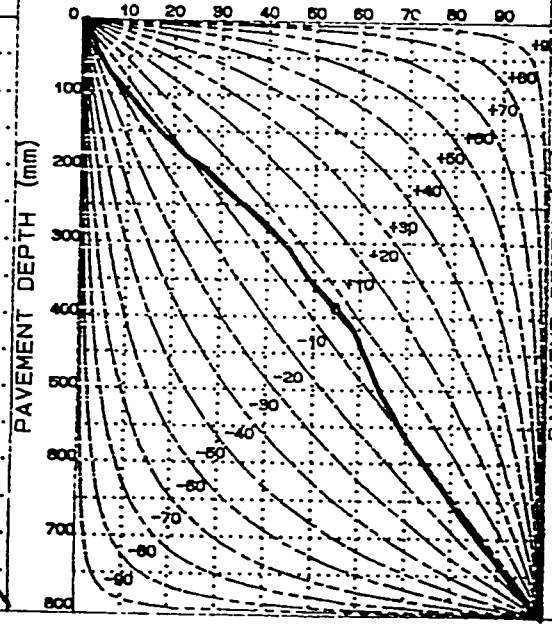
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

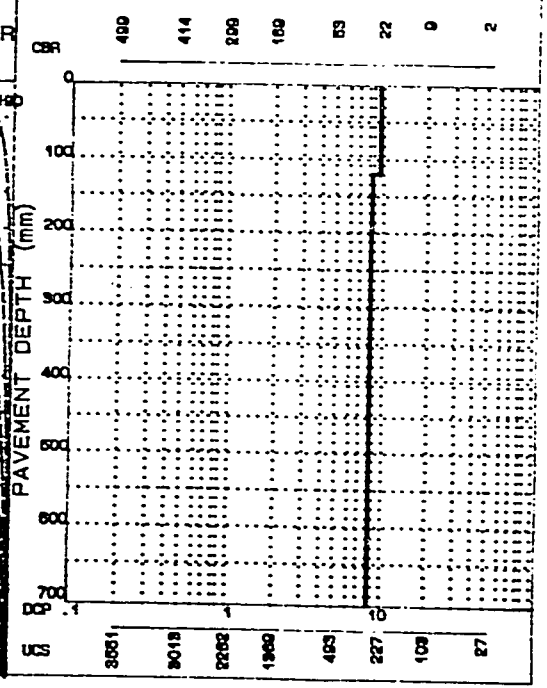


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE :LSKA14.DAT
REGION :CHILANGA KAFUE
ROAD NUMBER :TP14
DISTANCE : 27.0
POSITION : L [X] M R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE :89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	264	
BALANCE NUMBER (BN100)	14	19
DIFFERENCE IN BN100	-5	
BALANCE CURVE IS WHERE	B12=A= 2131	
STRUCT. CAP. (E80x10**6)	.97*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

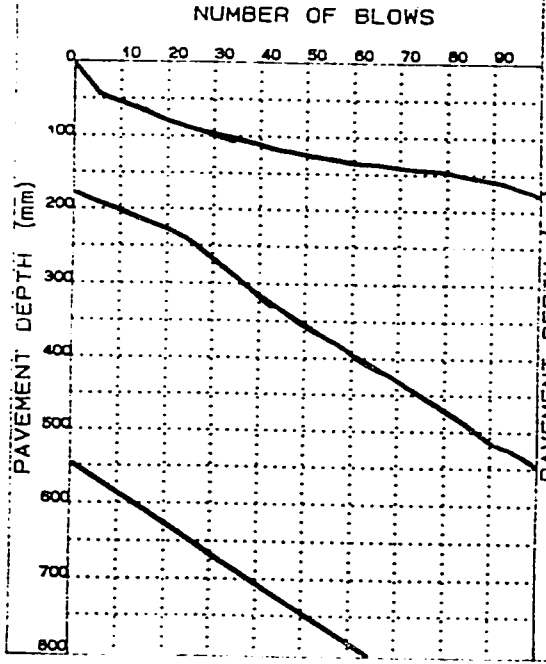
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0-185	3.0	2.4	7.0	102	878
186-285	3.7	1.1	5.8	78	693
286-800	4.0	0.3	4.5	70	630

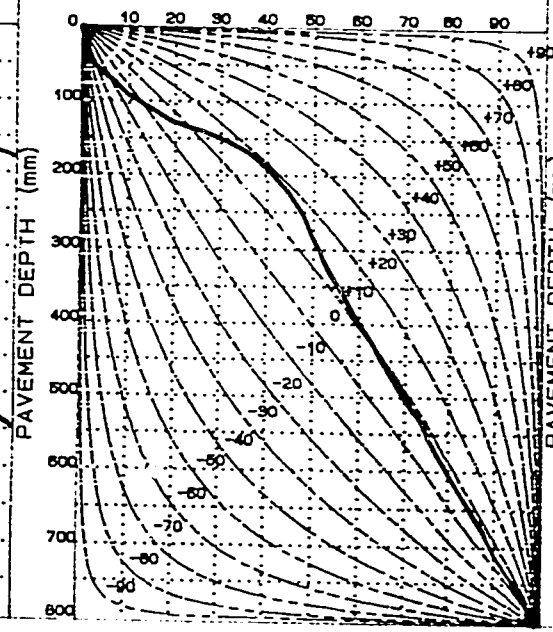
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

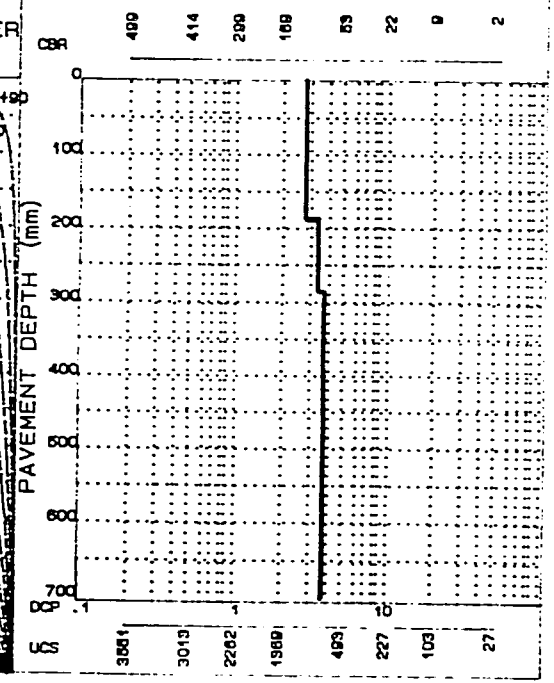


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA10.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TP10
DISTANCE : 26.8
POSITION : L | | | M | | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : [illegible]/05/90

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	162	
BALANCE NUMBER (BN100)	32	40
DIFFERENCE IN BN100	-8	
BALANCE CURVE IS WHERE B	38	A= 1917
STRUCT. CAP. (E80x10**6)	.62*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

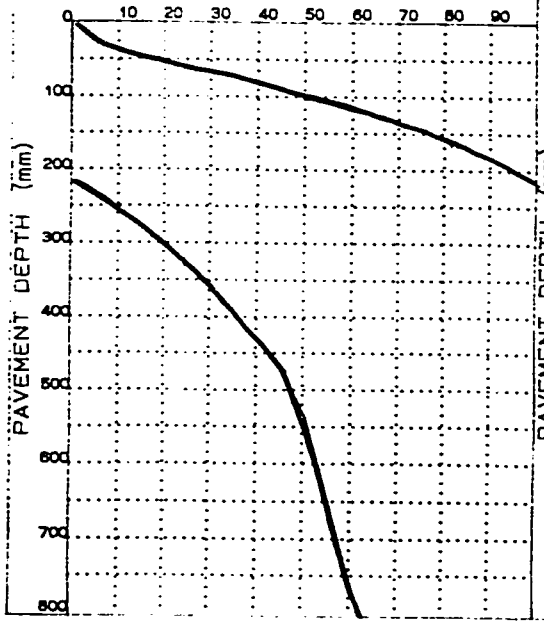
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	BSP	CBR	UCS
0-105	2.5	1.5	4.9	128	1072
106-800	12.5	8.4	26.3	16	172

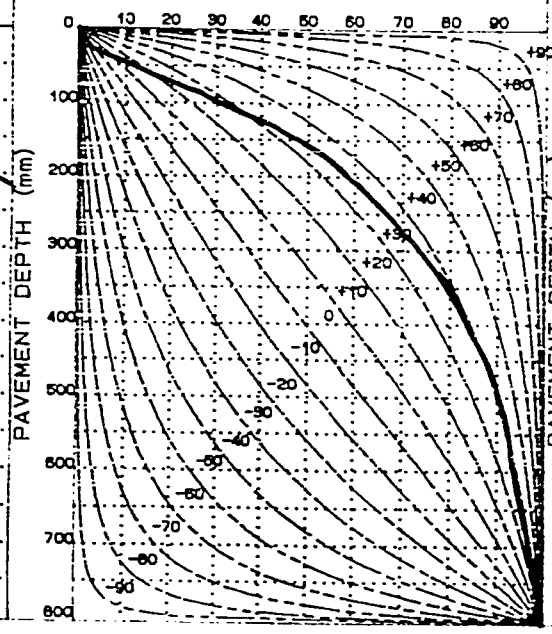
CATEGORY V : AVERAGELY BALANCED DEEP STRUCTURE (ABD)

DCP CURVE

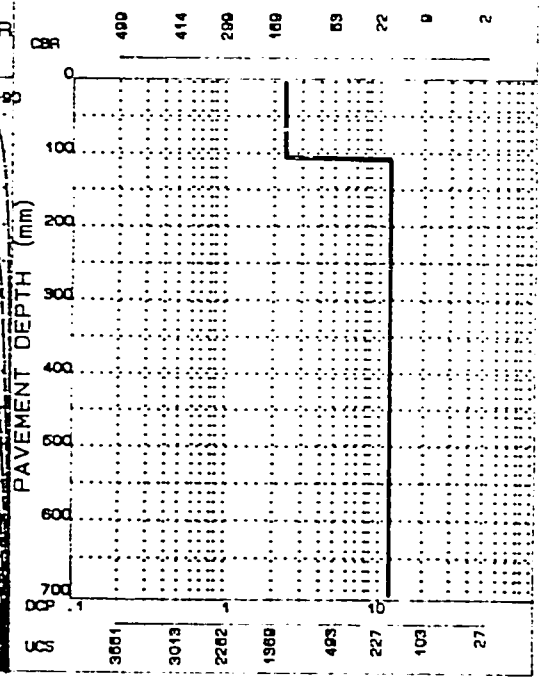


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA11.DAT
REGION : CHLANGA KAFUE
ROAD NUMBER : TP11
DISTANCE : 33.2
POSITION : L | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS | CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

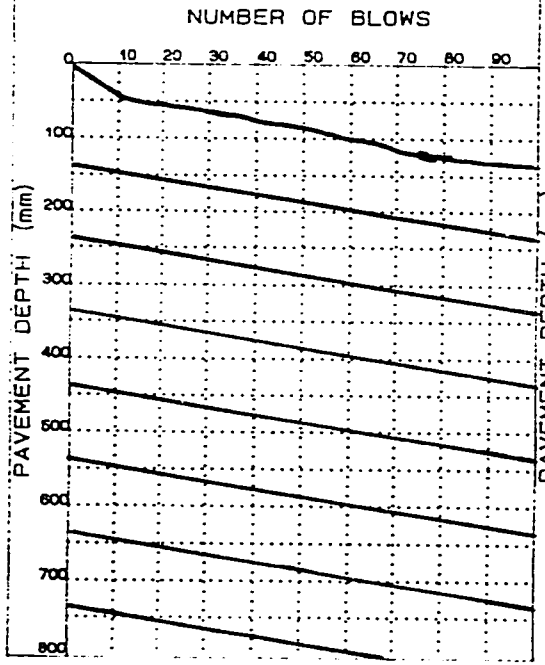
	DATA	B/CURVE
STRUCTURE NUMBER	770	
BALANCE NUMBER (BN100)	8	11
DIFFERENCE IN BN100	-3	
BALANCE CURVE IS WHERE B	-5	A= 74
STRUCT. CAP. (E80x10**6)	:	
ROAD CATEGORY	A:	
TRAFFIC : VERY HEAVY TRAFFIC		

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	85%P	CBR	UCS
0-220	1.6	1.2	3.6	200	1588
221-800	1.0	0.0	1.0	299	2282

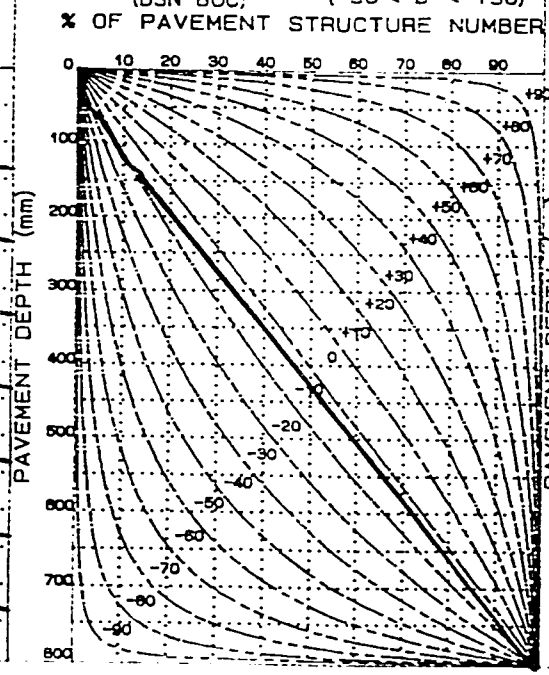
CATEGORY VII : WELL-BALANCED INVERTED STRUCTURE (WB)

DCP CURVE

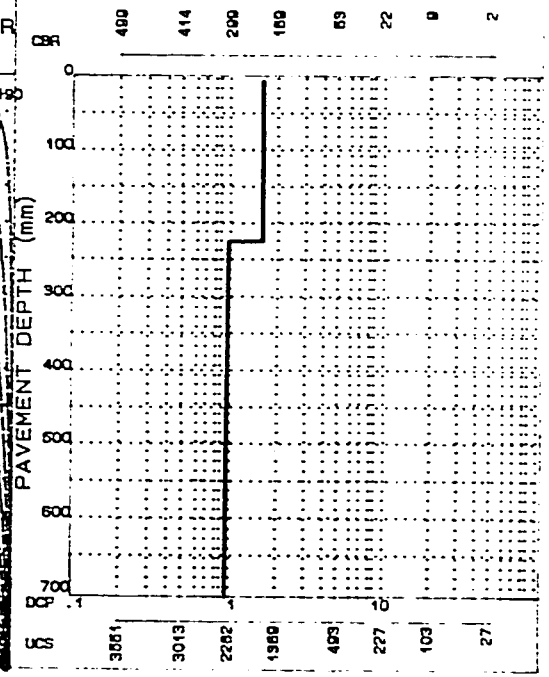


BALANCE CURVE

(DSN 80C) (-90 < B < +90)



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA21.DAT
REGION : NORTHBOUND
ROAD NUMBER : TP21
DISTANCE : 39.5
POSITION : L | X | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	265	
BALANCE NUMBER (BN100)	10	11
DIFFERENCE IN BN100	-1	
BALANCE CURVE IS WHERE B =	A= 2793	
STRUCT. CAP. (EBOx10**6)	8.21*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

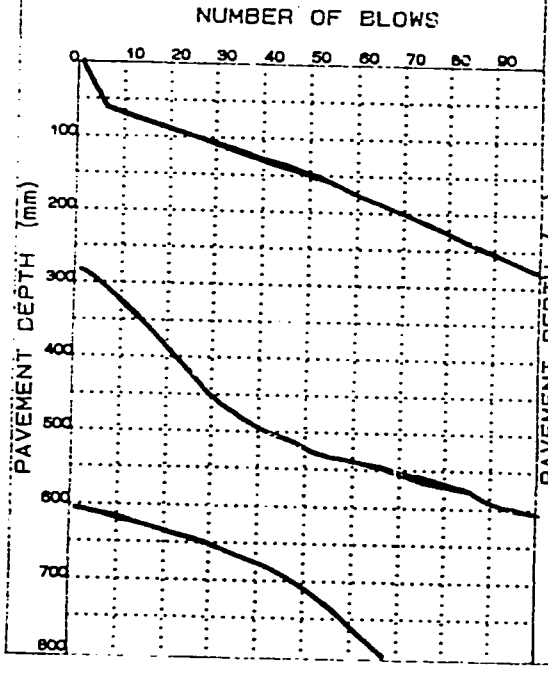
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	DP	CBR	UCS
0-150	8.0	4.9	14.1	42	402
151-300	2.8	0.5	3.7	109	931
301-450	6.3	0.6	7.2	40	385
451-600	2.8	1.4	5.0	111	946
601-800	3.9	1.9	5.9	73	354

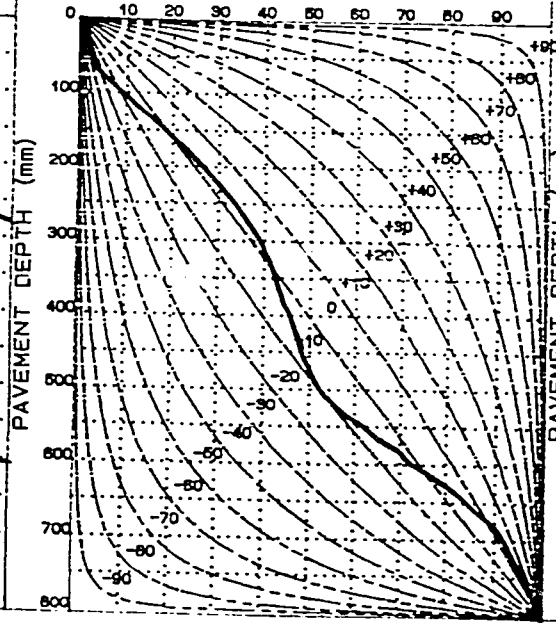
CATEGORY VIII : AVERAGELY BALANCED INVERTED STRUCTURE (AB

DCP CURVE

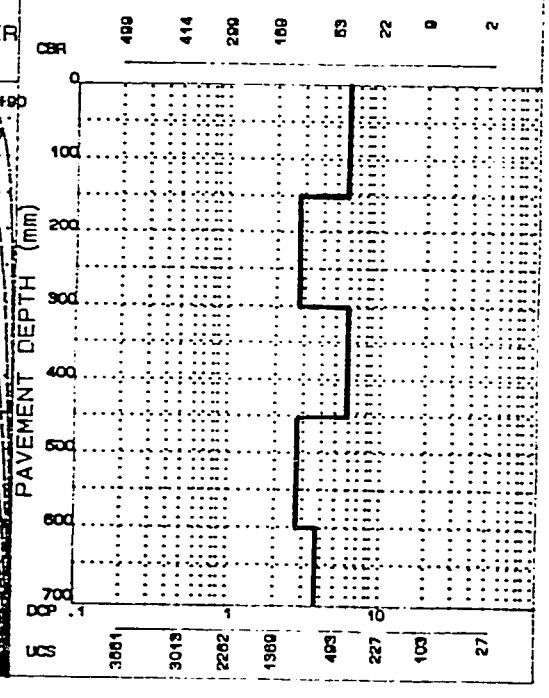


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA12.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TP12
DISTANCE : 43.1
POSITION : L | N | X | R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	246	
BALANCE NUMBER (BN100)	7	10
DIFFERENCE IN BN100	-3	
BALANCE CURVE IS WHERE B	-5	A= 1004
STRUCT. CAP. (E80x10**6)	1.44*	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

