REPUBLIC OF KENYA

MINISTRY OF PUBLIC WORKS

DETAILED DESIGN STUDY
ON
THE NAIROBI BYPASS PROJECT

MATERIALS REPORT
(MAIN)

MATERIALS INVESTIGATIONS
AND
PAVEMENT DESIGN

SEPTEMBER 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

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Japan International
Cooperation Agency

The Permanent Secretary
Ministry of Public Works
P.O.Box 30260
NAIROBI

The Chief Engineer (Roads)
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1. INTRODUCTION

1.1. THE MATERIALS SURVEY

(1) Aim

The materials survey was conducted to identify the subsurface geological condition of the road structures, the geological specification of the subgrade as well as the specification of the pavement materials, which will be used as basic data for road design.

Investigation covered mainly the following are as:-

a. Foundation of Bridges

b. Foundation of Embankments

c. Cutting and Embankment Works

d. Subgrade Condition

e. Stability Angle and Protection of Slopes

f. Present State of Existing Quarries

g. Location, Quality and Quantity of New Quarries

h. Ground Condition of Soak Pit

(2) Survey Item and Number

The survey item and number are shown in Tables 1.1.1 and 1.1.2.

(3) Survey Method

The number of investigations, and types of tests required, were determined by the MOPW's Road Design Manual Part III.

(4) Location of The Survey Points

The location of the survey points on the proposed alignment are shown in Fig. 1.1.1, Table 1.1.3 to 1.1.7 and APPENDIX 1, while the locations of the investigated quarries are shown in Fig. 1.1.2, Table 1.1.7 and APPENDIX 6 and 7.

(5) Work Periods

The first field works laboratory test	:	20th JUNE 1990 to 30th OCTOBER 1990
The additional field works laboratory test	:	20th SEPTEMBER 1991 to 30th OCTOBER 1991

1.2. PAVEMENT

(1) Outline of Pavement Design

The pavement design was based on the ROAD DESIGN MANUAL Part III in consideration of Traffic, Subgrade condition, availability of pavement material, availability of paving work and periodical maintenance referring to ROAD NOTE NO 29, AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES 1986 and ASPHALT PAVEMENT DESIGN MANUAL by the JAPAN ROAD ASSOCIATION.

A JICA design study team and the relevant engineers from the Material branch of the MOPW consulted over the pavement design, then design work commenced.

(2) Design Method

At first, the pavement of the main road was designed using the MOPW method, and then it was revised according to the ROAD NOTE NO 29, the AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES 1986 and the ASPHALT PAVEMENT DESIGN MANUAL by JAPAN ROAD ASSOCIATION.

After analysis, the pavement structure designed using ROAD NOTE No. 29 was adopted, while service roads were designed using the MOPW method. Approach roads, except the underpass at the UHURU monument junction, were designed to be the same as the existing pavement structure.

(3) Pavement Areas

Pavement areas are as follows:

1) Main road

2) Slip roads (Ramps)