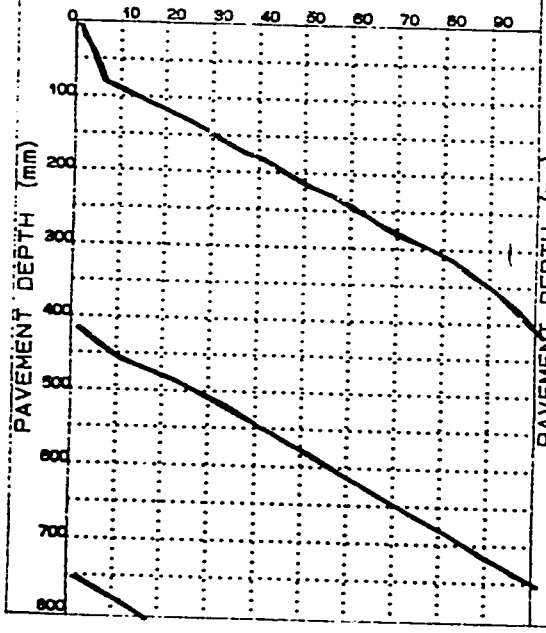
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0- 70	15.0	0.0	15.0	13	143
71-800	3.7	1.3	5.9	77	685

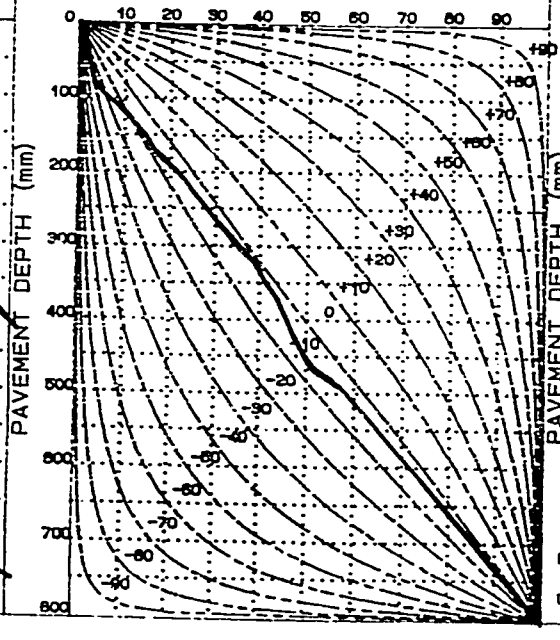
CATEGORY VII : WELL-BALANCED INVERTED STRUCTURE (WBI)

DCP CURVE

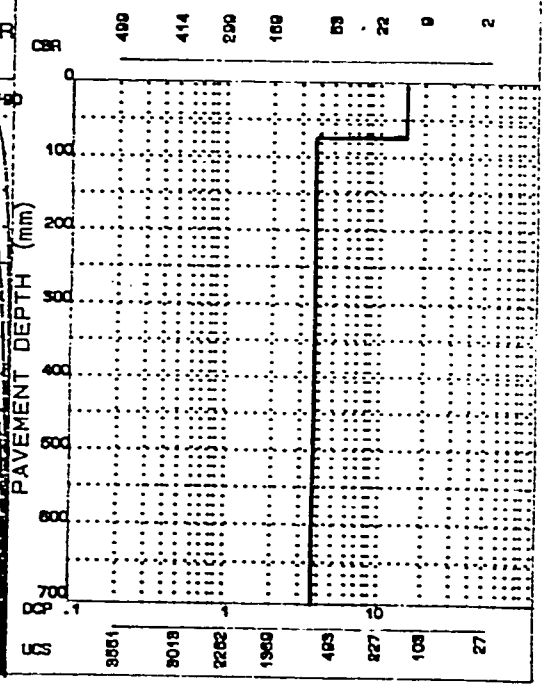


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA15.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TP15
DISTANCE : 45.5
POSITION : L | X | M | R
CONDITION : FAILED | OVERSTRESSED | SOUND
AUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

	DATA	B/CURVE
STRUCTURE NUMBER	541	
BALANCE NUMBER (BN100)	7	5
DIFFERENCE IN BN100	2	
BALANCE CURVE IS WHERE B=	29	A= 1749
STRUCT. CAP. (E80x10**6)	0 : *	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

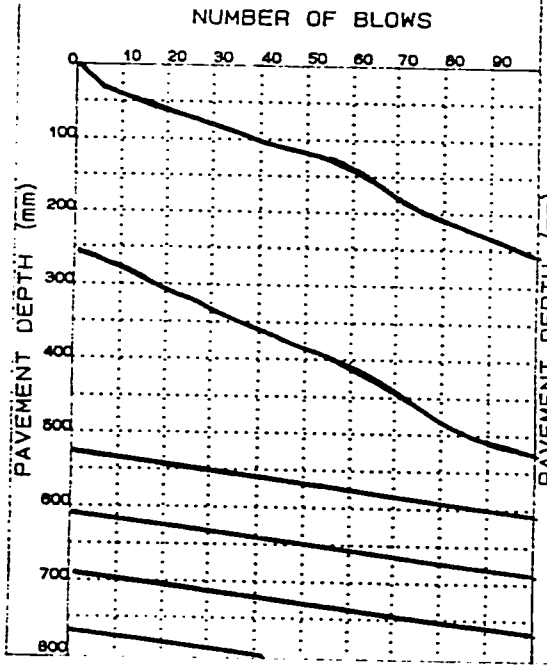
* Structural capacity not reliable

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	95P	CBR	UCS
0-340	2.8	1.1	4.6	108	923
341-800	1.7	1.1	9.5	199	1581

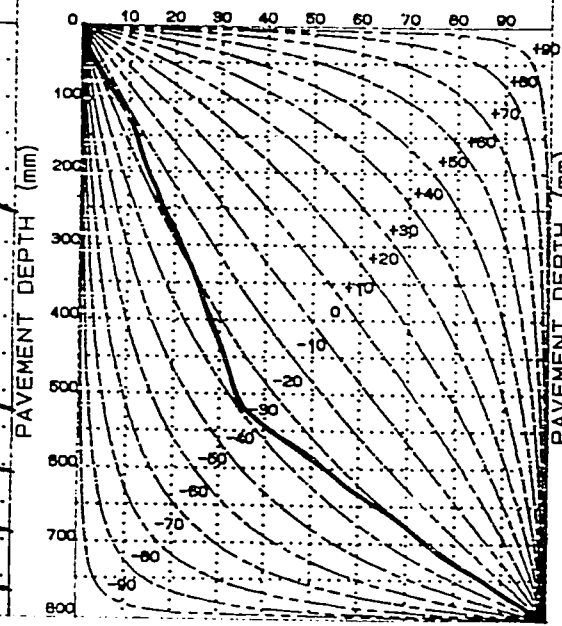
CATEGORY VIII : AVERAGELY BALANCED INVERTED STRUCTURE

DCP CURVE

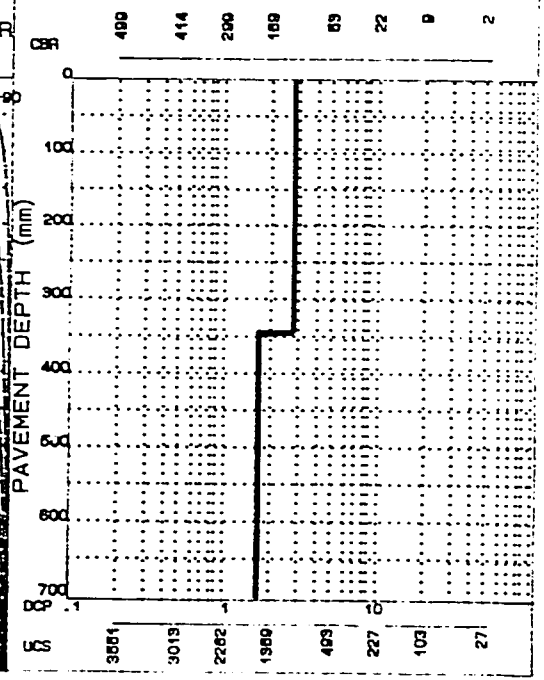


BALANCE CURVE

(DSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



MATERIAL TESTING SERVICES
SUMMARY OF DCP INVESTIGATION

RTT, CSIR, SA

DATA FILE : LSKA16.DAT
REGION : CHILANGA KAFUE
ROAD NUMBER : TP16
DISTANCE : 50.1
POSITION : L M X R
CONDITION : FAILED | OVERSTRESSED | SOUND
RUT. | DEFORM. | PUMP. | CRACKS : CROCK. | LONG. | OTHER
DATE : 89/05/30

PAVEMENT CHARACTERISTICS

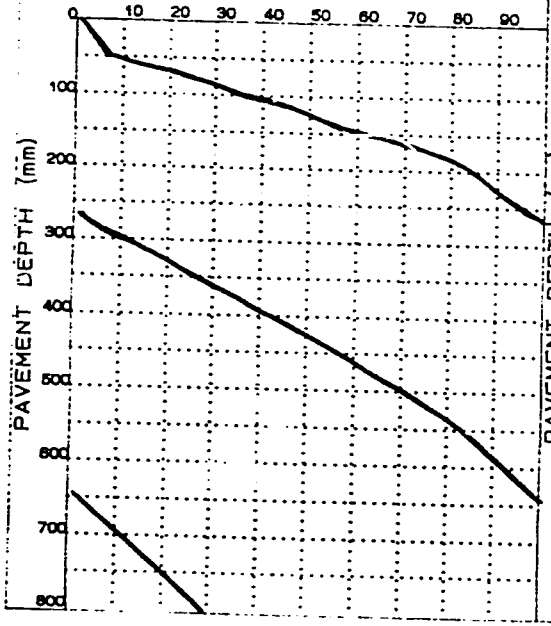
	DATA	B/CURVE
STRUCTURE NUMBER	228	
BALANCE NUMBER (BN100)	17	20
DIFFERENCE IN BN100	-3	
BALANCE CURVE IS WHERE B	15	A= 360
STRUCT. CAP. (E80x10**6)	3.37	
ROAD CATEGORY	A :	
TRAFFIC	VERY HEAVY TRAFFIC	

AVERAGE EQUIVALENT STRENGTH

FROM - TO	AV. PENETRATION	SD	DCP	CBR	UCS
0-255	3.8	2.6	8.1	75	870
256-600	4.4	1.1	8.2	61	558

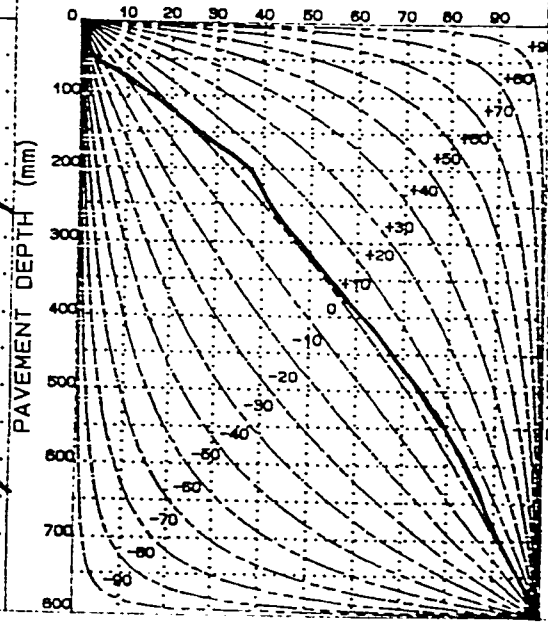
CATEGORY IV : WELL-BALANCED DEEP STRUCTURE (WBD)

DCP CURVE

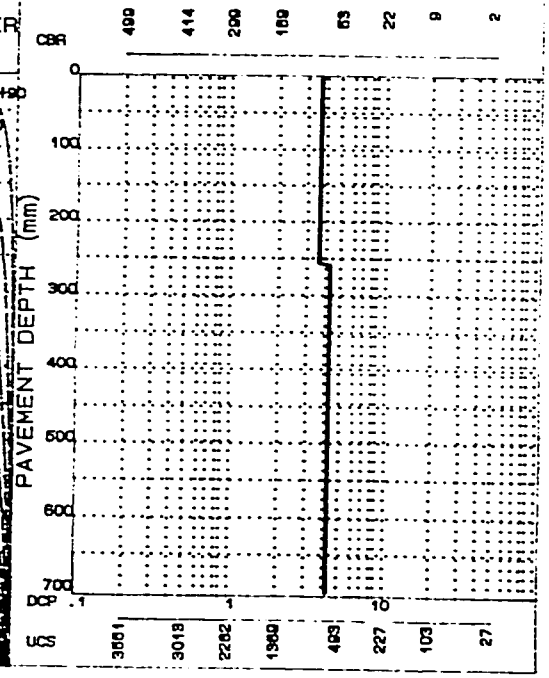


BALANCE CURVE

(CSN 800) (-90 < B < +90)
% OF PAVEMENT STRUCTURE NUMBER



LAYER STRENGTH DIAGRAM



APPENDIX C

Grading Curves

KAFUE - LUSAKA ROAD

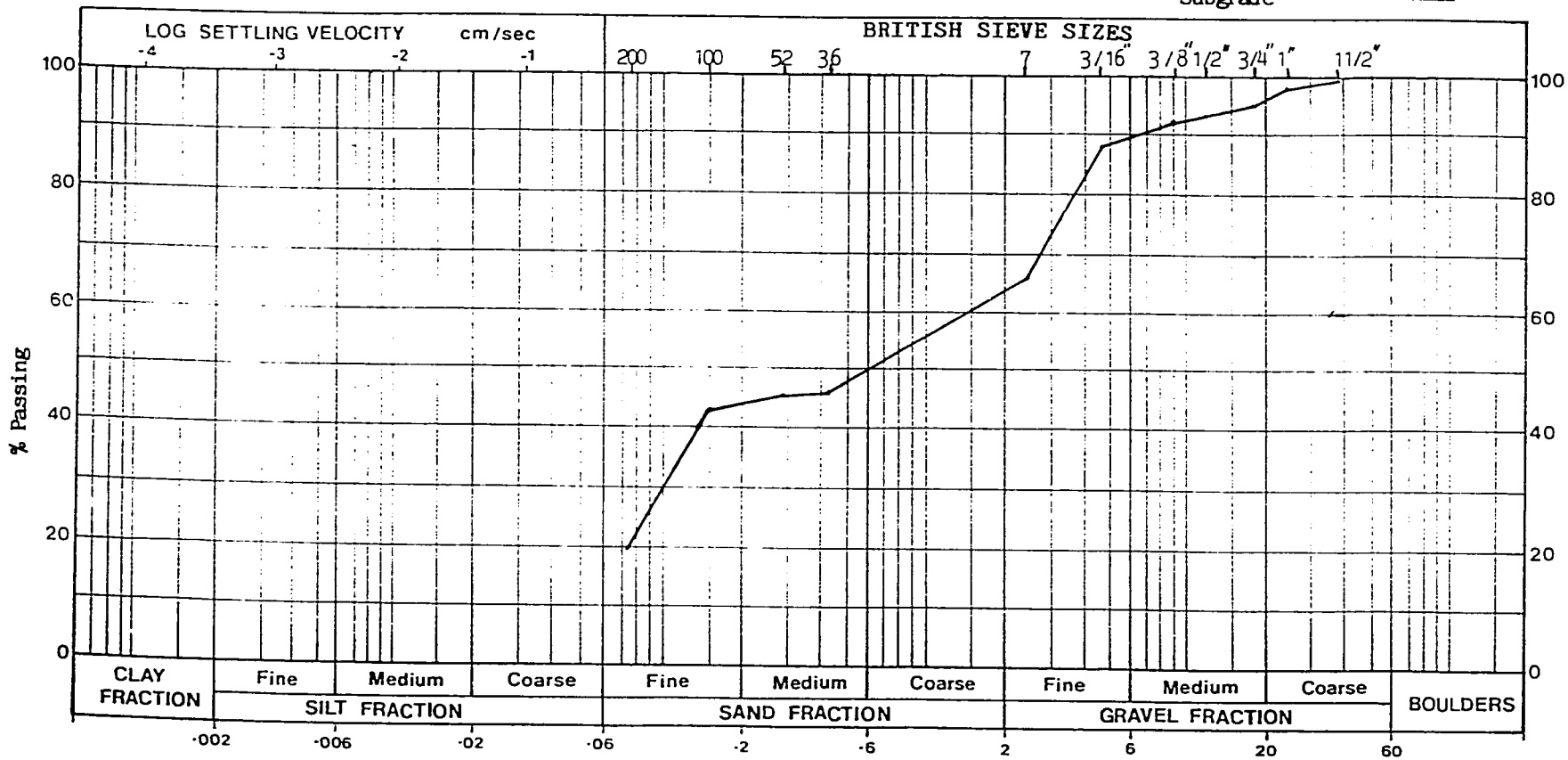
Test Pit No : 17

Location (km) : 1.5

Roadbase

Sub-base

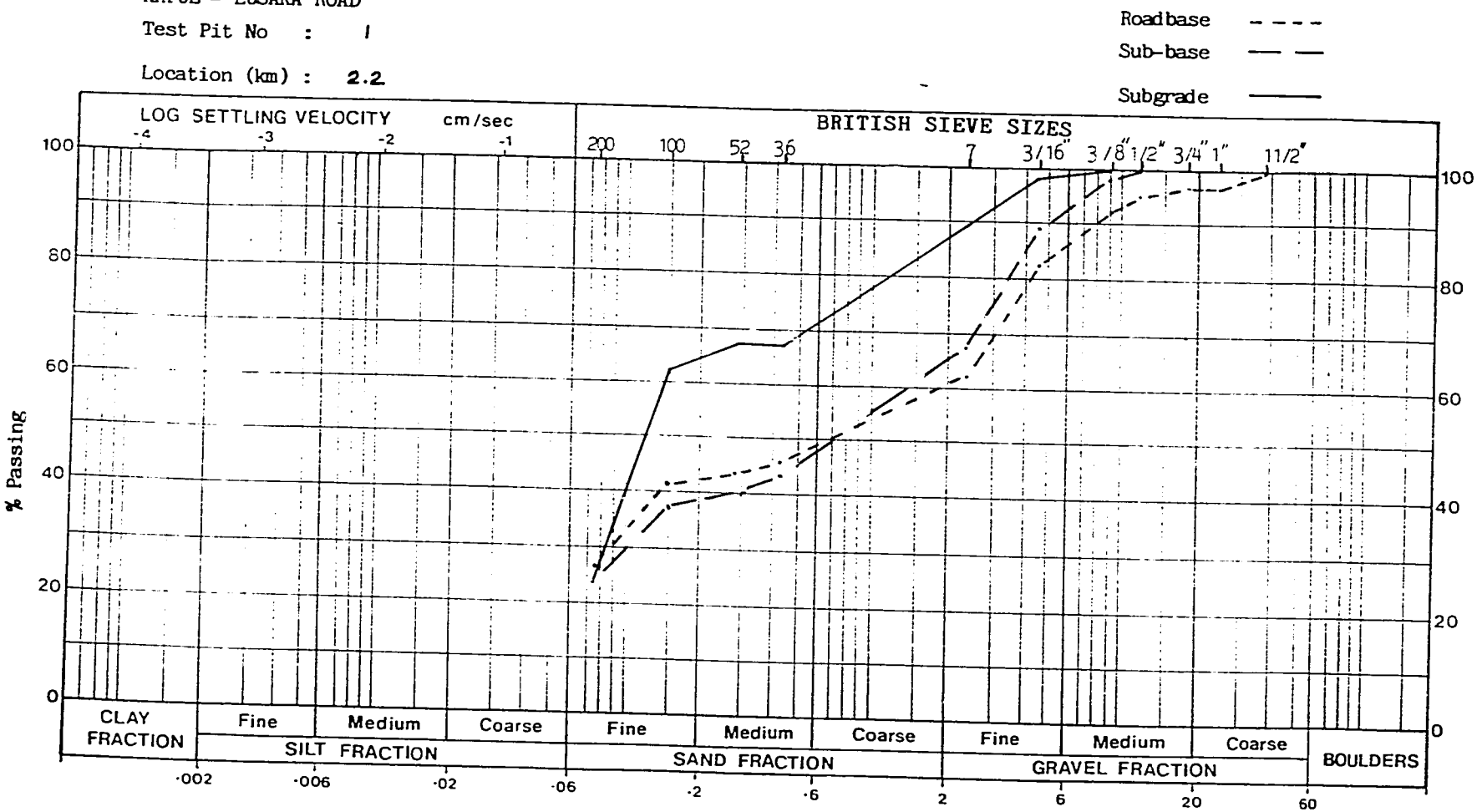
Subgrade



KAFUE - LUSAKA ROAD

Test Pit No : 1

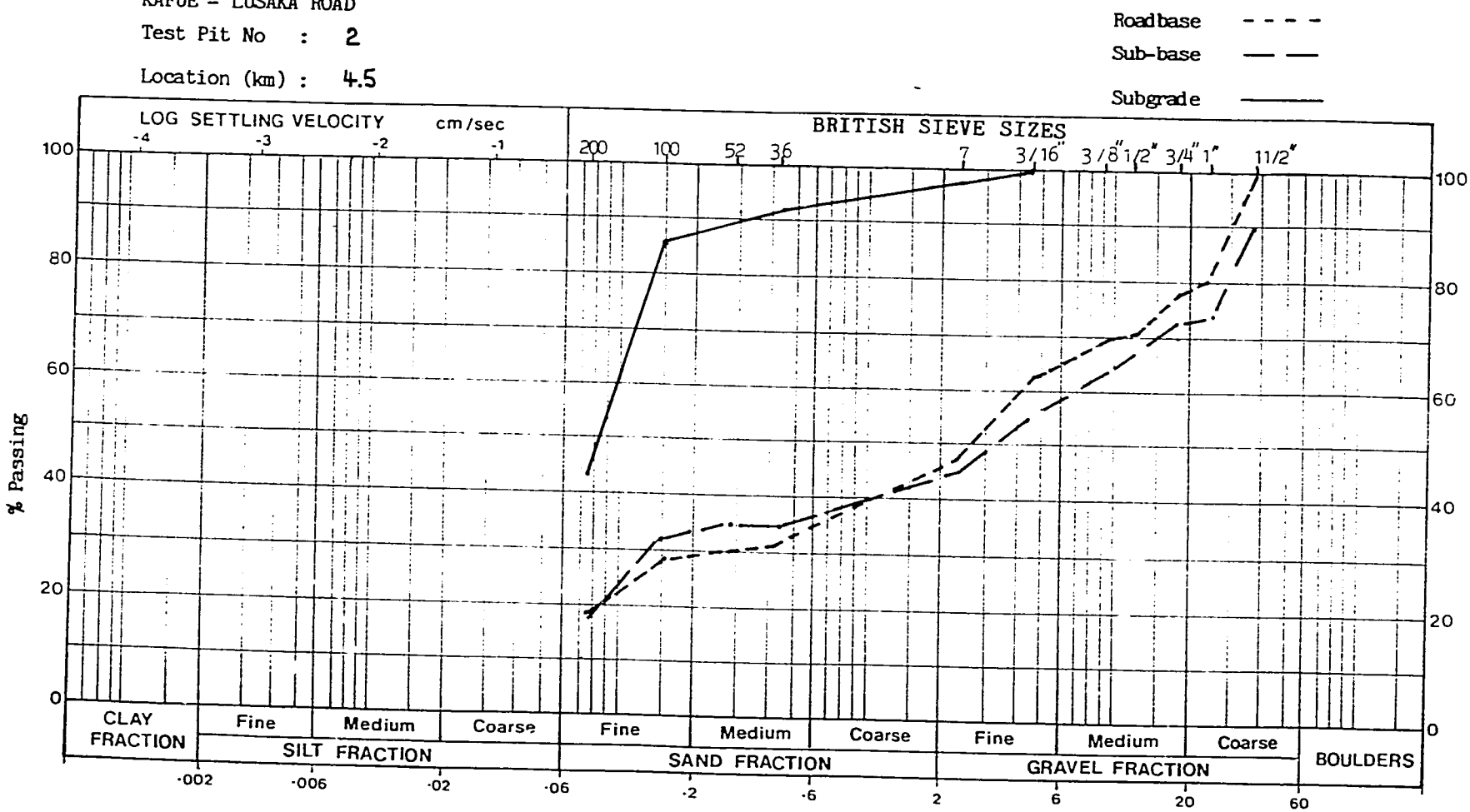
Location (km) : 2.2



KAFUE - LUSAKA ROAD

Test Pit No : 2

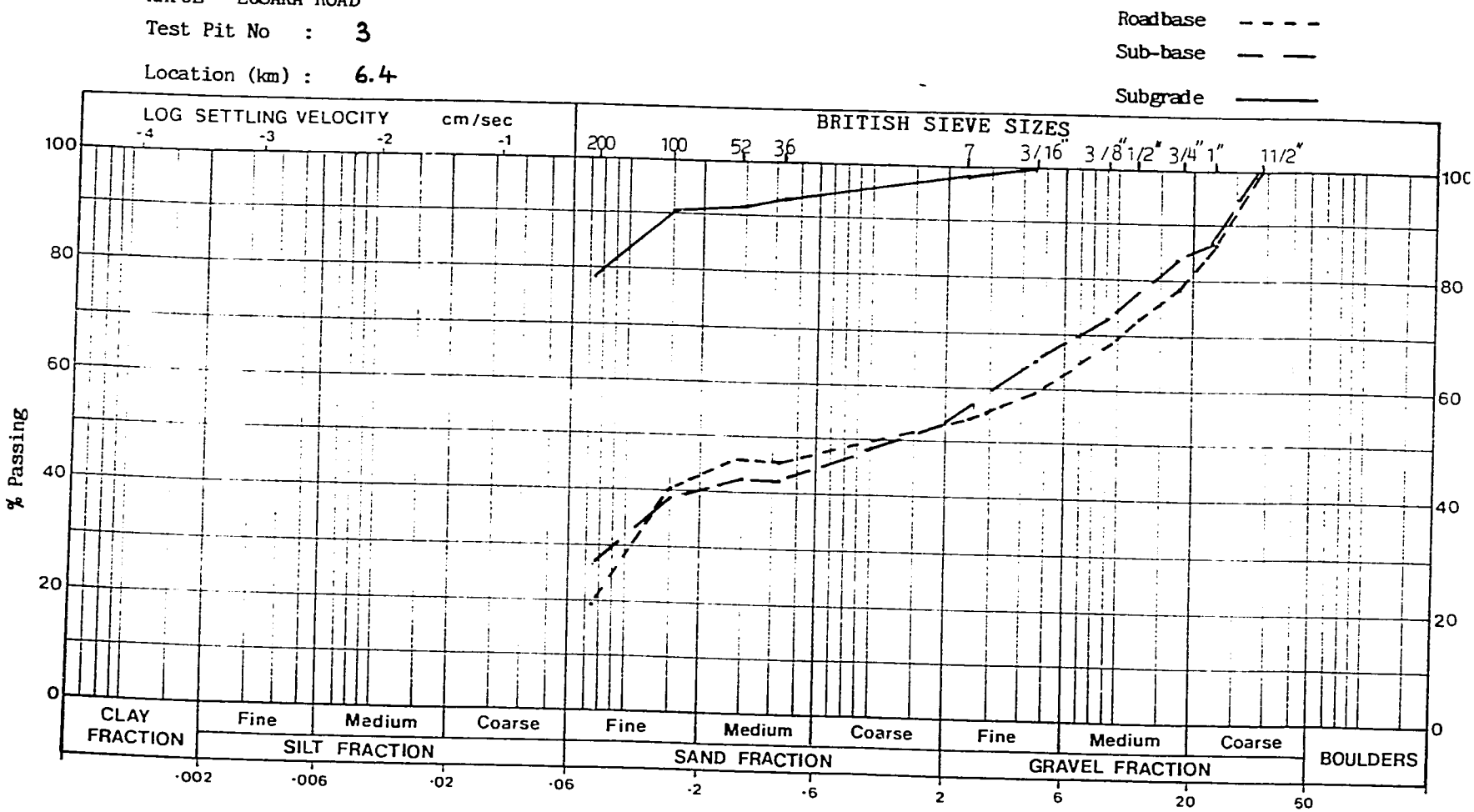
Location (km) : 4.5



KAFUE - LUSAKA ROAD

Test Pit No : 3

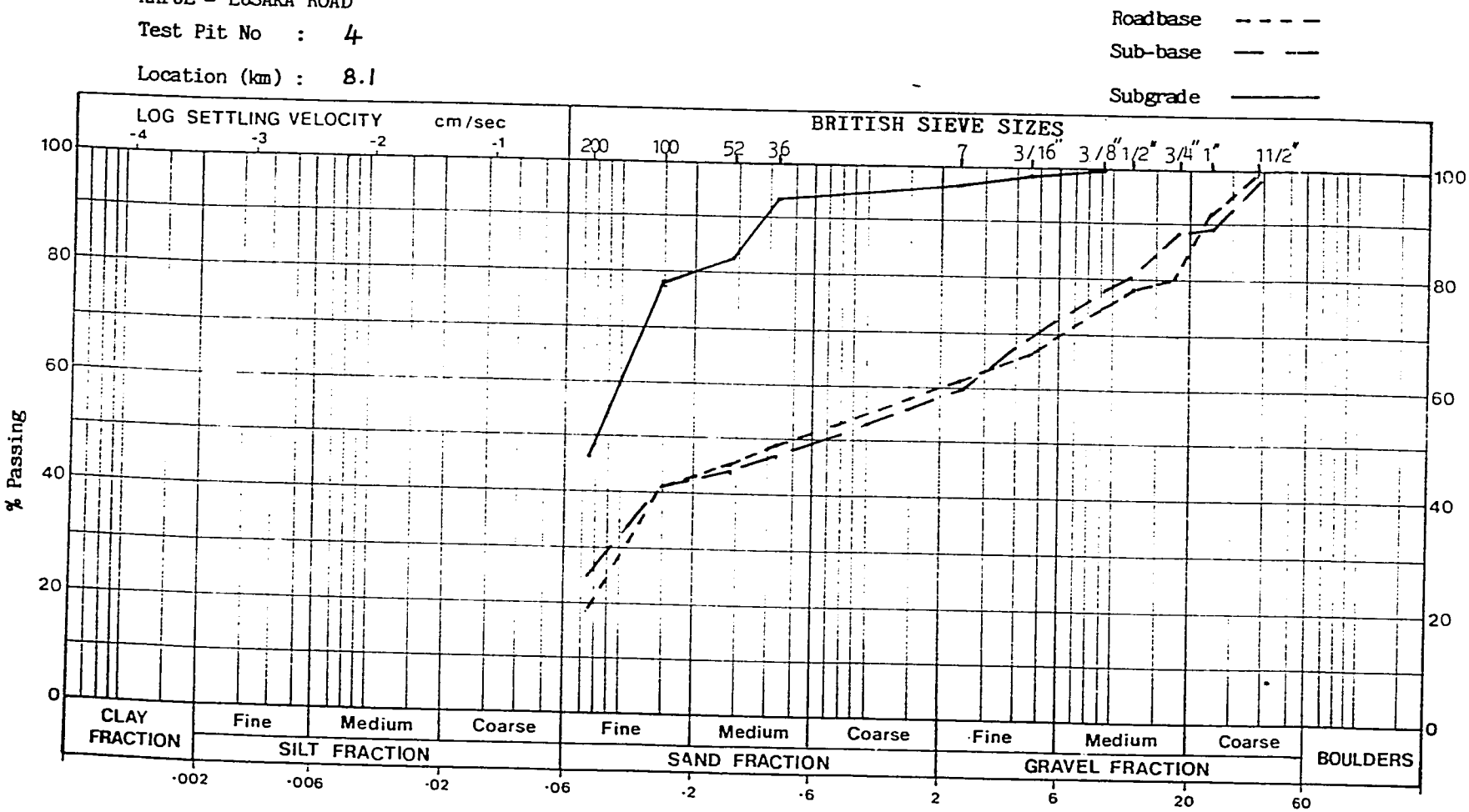
Location (km) : 6.4



KAFUE - LUSAKA ROAD

Test Pit No : 4

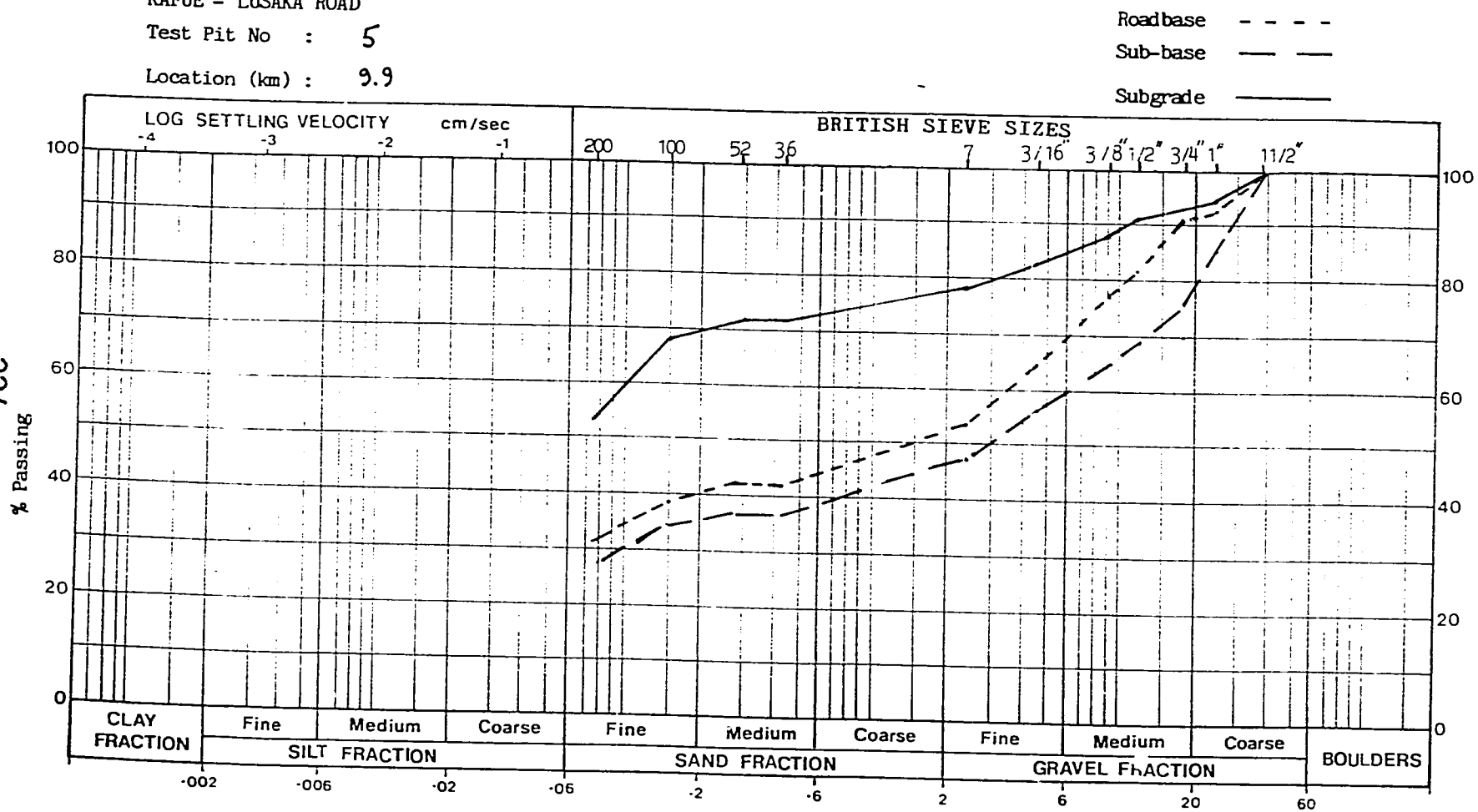
Location (km) : 8.1



KAFUE - LUSAKA ROAD

Test Pit No : 5

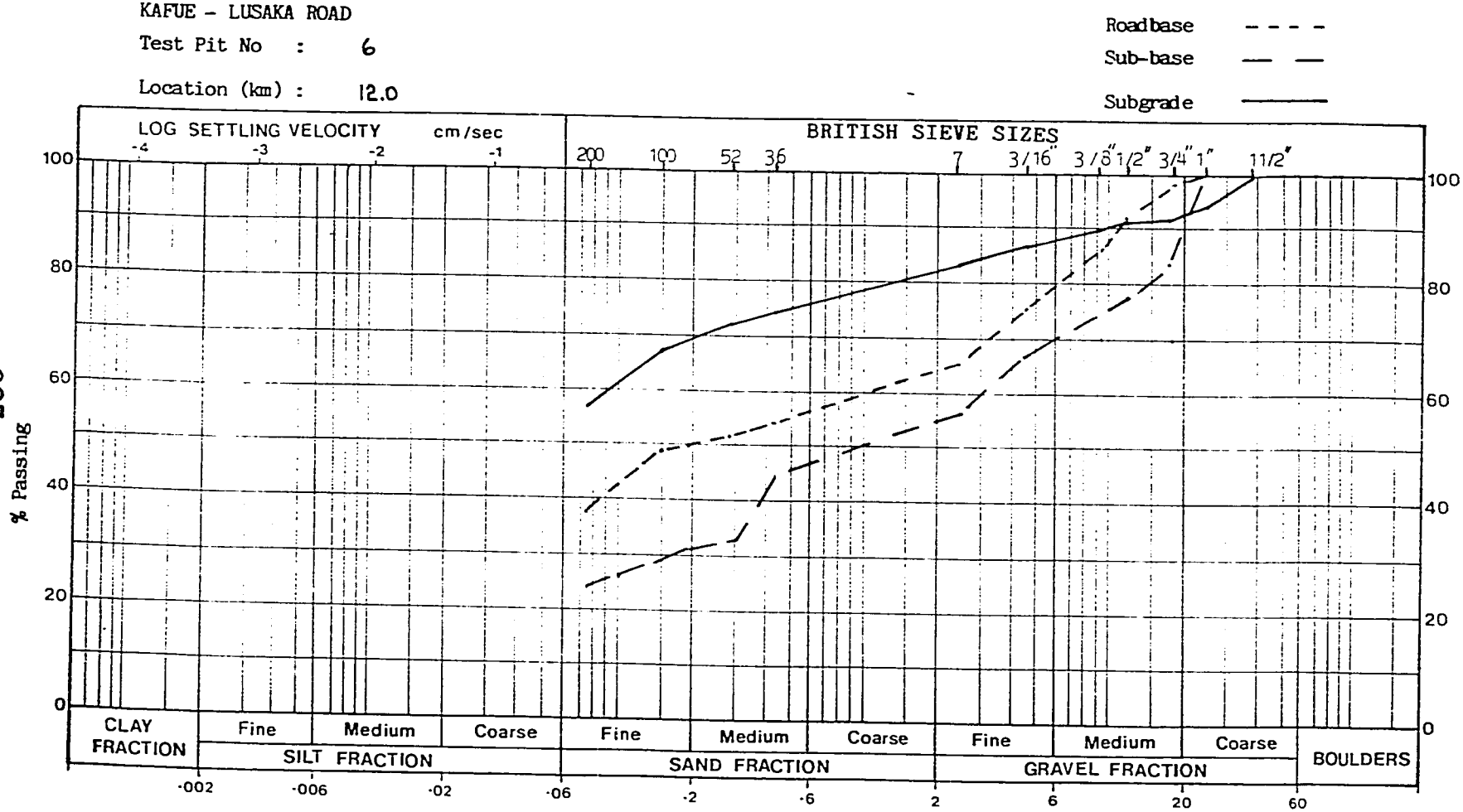
Location (km) : 9.9



KAFUE - LUSAKA ROAD