3) Approach roads

4) Service roads

Table 1.1.1 (1) The First Survey Item and Number

Item No.		Description	Quantity
1		**Earthworks and Subgrade Study**	
1-a		Rotary Drilling (core drills, 7~13 m depth, total 77 m depth)	8 points
1-b		Standard Penetration test (1 time/1 m)	22 times
1-c		Test Pit (1 ~ 4 m depth)	30 points
1-d		Basic Test	
	1)	Grading to 0.075 mm sieve	36 samples
	2)	Atterberg Limits	36 samples
	3)	Compaction test (2.5 kg rammer)	36 samples
	4)	CBR and swell on samples moulded at 100% M.D.D. and O.M.C.	36 samples
	5)	Traiaxial Compression test (Unconsolidated Untrained)	3 samples
	6)	Consolidation test	3 samples
1-e		Subgrade Soils Test	
	1)	Compaction test (4.5 kg rammer)	6 samples
	2)	3 point CBR test	6 samples
2		**Borrow Pits Study (New Gravel sites)**	
2-a		Test Pit (0.6 ~ 2.3 m depth)	96 points
2-b		Soil Test	
	1)	Grading to 0.075 mm sieve	34 samples
	2)	Atterberg Limits	34 samples
	3)	Compaction test (4.5 kg rammer)	34 samples
	4)	CBR test	34 samples
	5)	CBR (cement stabilization)	14 samples
	6)	CBR (lime stabilization)	14 samples
3		**Foundation Study for Bridges and Box-culvert**	
3-a		Rotary Drilling (core drills, 5 ~ 16 m depth, total 31 m)	4 points
3-b		Standard Penetration test (1 time/1 m)	48 times
4		**Foundation Study for Embankment**	
4-a		Test Pits (1 m depth)	10 points
4-b		Soil Test	
	1)	Grain-Size Analysis	9 samples
	2)	Grading to 0.075 mm sieve	6 samples
	3)	Atterberg Limits	21 samples
	4)	Specific Gravity	9 samples
	5)	Natural Water Content	9 samples
	6)	Unit Weight	5 samples
	7)	Consolidation test	2 samples
	8)	Triaxial Compression test (Unconsolidated Undrained)	2 samples
	9)	Compaction test (2.5 kg rammer)	10 samples
	10)	Compaction test (4.5 kg rammer)	1 samples
	11)	CBR and swell on samples moulded at 100% M.D.D and O.M.C.	10 samples
	12)	3 point CBR test	1 samples
	13)	Free Swelling test	9 samples
	14)	Swelling Pressure test	5 samples
	15)	Linear Shrinkage test	5 samples

Table 1.1.1 (2) The First Survey Item and Number

Item No.		Description	Quantity
5		**Hard Stone Quarry Study**	
	5-a	Rotary Drilling (core drills, 10~15 m depth, total 65 m depth)	5 points
	5-b	Crushed Stone Test	6 samples
	1)	Grading to 0.075 mm sieve	6 samples
	2)	Los Angeles Abrasion	6 samples
	3)	Aggregate Crushing Value	6 samples
	4)	Sodium Sulphate Soundness	6 samples
	5)	Plasticity Index on L.A.A. fines	6 samples
	6)	Specific Gravity	6 samples
	7)	Bitument Affinity	6 samples
	8)	FI	6 samples
6		**Ground condition Study for Soak Pit**	
	6-a	Rotary Drilling (core drills, 10~15 m depth, total 25 m depth)	2 points
	6-b	In-situ Permeability test	1 time

Table 1.1.2 The Additional Survey Item and Number

Item No.		Description	Unit	Quantity
1		**Earthworks and Subgrade Study**		
	1-a	Rotary Drilling (core drills, 14 m depth)	point	1
	1-b	Standard Penetration test (1 time/ 1 m)	times	7
	1-c	Test Pit (1 ~ 4 m depth)		
		1 m depth	points	3
		2 m depth	points	1
		3 m depth	points	2
		4 m depth	points	1
	1-d	Field Soil Density Test (Sand Replacement Method)	points	5
	1-e	Basic Test		
	1)	Grading to 0.075 mm sieve	samples	11
	2)	Atterberg Limits	samples	11
	3)	Linear Shrinkage test	samples	11
	4)	Compaction test (2.5 kg rammer)	samples	11
	5)	CBR and swell on samples moulded at 100% M.D.. and O.M.C	samples	11
	1-f	Soil Test for Black cotton soil		
	1)	Natural Water Content	samples	3
	2)	Free Swelling test	samples	3
	3)	Swelling Pressure test	samples	3

Table 1.1.3 The List of Mechanical Boring (First Survey)

Bore hole name	Location	Depth of boring (m)	S.P.T (No.)	Note
BE-1	KM. 9 +125 m	0	0	cutting
BE-2	KM. 10 +840 m	8	8	"
BE-3	KM. 14 +380 m	8	6	"
BE-4	KM. 18 +820 m	10	5	"
BE-5	KM. 19 +835 m	9	8	"
BE-6	KM. 21 +485 m	11	4	"
BE-7	KM. 21 +952 m	11	6	"
BE-8	KM. 24 +713 m	6	5	"
BE-9	KM. 26 +150 m	13	7	"
BF-1	KM. 0 +373 m	5	0	Bridge foundation
BF-2	KM. 0 +432 m	5	0	"
BF-3	KM. 15 +550 m	5	5	Box-culvert foundation
BF-4	KM. 27 +915 m	16	16	Bridge foundation
BK-1	KITENGELA ROCK QUARRY	15	0	Crushed stone
BK-2	"	15	0	"
BK-3	"	10	0	"
BM-1	MUTHIGA ROCK QUARRY	15	0	"
BM-2	"	15	0	"
BM-3	"	10	0	"
Total		187	70	