Test Pit No : 6

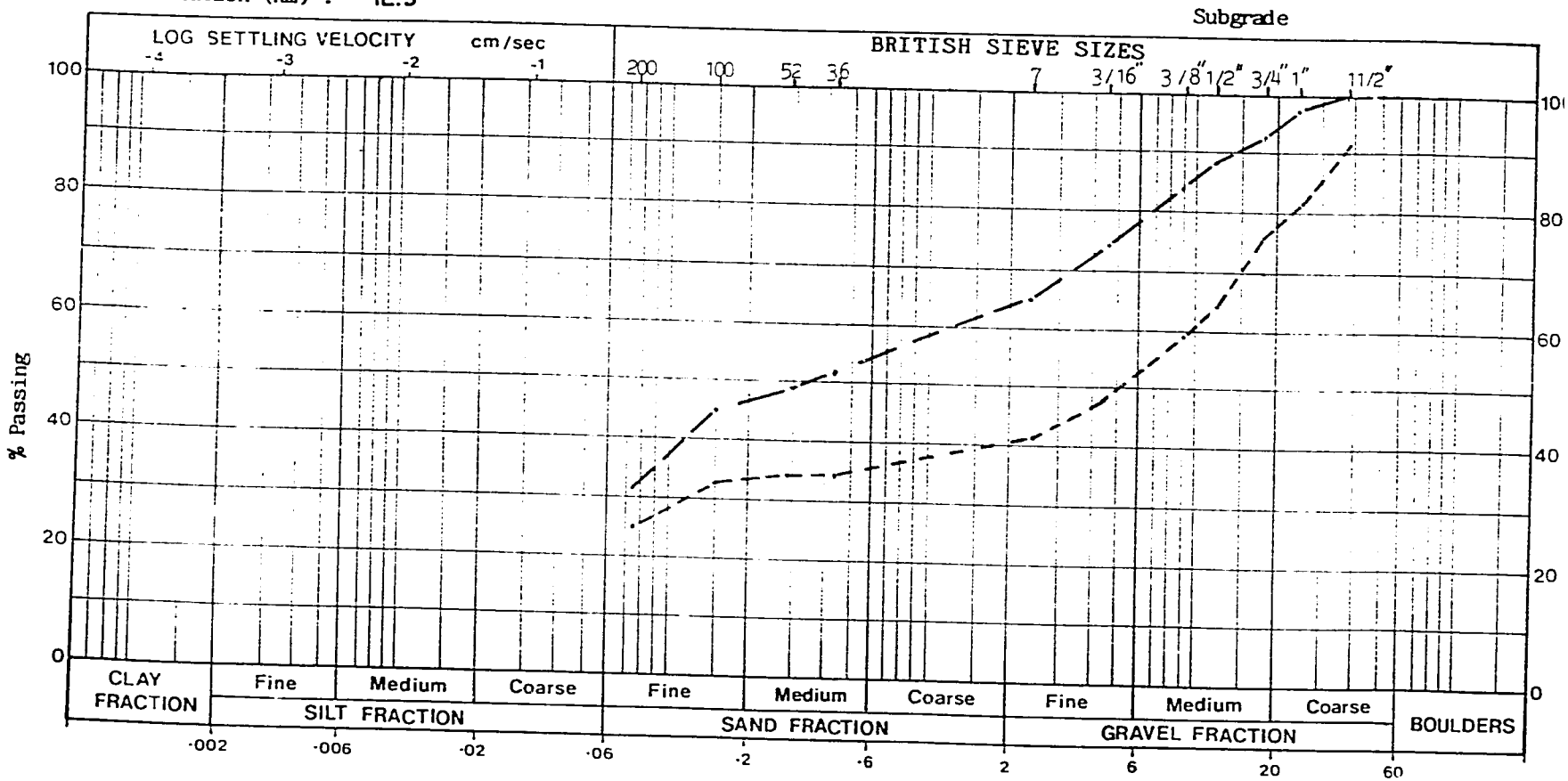
Location (km) : 12.0



KAFUE - LUSAKA ROAD

Test Pit No : 7

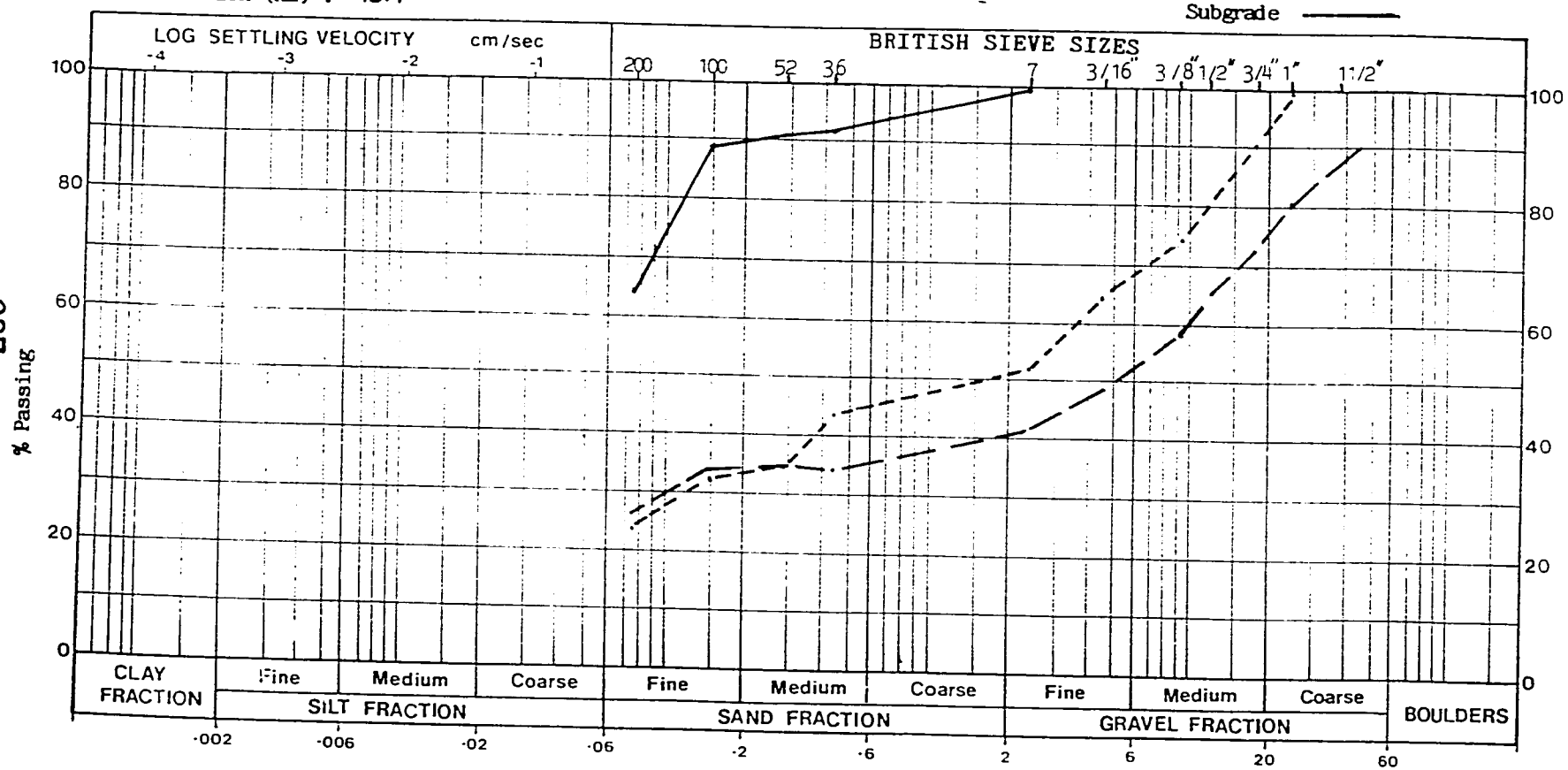
Location (km) : 12.5



KAFUE - LUSAKA ROAD

Test Pit No : 8

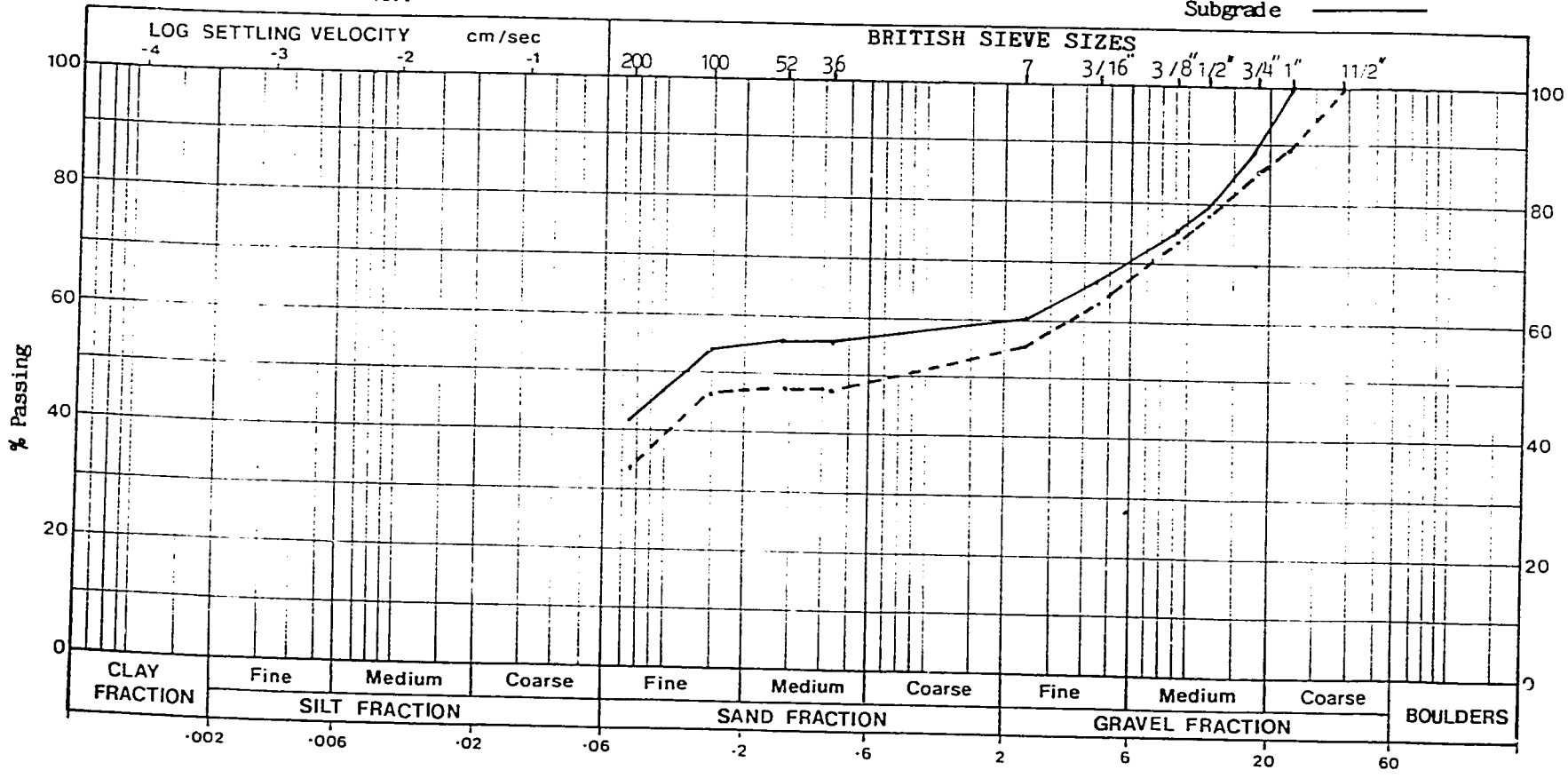
Location (km) : 13.7



KAFUE - LUSAKA ROAD

Test Pit No : 9

Location (km) : 16.9



KAFUE - LUSAKA ROAD

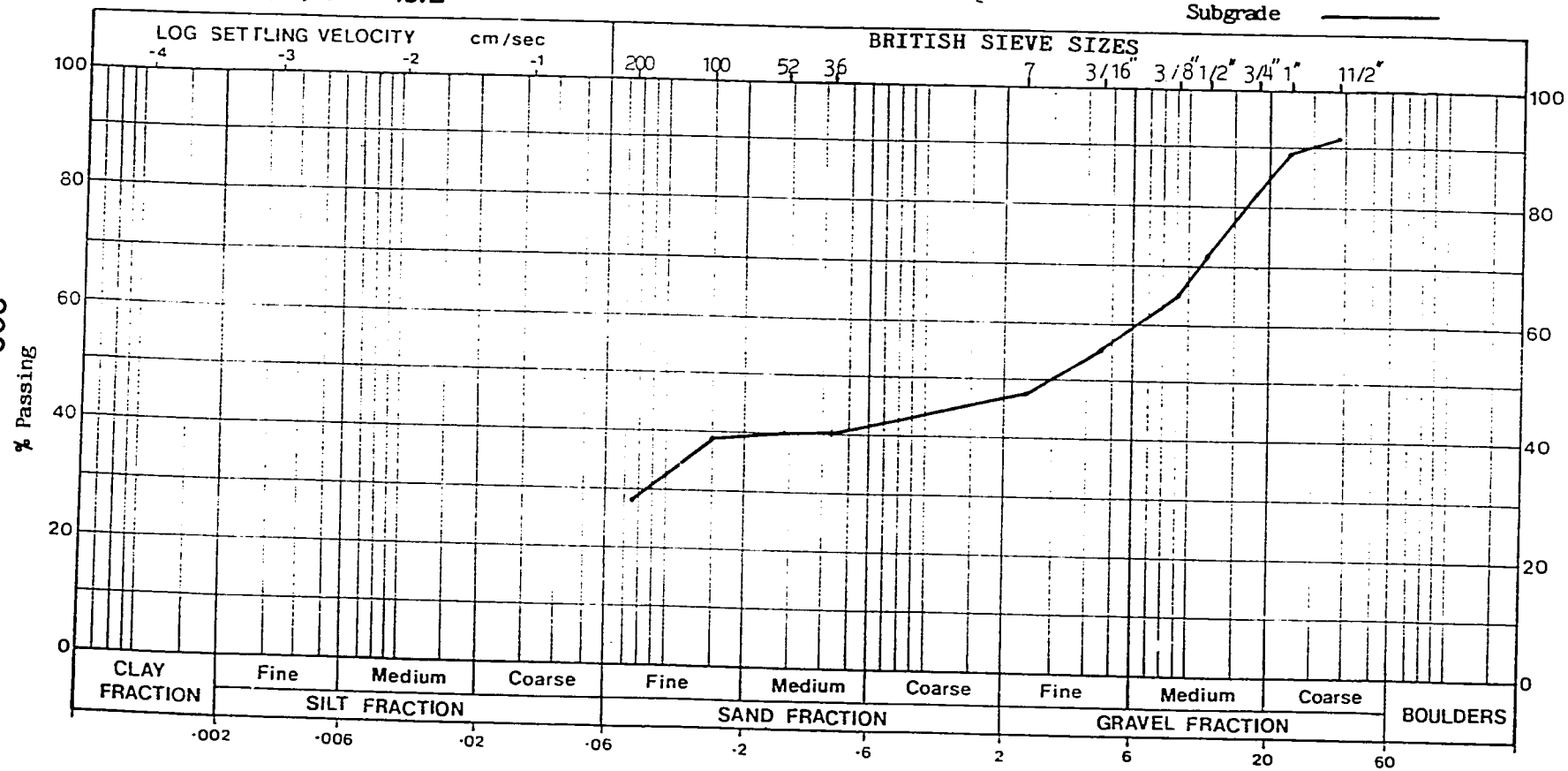
Test Pit No : 18

Location (km) : 18.2

Roadbase

Sub-base

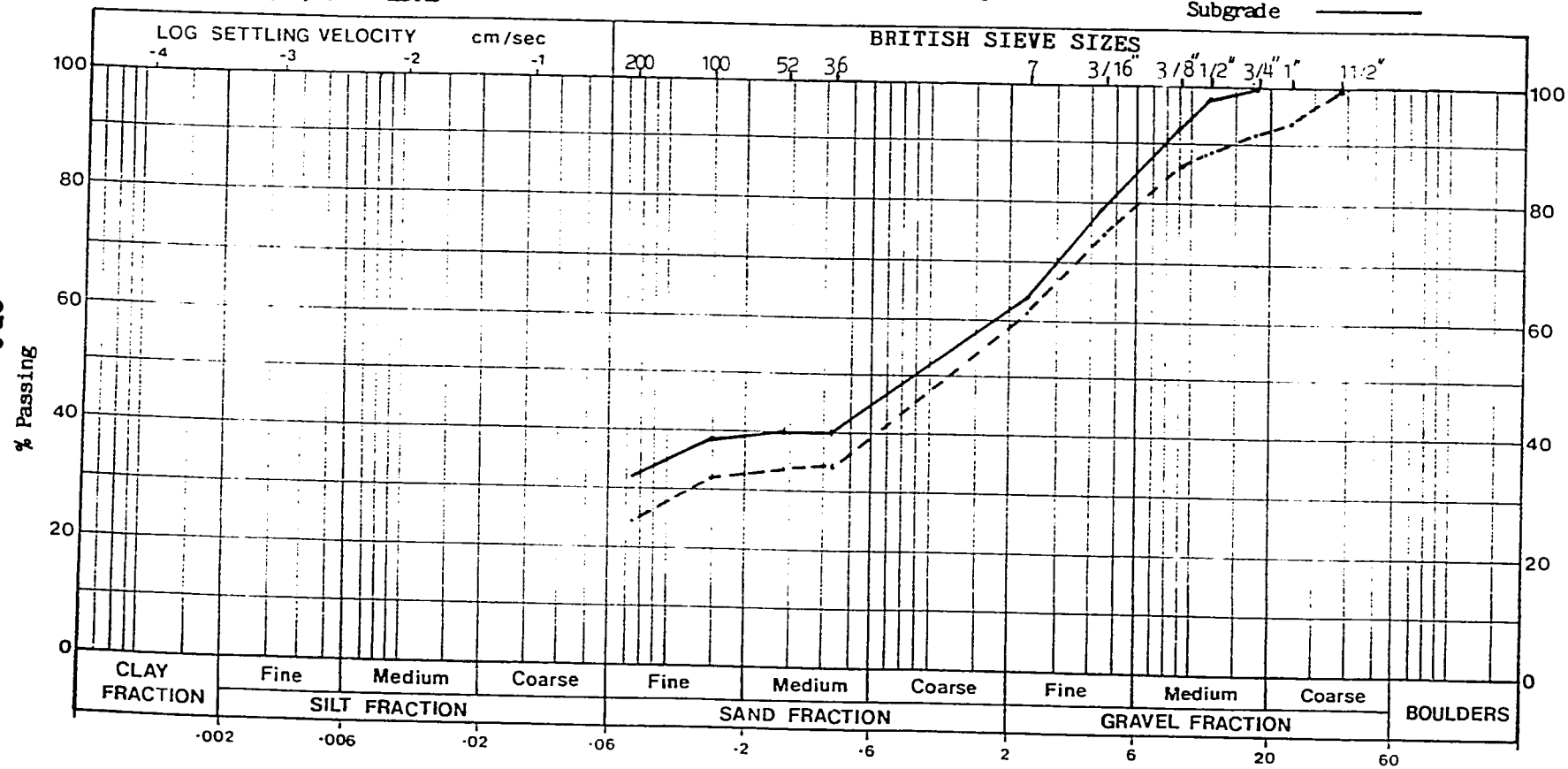
Subgrade



KAFUE - LUSAKA ROAD

Test Pit No : 13

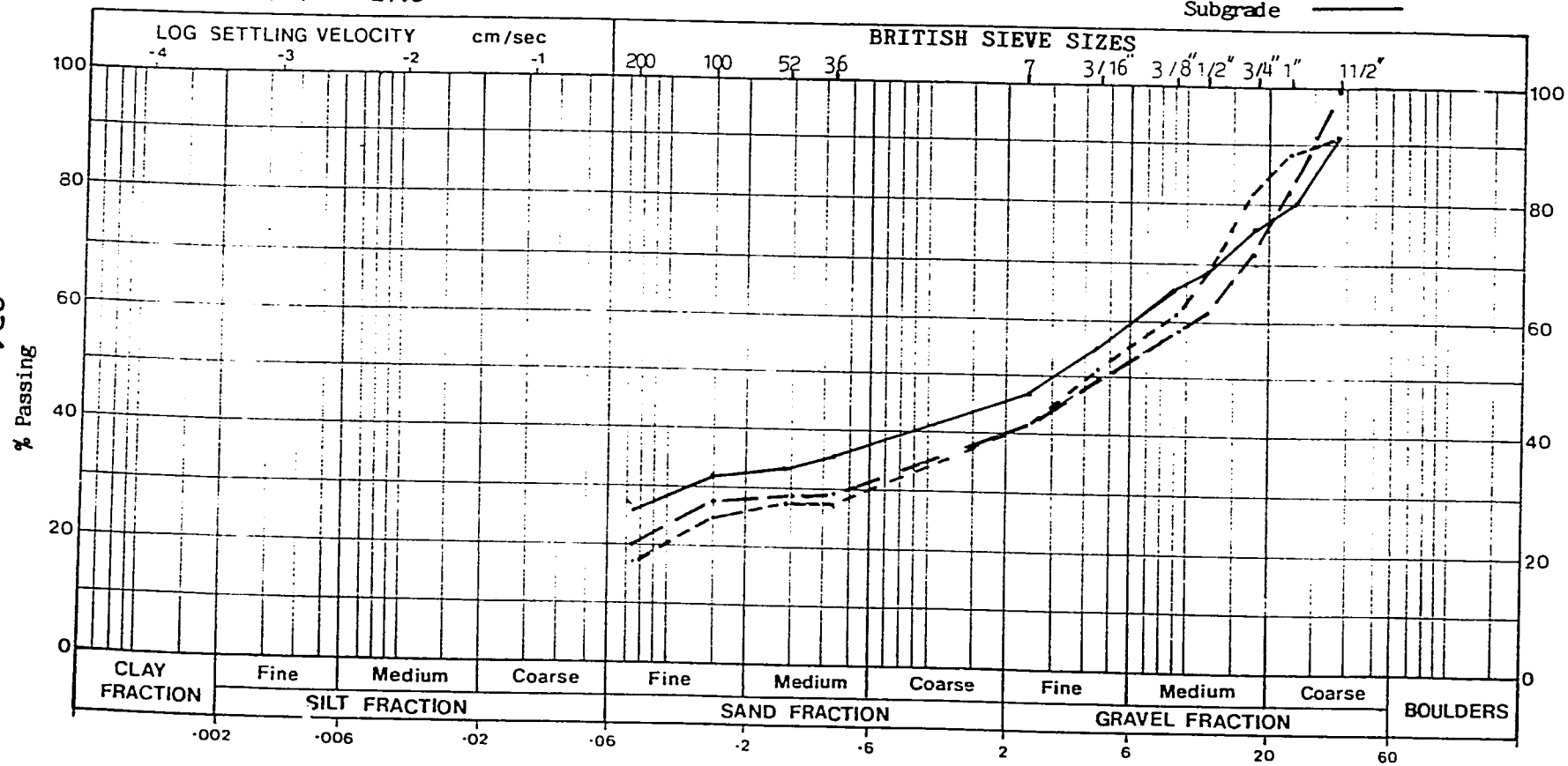
Location (km) : 20.2



KAFUE - LUSAKA ROAD

Test Pit No : 14

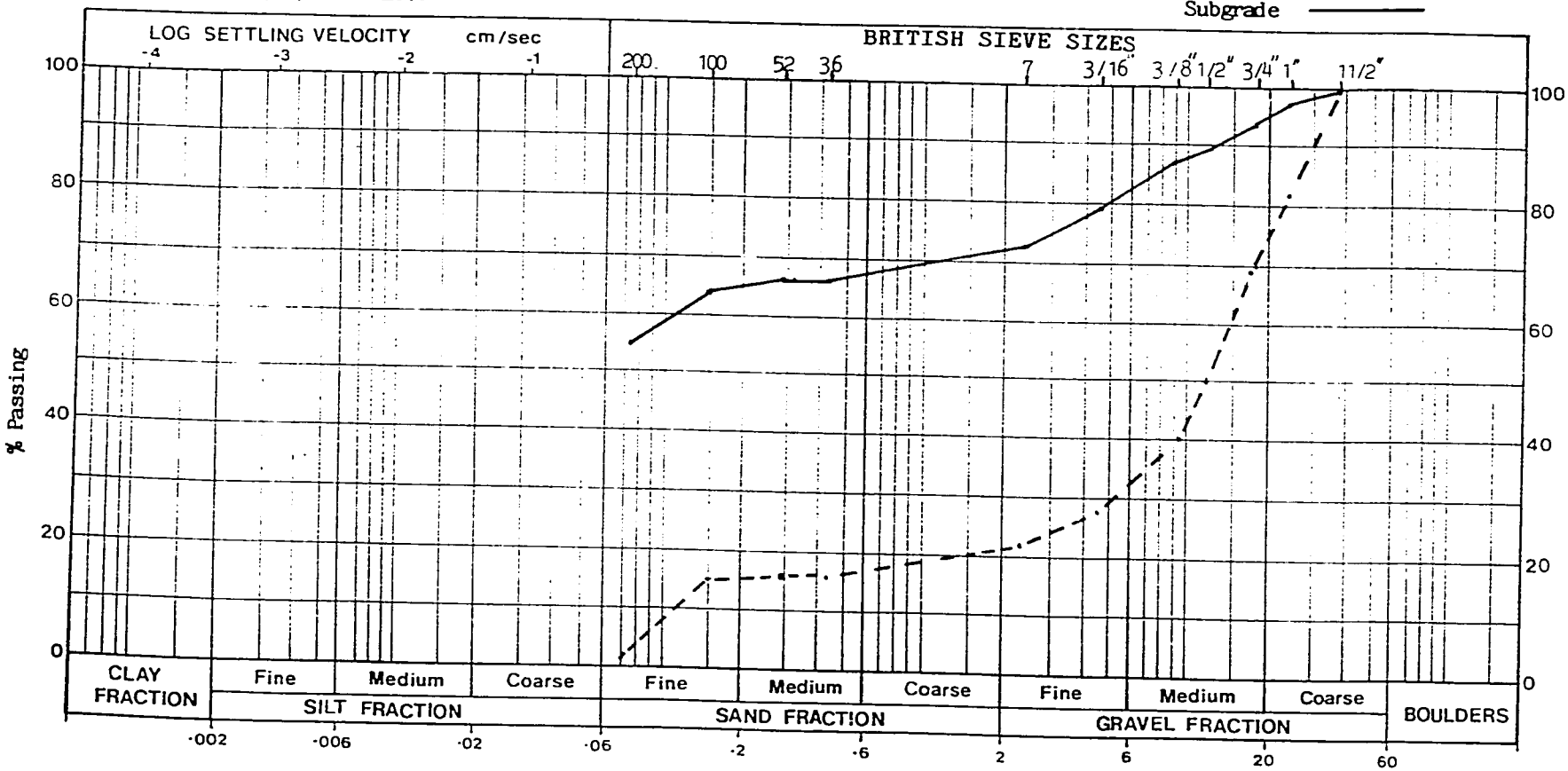
Location (km) : 27.0



KAFUE - LUSAKA ROAD

Test Pit No : 10

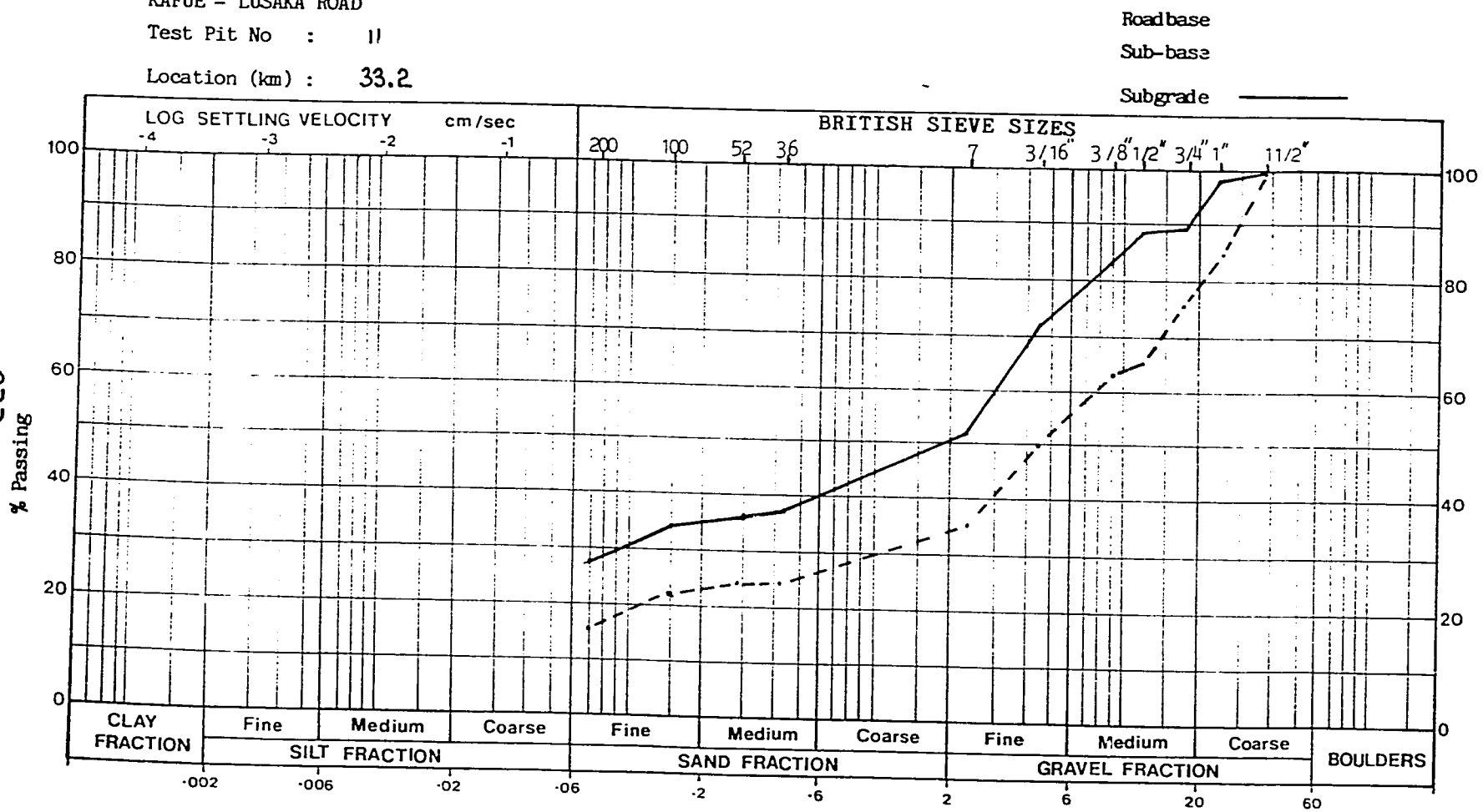
Location (km) : 28.8



KAFUE - LUSAKA ROAD

Test Pit No : 11

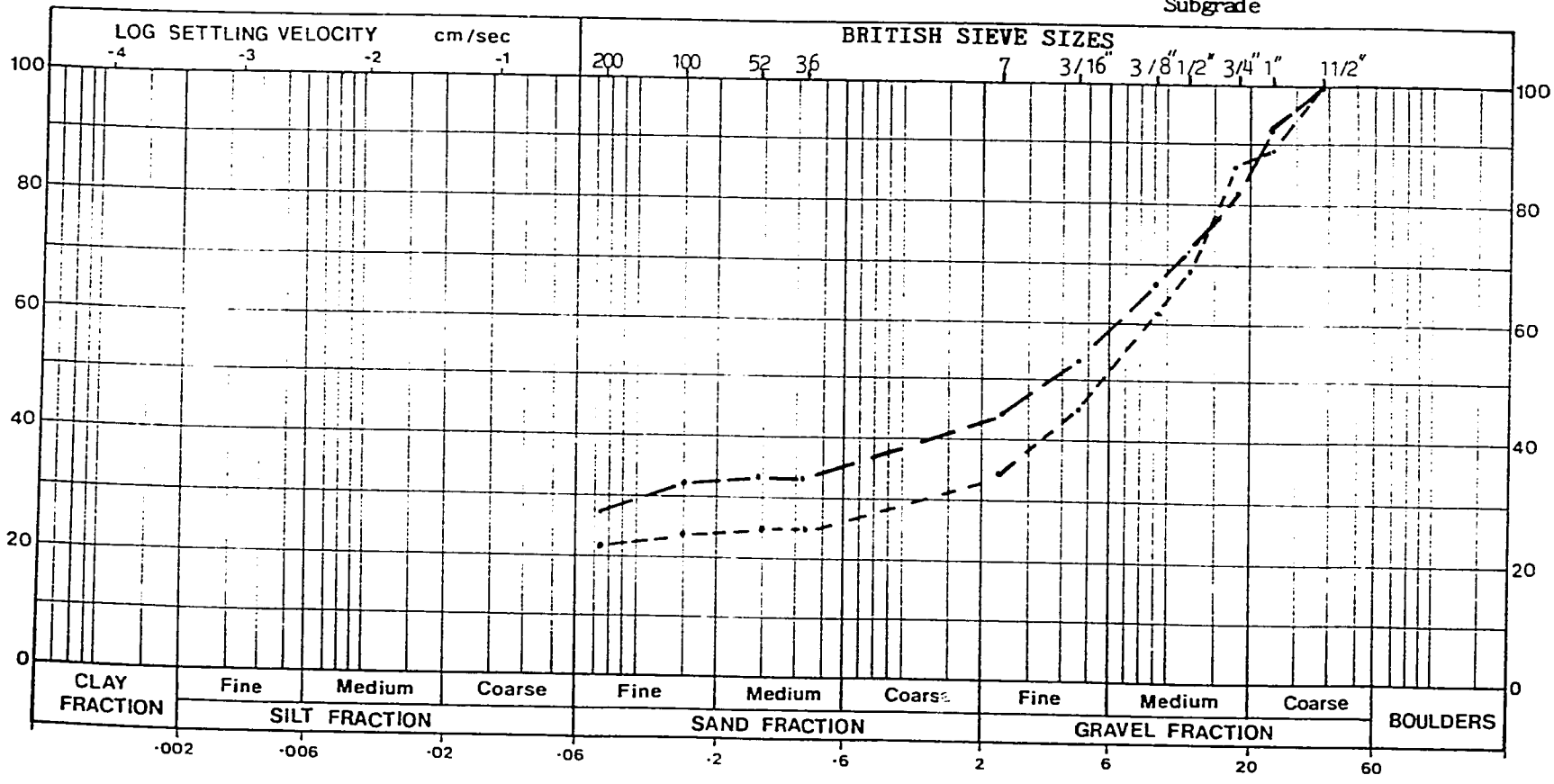