Table 1.1.4 The List of Mechanical Boring (Additional Survey)

Bore hole name	Location	Depth of boring (m)	S.P.T (No.)	Note
BE-10	KM.10 +000 m	14	7	cutting

Table 1.1.5 The List of Test Pit for Embankment Foundation (First Survey)

Test pit name	Location	Depth (m)	Disturbed Sampling (No.)	Block Sampling (No.)
Tb-1	KM.0 +900 m	1.1	1	1
Tb-2	KM.1 +435 m	1.9	1	0
Tb-3	KM.1 +900 m	1.5	1	1
Tb-4	KM.2 +400 m	1.0	1	0
Tb-5	KM.2 +900 m	1.0	1	1
Tb-6	KM.3 +400 m	0.65	1	0
Tb-7	KM.3 +900 m	0.6	1	1
Tb-8	KM.4 +400 m	1.0	1	0
Tb-9	KM.4 +895 m	0.8	1	1
Tb-10	KM.5 +400m	1.5	1	0
Total		11.05	10	5

Note: Disturbed Sampling from Black Cotton Soil Block Sampling from Under Layer of Black Cotton Soil

Table 1.1.6 The List of Test Pit for Subgrade (First Survey)

Test pit name	Location	Depth of pit (m)	Disturbed Sampling (No.)	Note
Tc-1	KM.5 +920 m	0.8	1	
Tc-2	KM.7 +400 m	0.85	"	
Tc-3	KM.7 +800 m	0.7	"	
Tc-4	KM.8 +600 m	1.2	"	
Tc-5	KM.9 +110 m	1.1	"	
Tc-6	OLD KM.9 +900 m	1.7	"	
Tc-7	OLD KM.10 +840 m	2.0	"	
Tc-8	OLD KM.12 +100 m	0.7	"	
Tc-9	OLD KM.12 +500 m	1.4	"	
Tc-10	OLD KM.13 +180 m	0.5	"	
Tc-11	OLD KM.14 +380 m	2.0	"	
Tc-12	OLD KM.15 +185 m	3.7	"	
Tc-13	KM.16 + 90 m	4.0	"	
Tc-14	KM.16 +700 m	0.7	"	
Tc-15	KM.17 +150 m	4.0	"	
Tc-16	KM.18 +820 m	2.0	"	
Tc-17	KM.19 +305 m	3.7	"	
Tc-18	KM.21 +480 m	2.25	"	
Tc-19	KM.21 +955 m	2.1	"	
Tc-20	KM.24 +178 m	2.2	"	
Tc-21	KM.24 +705 m	1.9	"	
Tc-22	KM.25 +288 m	3.7	"	
Tc-23	KM.25 +775 m	2.8	"	
Tc-24	KM.26 +158 m	2.1	"	
Tc-25	KM.26 +778 m	1.2	"	
Tc-26	KM.27 +178 m	1.4	"	
Tc-27	KM.27 +678 m	1.1	"	
Tc-28	KM.28 +103 m	1.1	"	
Tc-29	Aline KM.0 +420 m	4.0	"	
Tc-30	Aline KM.0 +870 m	1.0	"	
Total		71.5		

Table 1.1.7 The List of Test Pit for Subgrade (Additional Survey)

Test pit name	Location	Depth (m)	Desturbed Sampling (No.)	Block Sampling (No.)	Field Soil Density (No.)
Tc-31	KM.10 +400 m	2.5	2	0	1
Tc-32	KM.11 +380 m	3.0	2	0	1
Tc-33	KM.11 +880 m	0.8	2	1	1
Tc-34	KM.13 +660 m	1.5	1	1	0
Tc-35	KM.14 +155 m	1.0	1	0	1
Tc-36	KM.14 +650 m	4.0	2	0	1
Tc-37	KM.12 +575 m	0.9	1	1	0
Total		13.7	11	3	5

Table 1.1.8 The List of Test Pit for Borrow pit (GRAVEL MATERIAL SITE)

Test pit name	Depth (m)	Disturbed Sampling (No.)
KIRIBA		6
TP1	1.2	
TP2	1.5	
TP3	1.6	
TP4	1.0	
TP5	0.9	
TP6	1.5	
TP7	1.7	
TP8	0.9	
TP9	1.2	
TP10	0.6	
TP11	1.7	
TP12	1.8	
TP13	1.4	
TP15	1.7	
THANDI		6
TP1	1.9	
TP2	1.1	
TP3	1.4	
TP4	1.6	
TP5	1.0	
TP6	0.9	
TP7	0.4	
TP8	1.3	
TP9	1.3	
TP10	0.8	
TP11	1.8	
TP12	1.6	
TP13	1.4	
DAGORETTI		0
TP1	0.6	
TP2	0.9	
TP3	1.0	
TP4	1.5	
TP5	1.8	
TP6	1.2	
TP7	1.1	
TP8	0.7	
TP9	1.2	
TP10	1.4	
FOREST GUARD		0
TP1	0.7	
TP2	1.1	
TP3	1.3	
TP4	1.5	
TP5	1.8	
TP6	1.1	
TP7	1.0	
TP8	1.2	
TP9	1.0	
TP10	1.0	
CHURCH		4
TP1	1.1	
TP2	1.3	
TP3	1.1	
TP4	1.4	
TP5	1.4	
TP6	1.2	
TP7	1.3	
TP8	1.6	
TP9	1.9	
TP10	2.2	
TP11	2.3	
TP12	1.5	
TP13	1.9	
TP14	1.6	
TP15	1.5	
TP16	1.8	
CALSHEET		6
TP1	2.0	
TP2	2.5	
TP3	2.0	
TP4	2.5	
TP5	2.5	
TP6	2.0	
TP7	0.3	
TP8	0.2	
TP9	0.3	
TP10	0.4	
KAREN		4
TP1	1.3	
TP2	1.5	
TP3	1.4	
TP4	1.4	
TP5	1.7	
TP6	1.7	
TP7	1.3	
TP8	1.0	
TP9	1.0	
TP10	1.4	
TP11	1.0	
TP12	1.1	
TP13	1.2	
TP14	1.0	
Total	116.1	26