Location (km) : 33.2



KAFUE - LUSAKA ROAD

Test Pit No : 12

Location (km) : 43.1



KAFUE - LUSAKA ROAD

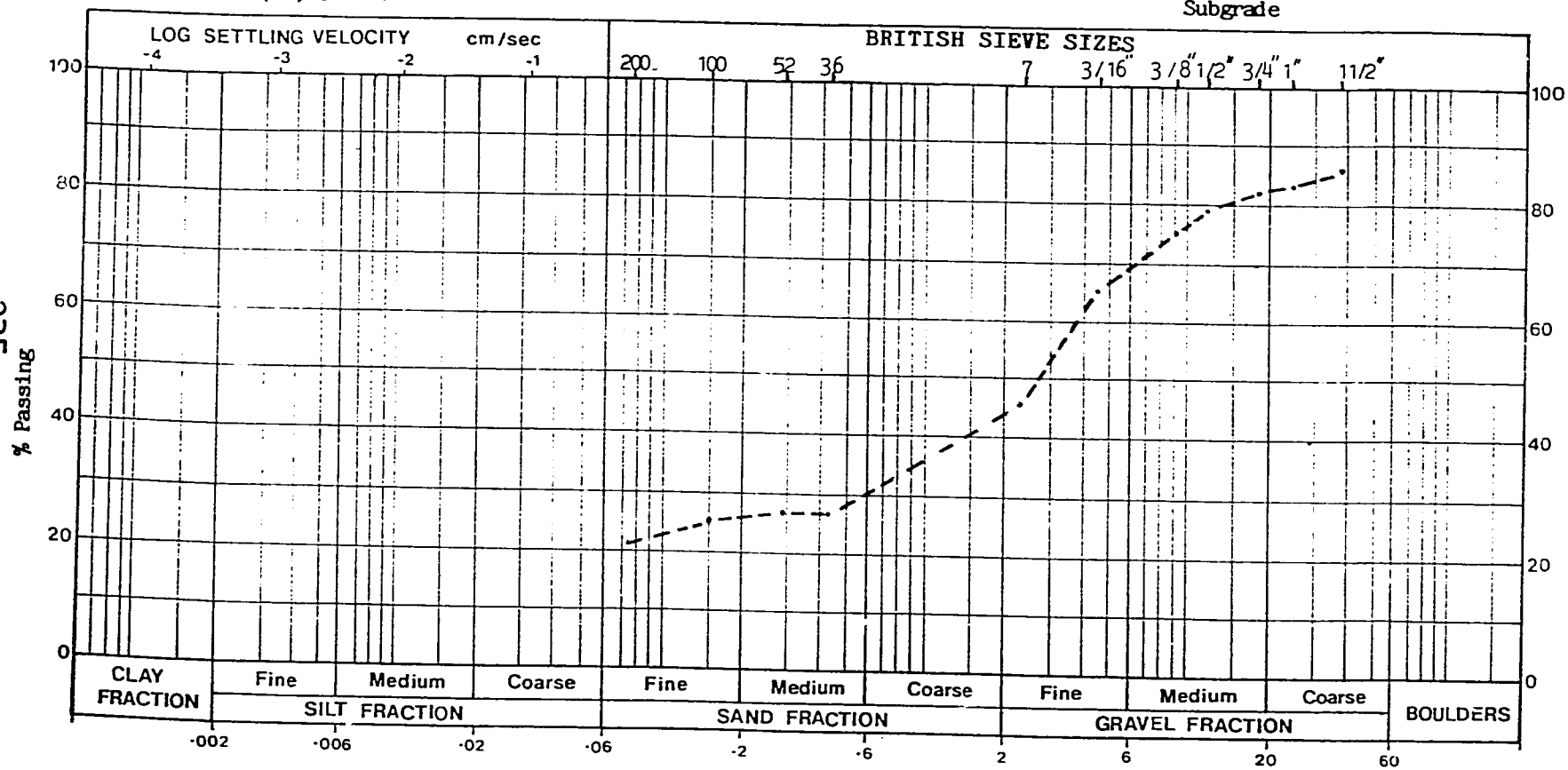
Test Pit No : 15

Location (km) : 45.5

Roadbase — — — —

Sub-base

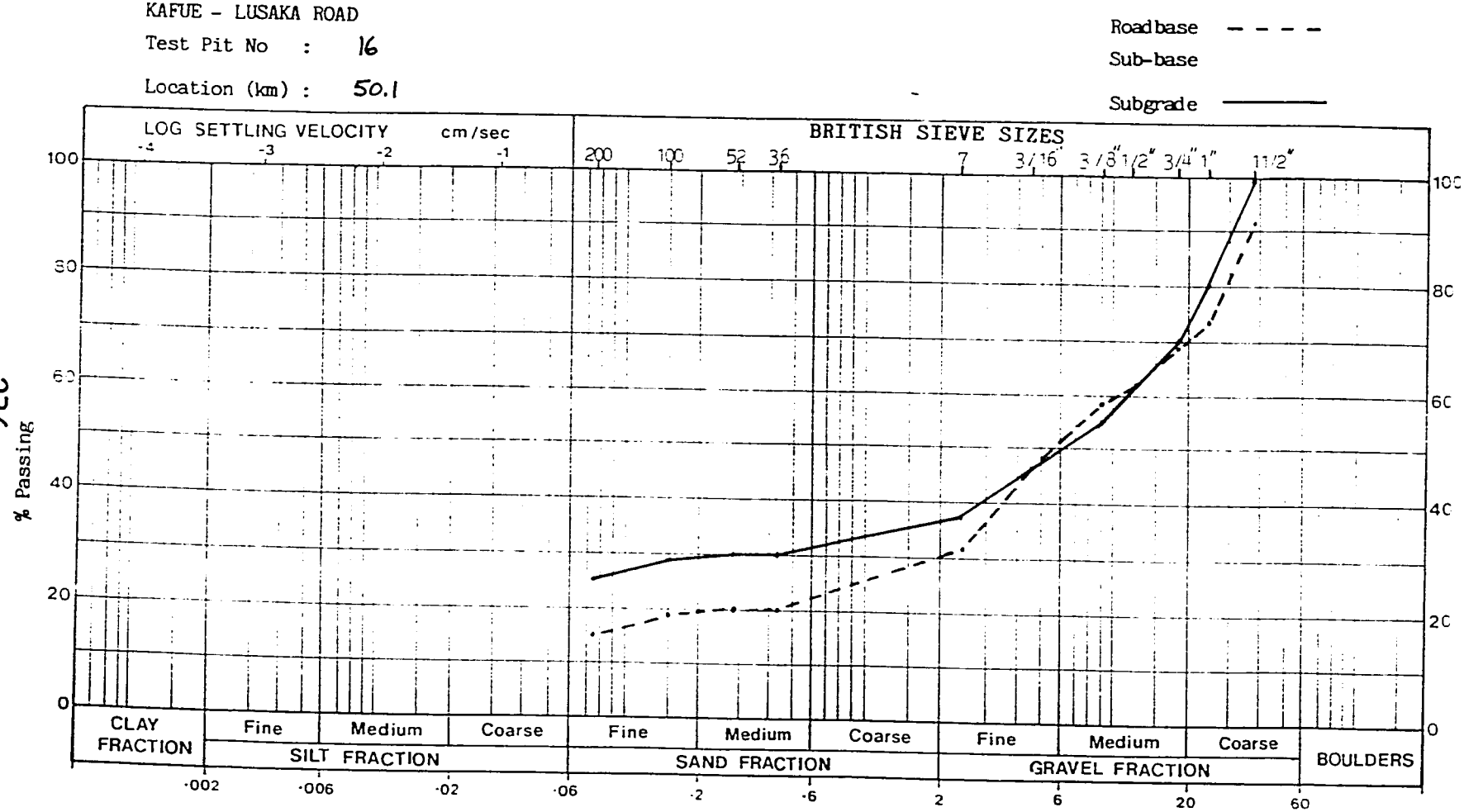
Subgrade



KAFUE - LUSAKA ROAD

Test Pit No : 16

Location (km) : 50.1



APPENDIX D

Typical Pan Analysis

```
*****************************        *****************************
*  Pavement  Analysis  Program  *        *  John  Burrow  &  Partners  *
*                               *        *                             *
*            P A N              *        *           Zambia            *
*                               *        *                             *
*                               *        *                Version 3.6A *
*****************************        *****************************
```

MEASURED SURFACE DEFLECTION : 80 (mm*100)

LOADING DETAILS

CONTACT PRESSURE : 590 kN/m2
RADIUS OF LOADED AREA : 130 mm (WHEEL LOAD : 31.3 kN)

PAVEMENT DETAILS (3 LAYERS)

LAYER No	LAYER THICKNESS mm	RESILIENT MODULUS MN/m2	POISSON'S RATIO
1	150	*	0.35
2	150	160	0.35
3	-	80	0.35

(* : deduced from axial deflection)

CORRECTION FACTORS

Deflections : Specified Factors

J	1	2
FDEF(J)	1.00000	0.80000

Strains : Specified Factors

J	1	2
FSTR(J)	1.00000	0.80000

Pavement Model B - Existing Pavement

Back Analysis to find Resilient Modulus of Roadbase Layer

BACK ANALYSIS TO FIND M(1)

ITERATION 1

LOG M(1): 2.5

P: 590 kN/m2 ;A: 130 mm ;R: 0 mm

HEDEF(2): 188.24265 ;HEDEF(3): 340.92722 ;

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	316.228	0.35	0.00000	150.00000	42.56692	25.10926	17.458
2	150	160.000	0.35	188.24265	338.24265	42.82758	26.81949	16.008
3	-	80.000	0.35	340.92722	-	53.26693	-	53.267
							TOTAL SURFACE DEFLECTION :	86.733

LOG D6: 1.938

ITERATION 2

LOG M(1): 3

P: 590 kN/m2 ;A: 130 mm ;R: 0 mm

HEDEF(2): 276.30236 ;HEDEF(3): 429.68586 ;

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	1000.000	0.35	0.00000	150.00000	13.46085	7.94025	5.521
2	150	160.000	0.35	276.30236	426.30236	31.89060	21.77113	10.119
3	-	80.000	0.35	429.68586	-	43.22638	-	43.226
							TOTAL SURFACE DEFLECTION :	58.866

LOG D6: 1.77

ITERATION 3

LOG M(1): 3.5

P: 590 kN/m2 ;A: 130 mm ;R: 0 mm

HEDEF(2): 405.55637 ;HEDEF(3): 559.96577 ;

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	3162.277	0.35	0.00000	150.00000	4.25670	2.51093	1.746
2	150	160.000	0.35	405.55637	555.55637	22.78993	16.98637	5.804
3	-	80.000	0.35	559.96577	-	33.71805	-	33.718
							TOTAL SURFACE DEFLECTION :	41.267

LOG D6: 1.616

ITERATION 1

LOG MULT: 1

P: 590 kN/m2 H: 150 mm R: 0 mm

HEDEF(2): 595.27531 :HEDEF(3): 751.19054 :

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	10000.000	0.35	0.00000	150.00000	1.34609	0.79403	0.552
2	150	160.000	0.35	595.27531	745.27531	15.90284	12.80115	3.102
3	-	80.000	0.35	751.19054	-	25.40620	-	25.406
							TOTAL SURFACE DEFLECTION :	29.060

LOG Dc: 1.463

ITERATION 5

LOG MULT: 1.5

P: 590 kN/m2 H: 150 mm R: 0 mm

HEDEF(2): 873.74496 :HEDEF(3): 1031.87017 :

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	31622.789	0.35	0.00000	150.00000	0.42567	0.25109	0.175
2	150	160.000	0.35	873.74496	1023.74496	10.96027	9.38017	1.580
3	-	80.000	0.35	1031.87017	-	18.61477	-	18.615
							TOTAL SURFACE DEFLECTION :	20.369

LOG Dc: 1.309

RESILIENT MODULUS FOR LAYER 1 : 10 MN/m2

CHECK DEFLECTION (offset: 0 mm)

P: 590 kN/m2 H: 150 mm R: 0 mm

HEDEF(2): 203.92005 :HEDEF(3): 356.72908 :

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	150	402.000	0.35	0.00000	150.00000	33.48470	19.75187	13.733
2	150	160.000	0.35	203.92005	353.92005	40.45066	25.76621	14.684
3	-	80.000	0.35	356.72908	-	51.17127	-	51.171
							TOTAL SURFACE DEFLECTION :	79.589

CHECK : (COMPUTED DEFLECTION : 80 (mm/100)
(F MEASURED VALUE 80

```
*****************************        *****************************
* Pavement Analysis Program *        *  John Burrow & Partners   *
*                           *        *                           *
*          P A M            *        *          Zambia           *
*                           *        *                           *
*                           *        *              Version 3.6A *
*****************************        *****************************
```

LOADING DETAILS

CONTACT PRESSURE : 590 kN/m2
RADIUS OF LOADED AREA : 130 mm (WHEEL LOAD : 31.3 kN)

PAVEMENT DETAILS (4 LAYERS)

LAYER No	LAYER THICKNESS mm	RESILIENT MODULUS MN/m2	POISSON'S RATIO
1	70	2500	0.35
2	150	400	0.35
3	150	160	0.35
4	-	80	0.35

CORRECTION FACTORS

Deflections : Specified Factors

J	1	2	3
FDEF(J)	1.00000	0.80000	0.80000

Strains : Specified Factors

J	1	2	3
FSTR(J)	1.00000	0.80000	0.80000

SURFACE DEFLECTION UNDER SPECIFIED LOAD. 31.3 kN

P: 590 kN/m2 ;A: 130 mm ;R: 0 mm
HEDEF(2): 128.94110 ;HEDEF(3): 302.86505 ;HEDEF(4): 532.77660 ;

LAYER	H mm	M MN/m2	U	ZTOP mm	ZBOT mm	DTOP mm/100	DBOT mm/100	DEFLECTION mm/100
1	70	2500.000	0.35	0.00000	70.00000	5.38434	4.38890	0.995
2	150	400.000	0.35	128.94110	278.94110	21.61432	12.65579	8.959
3	150	160.000	0.35	302.86505	452.86505	29.51670	20.58764	8.929
4	-	80.000	0.35	532.77660	-	35.35046	-	35.350
							TOTAL SURFACE DEFLECTION :	54.234

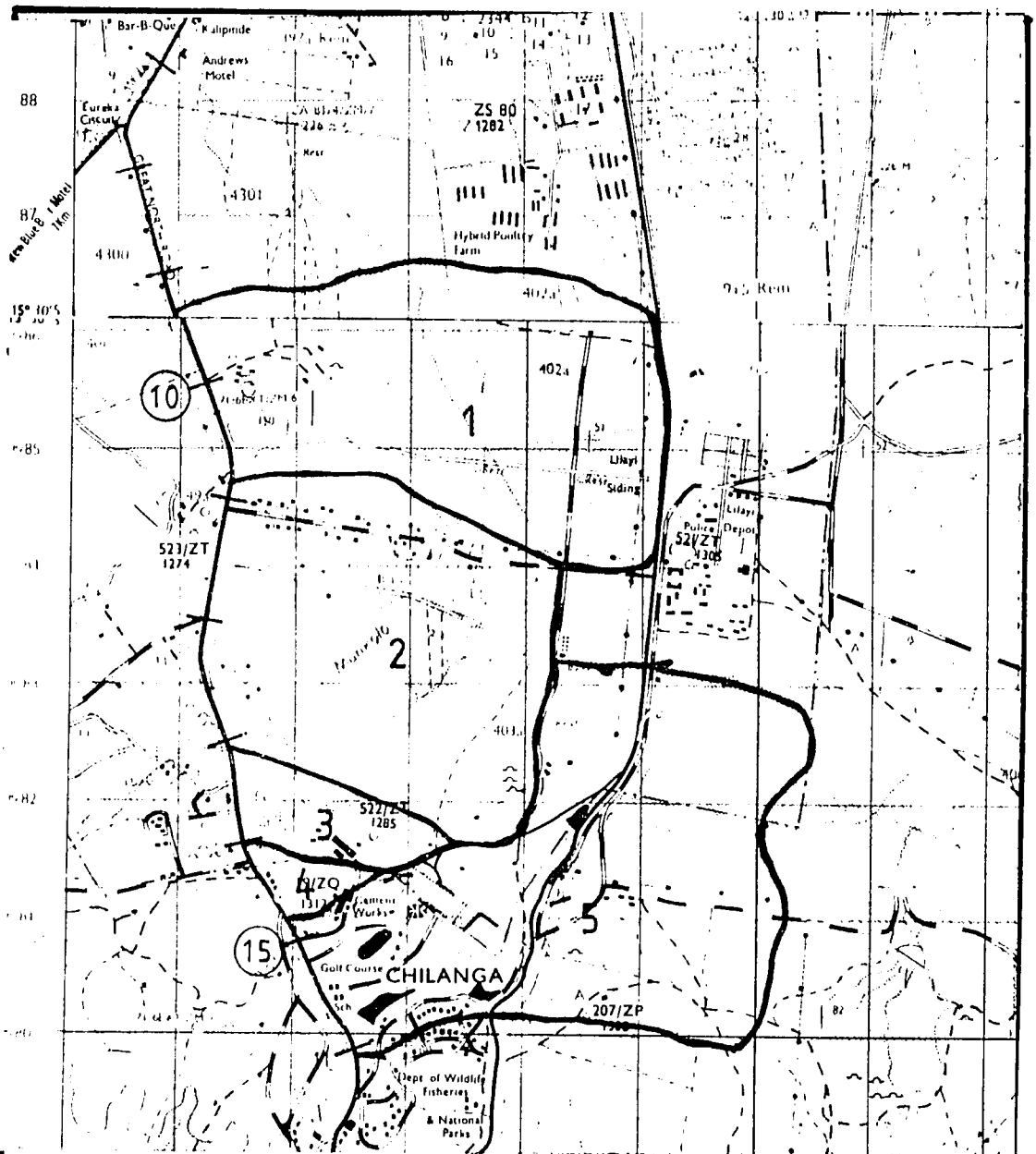
COMPUTED SURFACE DEFLECTION : 54 (mm/100)

Pavement Model B - with 70 mm overlay

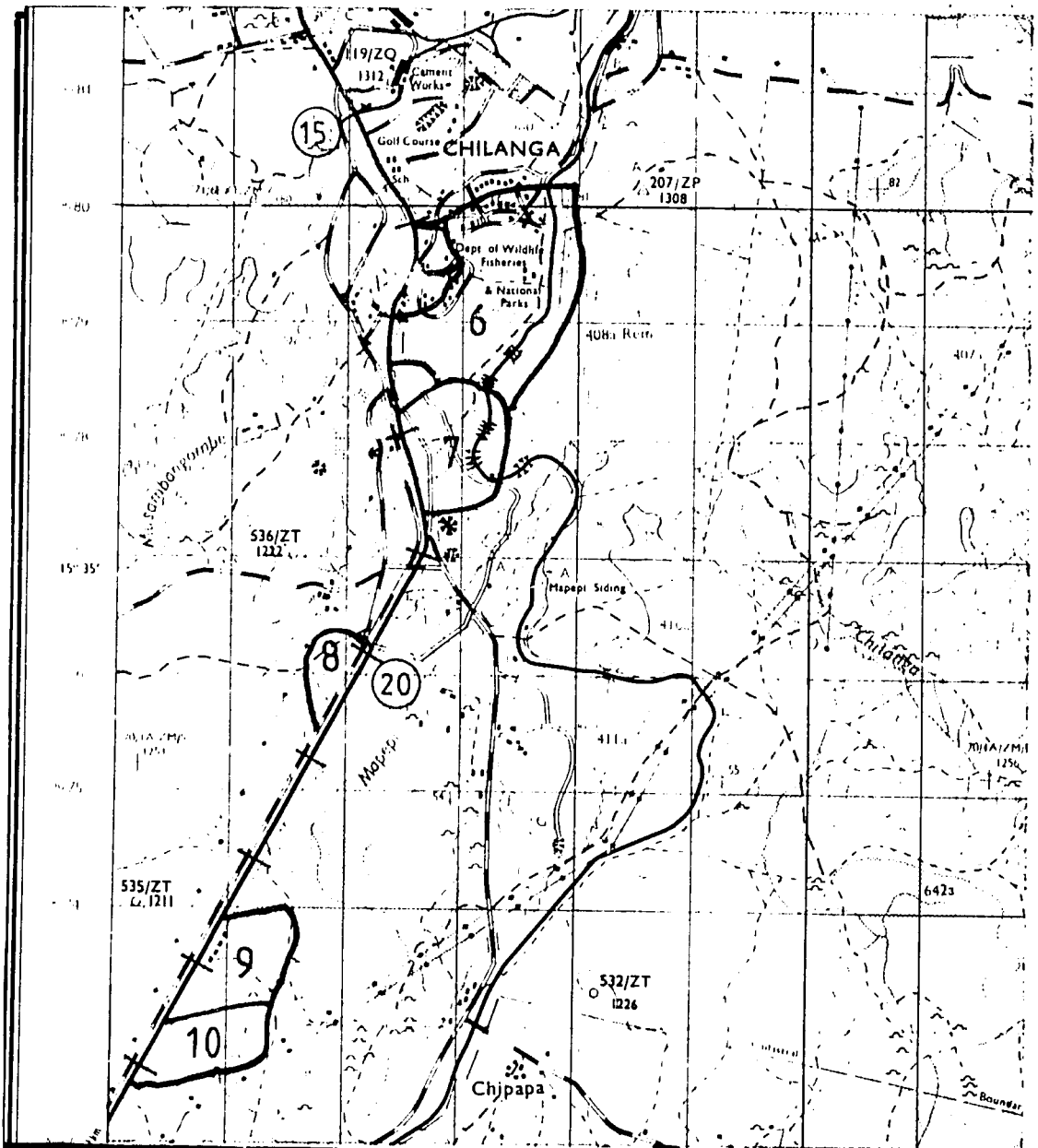
Prediction of surface deflection after overlaying