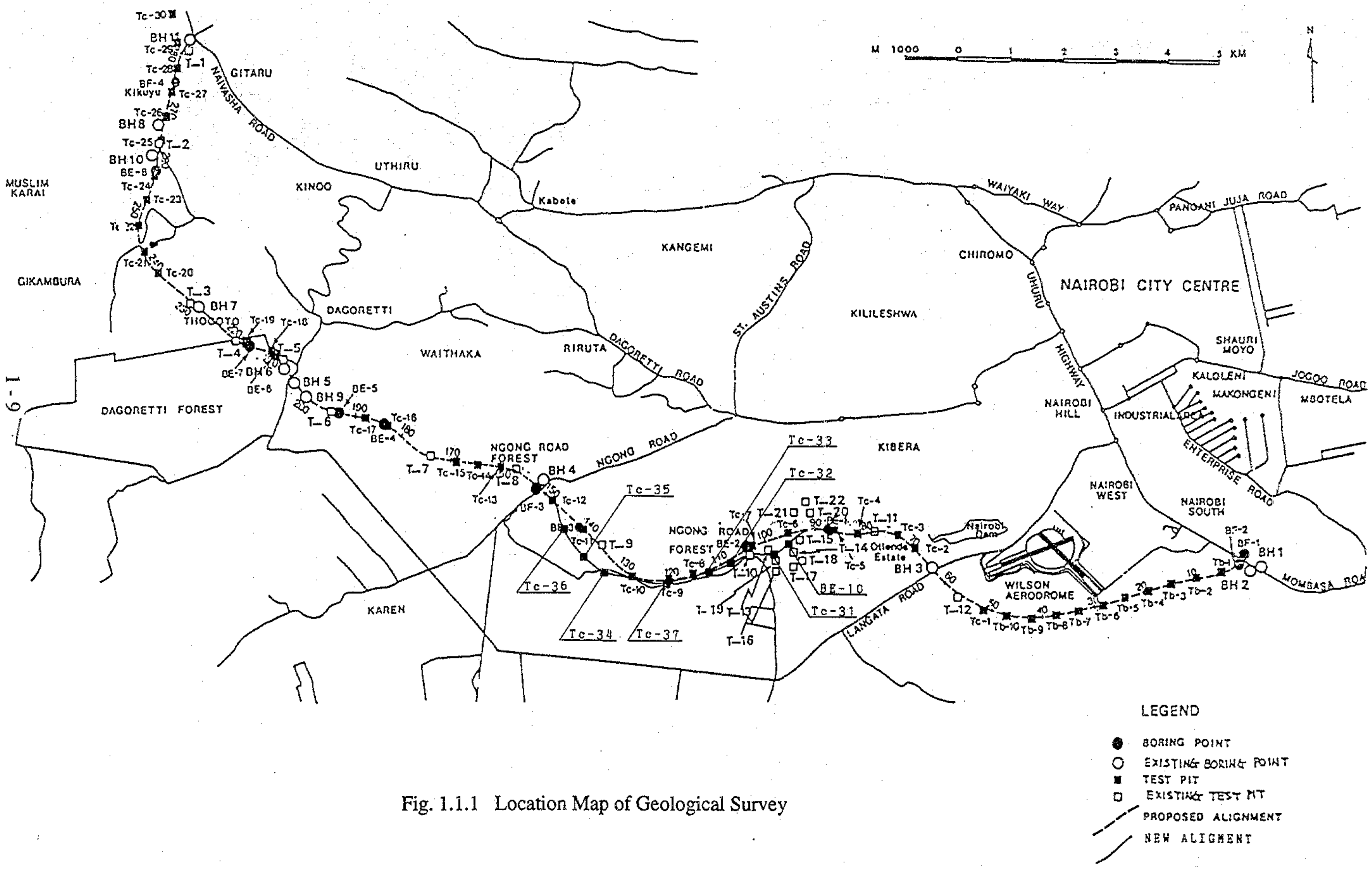


Fig. 1.1.1 Location Map of Geological Survey

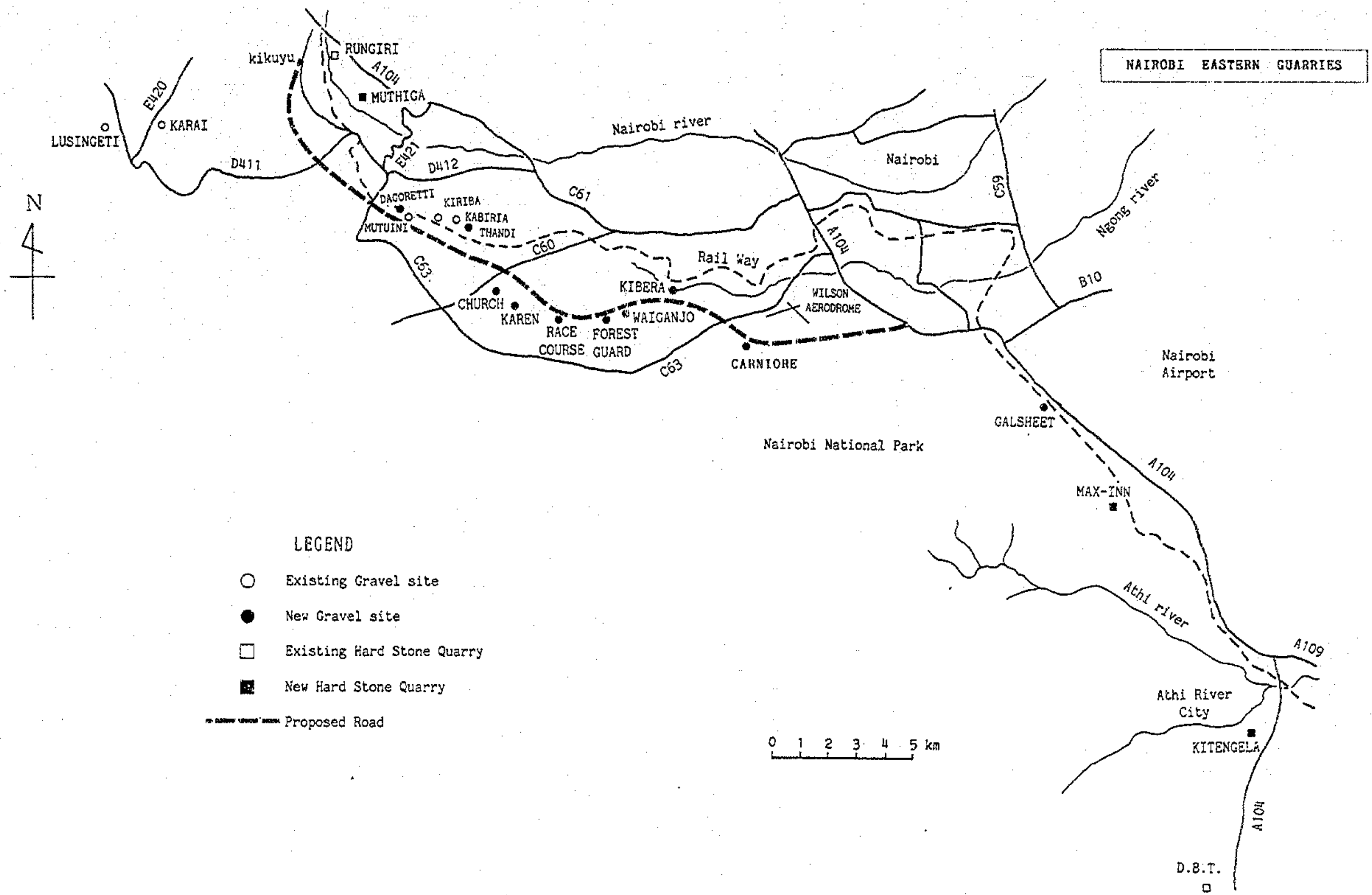


Fig. 1.1.2 Location of the Investigated Quarries

2. OUTLINE OF TOPOGRAPHY AND GEOLOGY

2.1 TOPOGRAPHY

Nairobi and its environs are located on the east side highland of the Rift Valley at an altitude which ranges between 1,600 and 2,000 m.

The latitude is about 36° East, and the longitude is about 1° South. The city is approximately 140 km South of the Equator, with the port of Mombassa, on the Indian Ocean about 490 km distant. Lake Victoria is about 340 km away from Nairobi.

The western and northern parts of Nairobi are hilly land such as Nairobi hill which is below 2,000 m. The southern and eastern parts of Nairobi are spreaded on the Athi and Kapiti Plains and are below 1,800 m. Traveling from West to East the topography inclines gently.

Highland regions to the West and North of Nairobi are mainly used as farm-land with the exception of town areas and forestation. The surface of the Athi plain is covered with Black cotton soil, which is an expansive clay. This area belongs to the Athi River Drainage System and is crossed by many rivers which flow East from the eastern highlands of the Rift Valley forming alluvial deposits in some places.

2.2 GEOLOGY

Bed rock in East Africa, including Kenya, is formed by crystalline Precambrian rocks belonging to the Mozambique Belt. The geological structure is typified by the Great Rift Valley which runs from North to South.