APPENDIX E

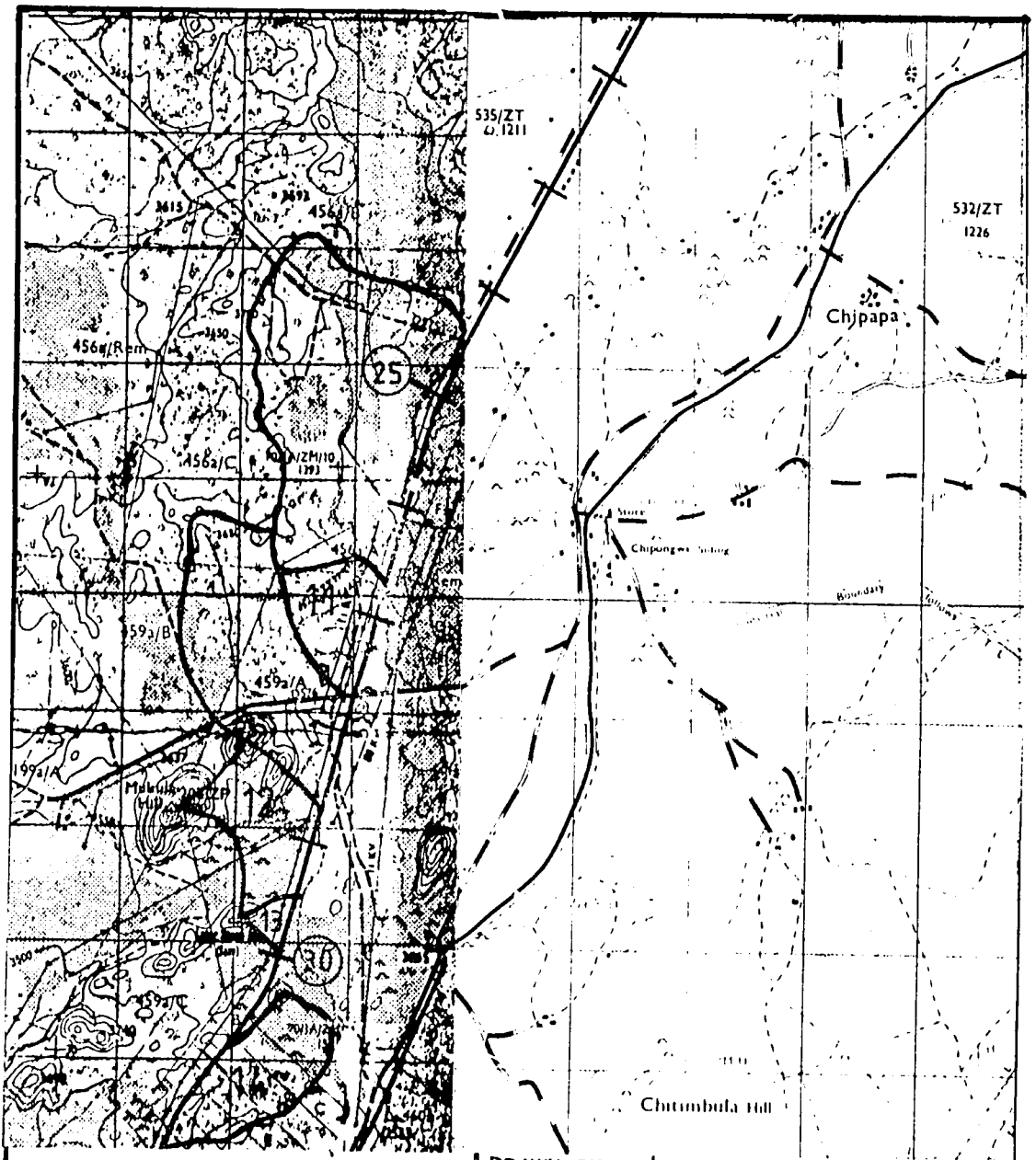
Schedule of Catchment Areas



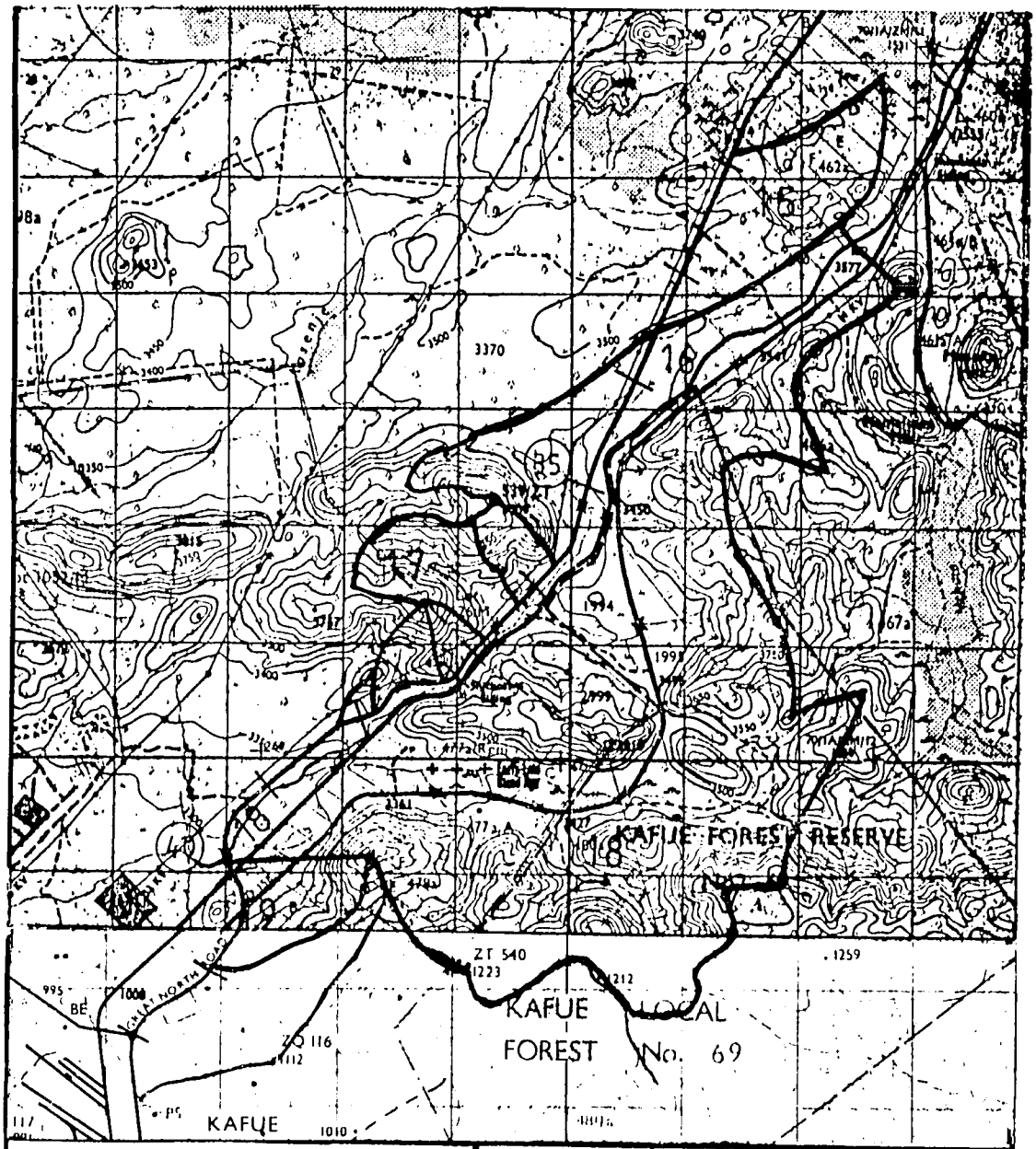
db John Burrow & Partners CONSULTING ENGINEERS			DRAWN BY:		DRG. No.			
			CHECKED BY:		DATE:			
CULVERT No.	1	2	3	4	5			
CHAINAGE	10·3	12·5	13·5	14·7	15·9			
CATCHMENT AREA	760	730	200	30	900			
Q	26	25	7	1	31			
COMMENT								



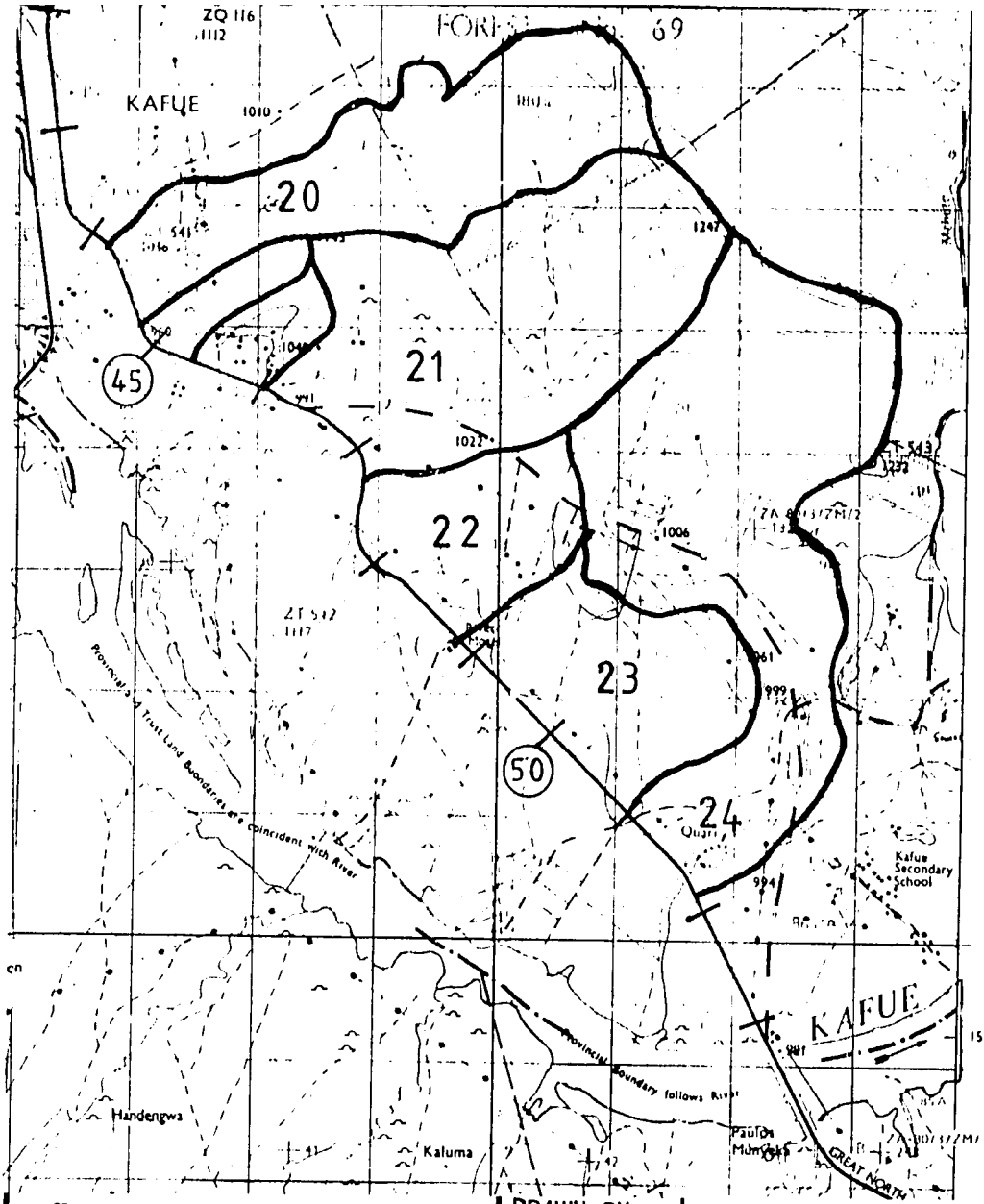
db John Burrow & Partners CONSULTING ENGINEERS			DRAWN BY:		DRG. No.			
			CHECKED BY:		DATE:			
CULVERT No.	6	7	8	9	10			
CHAINAGE	17·2	18·2	20·1	23·45	23·7			
CATCHMENT AREA	200	100	20	130	60			
Q	7	3	0·7	4	2			
COMMENT								



db John Burrow & Partners CONSULTING ENGINEERS			DRAWN BY:		DRG. No.			
			CHECKED BY:		DATE:			
CULVERT No.	11	12	13	14				
CHAINAGE	26·8	28·9	29·85	31·8				
CATCHMENT AREA	400	200	40	90				
Q	13	7	1	3				
COMMENT								



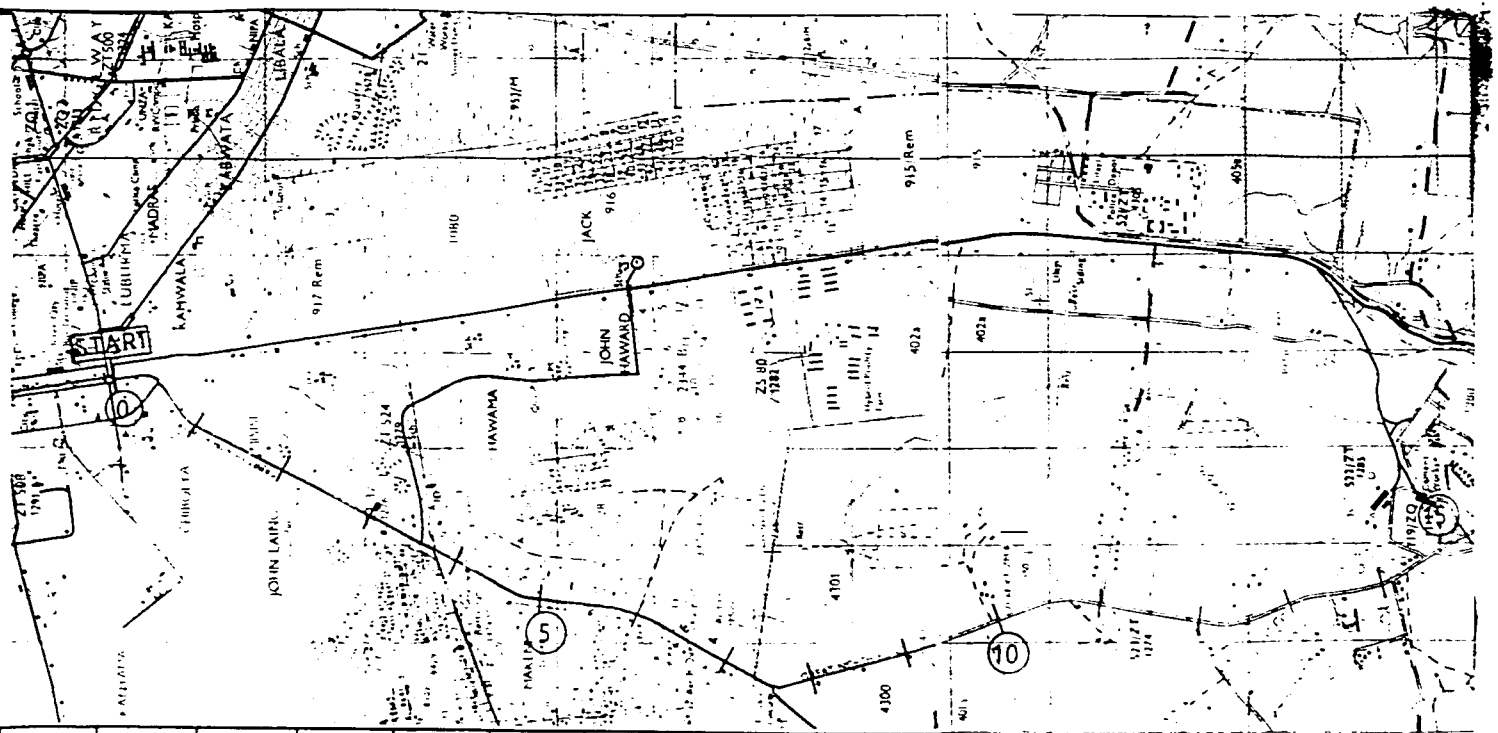
db John Burrow & Partners CONSULTING ENGINEERS			DRAWN BY:		DRG. No.			
			CHECKED BY:		DATE:			
CULVERT No.	15	16	17	18	19			
CHAINAGE	32·7	34·7	36·55	39·8	40·8			
CATCHMENT AREA	160	142	98	2370	200			
Q	5	5	3	80	7			
COMMENT								



db John Burrow & Partners CONSULTING ENGINEERS			DRAWN BY:		DRG. No.			
			CHECKED BY:		DATE:			
CULVERT No.	20	21	22	23	24			
CHAINAGE	44·15	46·1	48·4	50·4	51·5			
CATCHMENT AREA	440	605	215	290	760			
Q	15	20	7	10	26			
COMMENT								

APPENDIX I'

Schedule of Pavement Rehabilitation Proposals

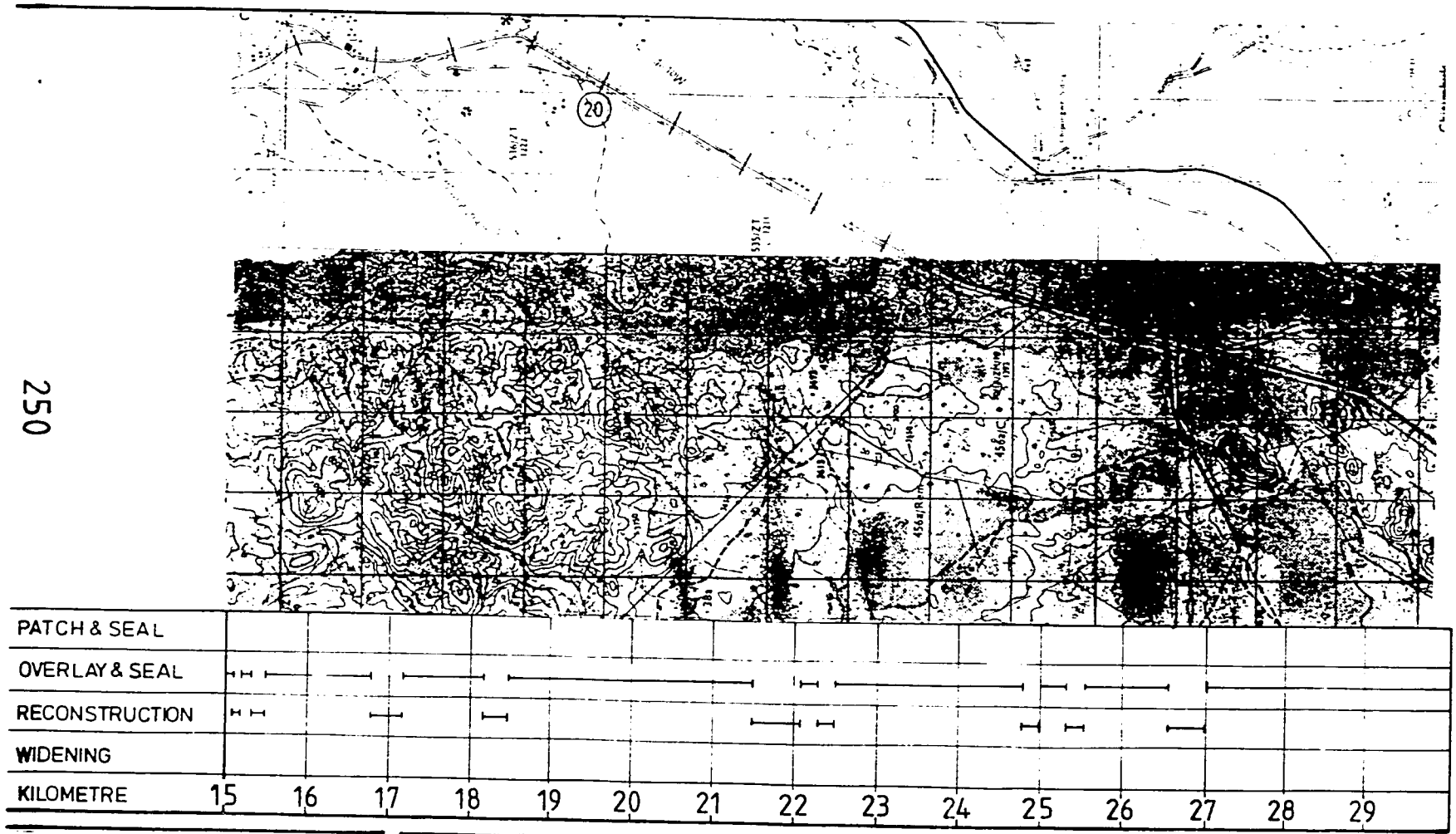


PATCH & SEAL															
OVERLAY & SEAL															
RECONSTRUCTION															
WIDENING															
KILOMETRE	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4

John Burrow & Partners
CONSULTING ENGINEERS

KAFUE-LUSAKA ROAD
FEASIBILITY STUDY/COST ANALYSIS

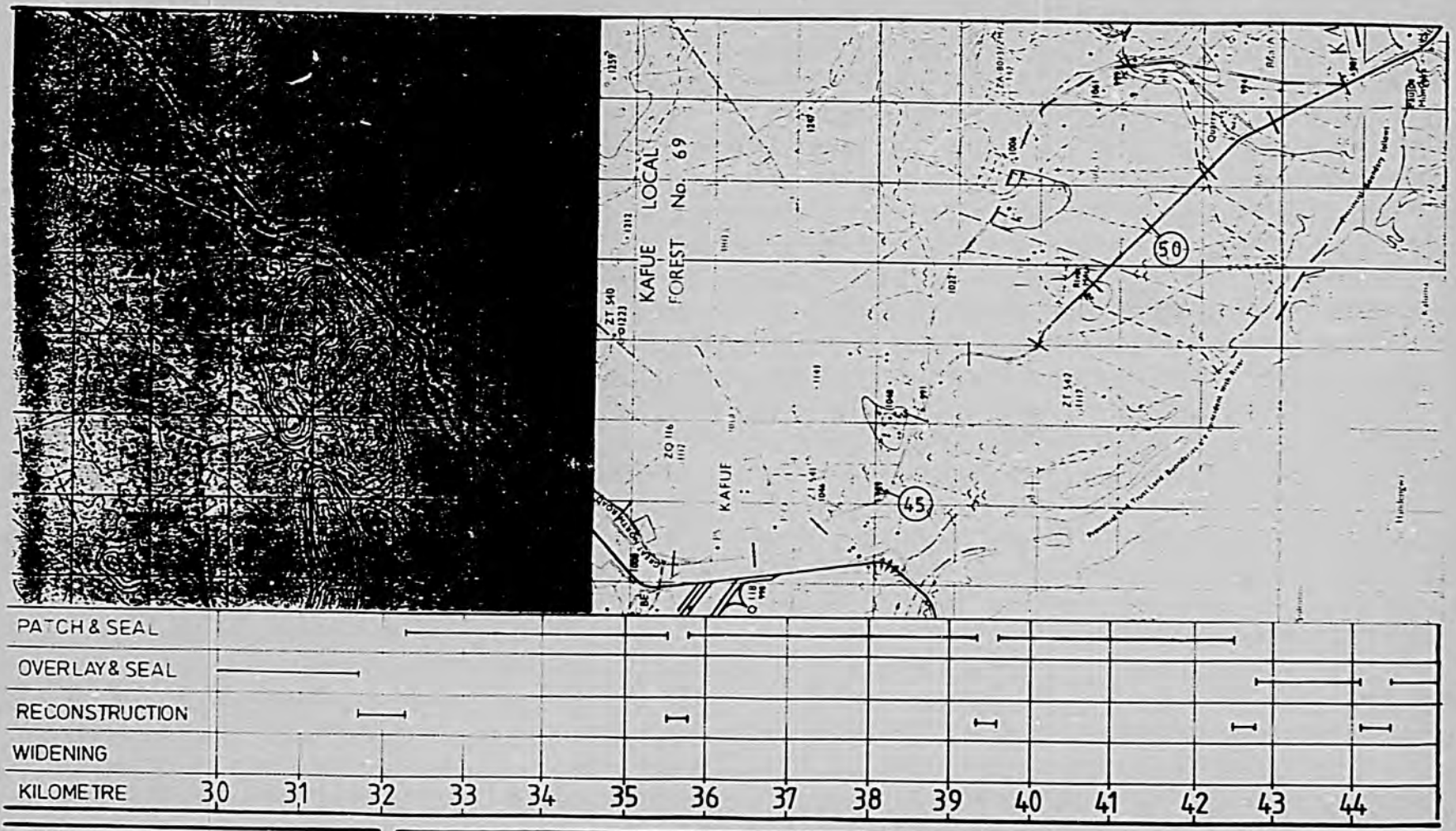
REHABILITATION PROPOSALS
KM 0+000—KM 15+000



John Burrow & Partners

KAFUE-LUSAKA ROAD
FEASIBILITY STUDY/COST ANALYSIS

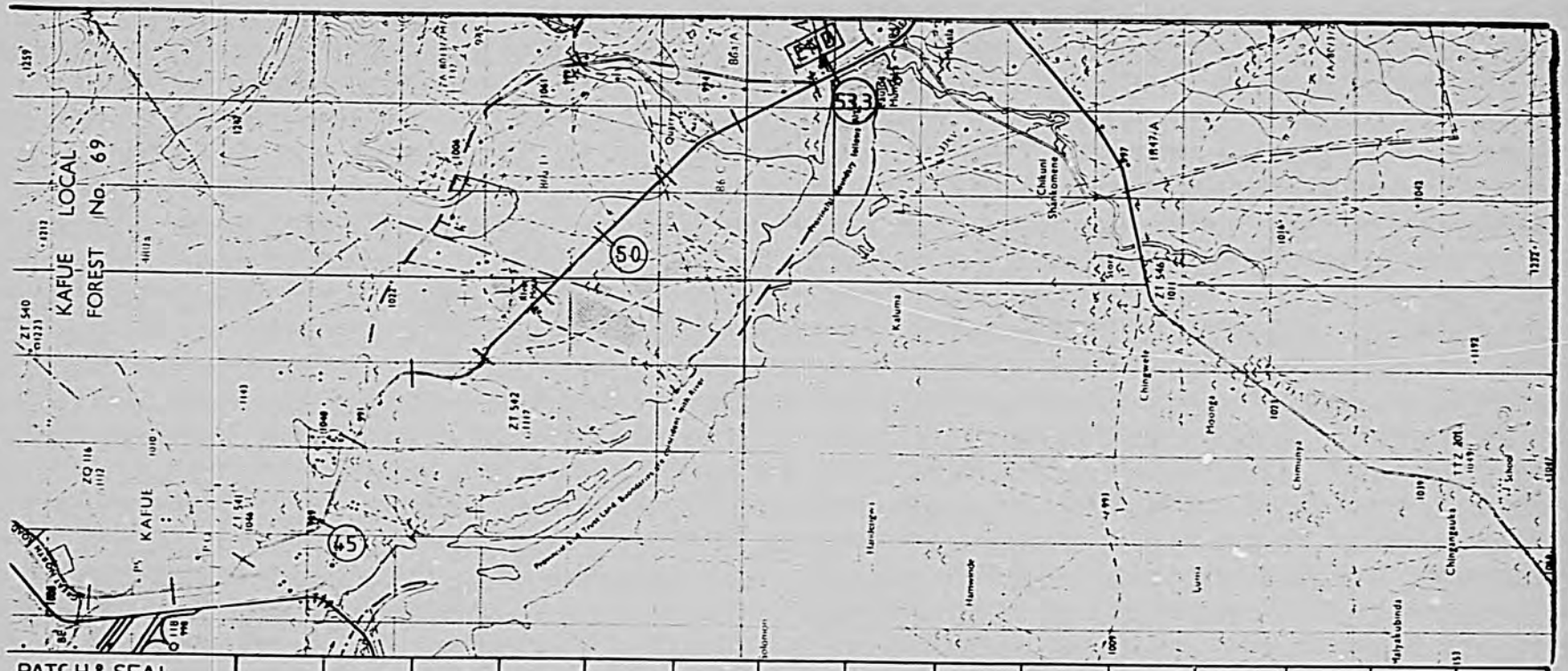
REHABILITATION PROPOSALS
KM 15+000–KM 30+000



John Burrow & Partners

KAFUE–LUSAKA ROAD
FEASIBILITY STUDY/COST ANALYSIS

REHABILITATION PROPOSALS
KM 30+000–KM 45+000



PATCH & SEAL									
OVERLAY & SEAL									
RECONSTRUCTION									
WIDENING									
KILOMETRE	45	46	47	48	49	50	51	52	53

John Burrow & Partners
CONSULTING ENGINEERS

KAFUE-LUSAKA ROAD
FEASIBLITY STUDY/COST ANALYSIS

REHABILITATION PROPOSALS
KM 45+000-KM 53+300

APPENDIX G

Preliminary Bills of Quantities

Presented in a Separate Volume

APPENDIX H

Schedule of Cross Section Data

Presented in Separate Volume

APPENDIX I

The World Banks Highway Design and Maintenance Standards Model (HDM3)

THE WORLD BANK'S HDM3 MODEL

by

Dr H R Kerali
University of Birmingham

1 THE DEVELOPMENT OF ROAD INVESTMENT APPRAISAL MODELS FOR DEVELOPING COUNTRIES

The first step towards the development of fully integrated road investment appraisal models for developing countries was made by the World Bank in 1968. In conjunction with research institutions in the United States, and the TRRL in Britain, a study was initiated to develop a system of evaluating the effects of construction and maintenance standards on vehicle operating costs for low volume roads. The first model to emerge was the Highway Cost Model (HCM) developed at the Massachusetts Institute of Technology (MIT) in 1971. Although the HCM was a considerable advance over other models existing at the time, its main impact was that it highlighted areas where more research was needed particularly the development of relationships pertaining to the environment in developing countries.

In 1973, the TRRL in collaboration with the World Bank undertook a major field study in Kenya with the objective of developing relationships for road deterioration and road user costs. The results of this study were used to calibrate a prototype Road Transport Investment Model (RTIM) for evaluating the costs of road construction, annual road maintenance, and vehicle operation. Further co-operation between the World Bank, TRRL and the MIT resulted in an extended version of the models HCM and RTIM and was known as the Highway Design and Maintenance standards model (HDM). An extensive research study in Brazil sponsored by the World Bank was concluded in 1986. The results from this study were used to derived new relationships for pavement deterioration and vehicle operation. The latest version of the World Bank's model, HDM3 incorporates the results from this study together with the relationships derived from earlier studies in Kenya, India and the Caribbean. The RTIM and HDM models have been used extensively in road investment projects in Africa, Asia, and Central America.

2 THE WORLD BANK'S HDM3 MODEL

The main difference between the World Bank's HDM3 model and the TRRL RTIM2 model is that HDM3 is a multiple link-alternative model. Several road links can be analysed in one program run, where a link is defined as a length of road carrying uniform traffic loading. The model is thus suitable for analysing transport costs for a network of roads.

The prime function of an investment appraisal model such as HDM3 is to calculate the components of the transport cost in each year of an analysis period and conduct an economic comparison of different construction alternatives. To achieve this, the HDM3 model executes four modules ; a data input module, a performance modelling module, an economic evaluation module, and a report generation module. A summary flow chart of the HDM3 model is illustrated in Figure 1.

3 DATA INPUT AND DIAGNOSTICS MODULE

The HDM3 model can evaluate the transport costs incurred on up to twenty different road links, each having up to ten homogeneous sections with different pavement design standards and environmental conditions. A road link is defined by the traffic loading, thus all homogeneous sections within a link must carry the same traffic loading. Up to thirty different maintenance standards may be input to the model in the form of combinations of maintenance activities including reconstruction. Each road section within a link must be assigned at least one maintenance standard. Construction alternatives may also be specified for each of the road sections.

The vehicle characteristics input to the model (e.g. axle loads, vehicle prices, vehicle utilisation etc) apply to all road links being analysed. Traffic characteristics however are assigned to the road links separately in terms of traffic compositions and growth rates. Other costs and benefits which cannot be calculated within the model may be specified directly. These may include road user savings due to a reduction in accidents and discomfort, or values assigned to socio-economic benefits such as increased agricultural income. All unit costs to the model are input in financial, economic, and foreign exchange terms.

Up to five project alternatives may be specified for each road link. Each project alternative (referred to as a link-alternative) consists of the following:

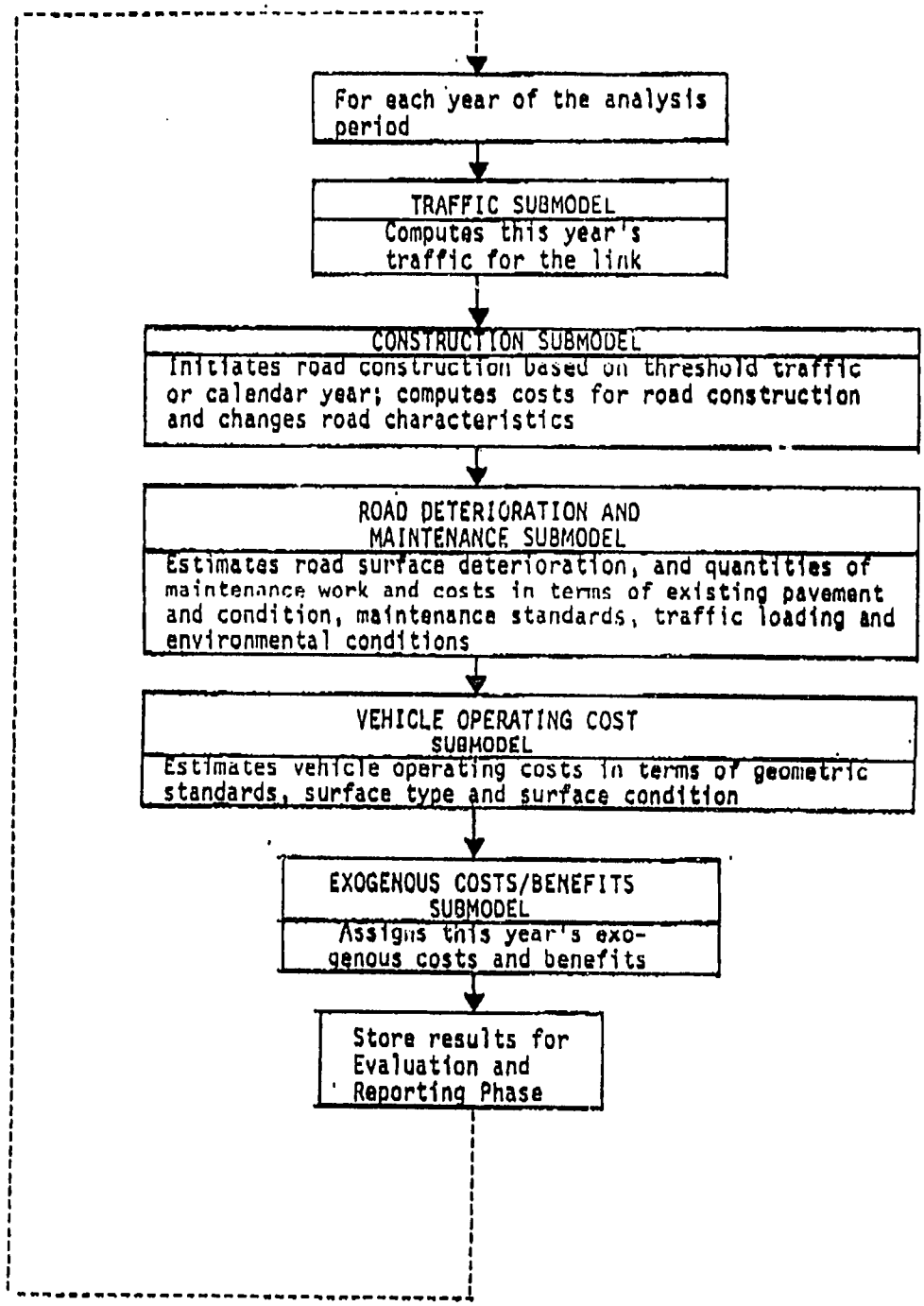


Figure 1 Simulation of a link-alternative in HDM3

(i) traffic loading group,
(ii) construction option (if required)
(iii) capital expenditure (if required) for mobilising maintenance plant, vehicles, spares, workshops, manpower etc.
(iv) maintenance standards consisting of combinations of activities such as overlays, reconstruction, and surface treatments for each link-section.

The modelling and evaluation phases of the HDM program are carried out for each of the link-alternatives defined by the above specifications. For ease of adaptation to performance trends in other areas, calibration factors may be input for each type of defect.

All data input to the model is checked and validated by a data diagnostics routine which compares the data input formats and data range limits with those required by the model. Warnings and error messages are given if the input data does not fully meet the requirements of the model.

4 THE MODELLING MODULE

The modelling module shown in Figure 1 is applied to all road sections specified in the data input module. The results of the modelling phase are stored for each year of analysis and for each link-alternative in direct access files for retrieval by the evaluation and reporting phase. For each year of the analysis period, the following submodels within HDM3 are applied :

4.1 The traffic submodel

The function of the traffic submodel is to calculate the traffic loading in the current year of analysis beginning with the ADT specified for the first year of the analysis period (the base year), and incremented according to specified traffic growth rates.

4.2 The construction submodel

The submodel is used to calculate the total cost of construction (if required) and to update the geometric and pavement design standards of affected road sections in each year.

4.3 The deterioration and maintenance submodel

This is the largest submodel which performs the calculation of annual road conditions in terms of road roughness, pavement cracking, rutting, ravelling, potholing and gravel loss on unpaved roads. Deterioration is modelled as a function of the initial pavement design standard and the effects of maintenance. The amount of maintenance carried out in a given year depends on the maintenance standards specified by the user. This can be done either as a condition responsive policy, e.g. apply an overlay if the specified critical condition is attained during the analysis period, or as a time dependent policy, e.g. apply an overlay every 8 years. Most types of maintenance treatments can be specified in HDM3 as part of a maintenance standard. This comprises a set of scheduled maintenance activities rationalised to represent practical maintenance policies. For example, a maintenance standard may include patching of potholes, surface dressing and overlaying, but these will be rationalised so that surface dressing and overlaying are not applied in the same year.

The performance of the constructed pavement is modelled using two factors which greatly influence the results of the economic appraisal;

(i) structural number, and
(ii) road roughness.

The pavement Structural Number (SN) is an index which represents pavement strength in such a way that all pavements with the same SN should have similar performance. The structural number of a pavement is defined by an empirical relationship in which the thicknesses and strengths of the pavement layers are combined in the equation below:

$$SN = a_i \cdot h_i \qquad \text{.....(2)}$$

where a_i = strength coefficient of pavement layer (i)
h_i = thickness in inches of pavement layer (i)

The values of the strength coefficients for the layers depend mainly on the type and strength of the materials. The above equation is usually modified to take into account the contribution to pavement strength made by subgrades of greater than 3% CBR. Structural number is used in HDM3 to model pavement performance. Strong pavements will have high structural numbers and therefore a low rate of deterioration.

Road roughness is the second crucial parameter used in modelling pavement performance. It is the single pavement condition which is used in the calculation of road user costs, particularly VOC. It represents the uneveness of a road surface and is taken to be the cause of wear and tear in vehicles. Roughness is commonly measured using one of three systems;

(i) the TRRL towed fifth wheel bump integrator (BI) in mm/km,
(ii) the Portland Cement Association (PCA) road meter, giving the Quarter-car Index (QI) which simulates the vertical motion of a standard car and is measured in counts per km,
(iii) vertical profile measurements using a rod and level. This is the internationally adopted standard referred to as the International Roughness Index (IRI) measured in meters/km.

The following relationships, developed at the World Bank are used to convert roughness values between the three units ;

$$BI = 55 \times QI \qquad \text{.....(3)}$$

and,

$$BI = 630 \times IRI^{1.12} \qquad \text{.....(4)}$$

A newly constructed road has to be assigned a structural number and an initial roughness. The very high dependence of road user costs on roughness, and of roughness progression on the pavement structural number, suggests that the results of an economic economic analysis will be affected by the values assigned to these two variables at the beginning of the analysis.

4.4 The VOC submodel

This estimates the costs of vehicle operation as a function of the average road condition (particularly rod roughness) in each year of the analysis period, the geometric characteristics of the road (i.e. vertical rise and fall, horizontal curvature, and road width), and the specified traffic and vehicle characteristics. The VOC submodel is also used to calculate passenger and cargo travel time costs.

4.5 The exogenous benefits and costs submodel

This submodel is used to assign costs and benefits in the current year which cannot be estimated within the model. Such costs and benefits may include toll charges, savings in accident and discomfort costs, and the salvage value of the road in the final year of the analysis period.

5 THE EVALUATION MODULE

The evaluation module performs economic analyses for each of the specified road links based on the costs of construction, maintenance, and vehicle operation calculated for each year in the analysis period. Benefits are calculated by comparing each link-alternative against the specified Do Nothing link-alternative. For each link-alternative, the model calculates the annual financial, economic and foreign exchange costs of construction, capital investment, maintenance, vehicle operation, time, and exogenous costs or benefits. The results of the analyses are given using the two economic indicators; Net Present Value, and Internal Rate of Return. Groups of link-alternatives can also be compared, for example, to determine the benefits of different maintenance standards over a network of roads.

6 THE REPORT GENERATION MODULE

Results of the modelling and evaluation performed by HDM3 are stored in direct access data files. In the input module, the user may specify the types of output required from the model. The report generation module produces the required output from the results held in the direct access files. These include tables giving a summary of the transport costs incurred each year, the benefits derived from specified projects together with their Net Present Values and Internal Rates of Return.

7 THE MICROCOMPUTER VERSION OF HDM3

The World Bank has recently completed the adaptation of the HDM3 model to microcomputers. A user-friendly front end program is included in the microcomputer package to assist users with data input. The front end program performs all formatting requirements for input files, and checks the validity of the data. Unlike the diagnostics module in HDM3

however, the front end program does not check the logic of the data sequence. For example, in the diagnostics module, it is not possible to run HDM3 with link-alternatives which are not defined in Series A. The front end program will not check for the existence of a link-section if it is specified as an alternative. Typical runtime duration of the HDM3 model on microcomputers range from 3 minutes to over 30 minutes depending on the hardware configuration of the micro (e.g. IBM AT with math coprocessor will take around 4 minutes, and an IBM XT will take more than 30 minutes). This when compared against the total turn-round time for mainframe computers is quite acceptable.

8 SUMMARY

The HDM3 model can be used used to perform a number of economic analyses. One important type on analysis is in the choice of surfacing type during the design of a pavement. A simple example is the choice between a surface dressing or an asphalt concrete surfacing for a pavement to be constructed in a developing country. In this case the surface dressing is likely to have a high initial roughness with higher roughness progression rates resulting in a high VOC component but with a lower cost of construction. The choice between the two surfacings will therefore be governed by the total VOC calculated for the two alternatives.

In the geometric design of new roads, the investment models can only be used to reject those designs which would be extravagant e.g. the provision of a dual carriageway when a single carriageway would suffice. It should be noted that the geometric design of roads should be done to satisfy capacity requirements and to avoid accidents. The investment models do not evaluate benefits from savings in accident rates, and since the models do not evaluate the effects of traffic congestion, they cannot be used to determine optimum geometric designs. For example the similar results would be obtained for a single carriageway of 12 m width and a dual carriageway of 6m width on either side if construction costs are equal.

Although the HDM3 model does not carry out mathematical optimisation of, for example, maintenance activities or the timing of construction or maintenance, it can be used to conduct a sensitivity analysis of the results to study the effects of changes in construction cost, maintenance cost, VOC, traffic growth, discount rates, and time values. The effects of time staging project alternatives can also be carried out by using the the expenditure budgeting module of HDM3.