Bed rock along the Rift Valley has been cut by many faults, and the Rift Valley environs were covered with thick volcanic ash following the tertiary period.

Phonolite, Trachyte, Tuff, etc. spouted sometime between the tertiary and Pleistocene periods and was distributed on the East highlands of the Rift Valley including the Nairobi area.

The oldest lava flow forms the eastern plains where the Industrial Area and the Airport are located, it is called the Nairobi Phonolite. The next flow was the Nairobi Trachyte which terminates just northwest of the City Centre at Nairobi Hill and swings westward to form the heights on which Karen and Langata are located. During a pause in the volcanic activity, deep valleys were eroded into

the Trachyte block and then filled with material from the subsequent eruptions. These are known as the Kirichwa Valley Tuffs. They have been used extensively for building purposes, under the name Nairobi Stone, which accounts for the drab gray color of so many buildings in the area.

The youngest lava flow is called Limuru Trachytes which is located in western Nairobi.

Surface soil in the northwest highland region of Nairobi is composed of soil from volcanic ash, weathered volcanic rocks, etc. While black cotton soil, which is cohesive and expansive is distributed in the Athi Plain. The study area is crossed by many rivers belonging to the Athi River Drainage System and there is some intermittent distribution of the alluvial deposits.

A Geological Map of Nairobi is shown in Fig. 2.2.1.

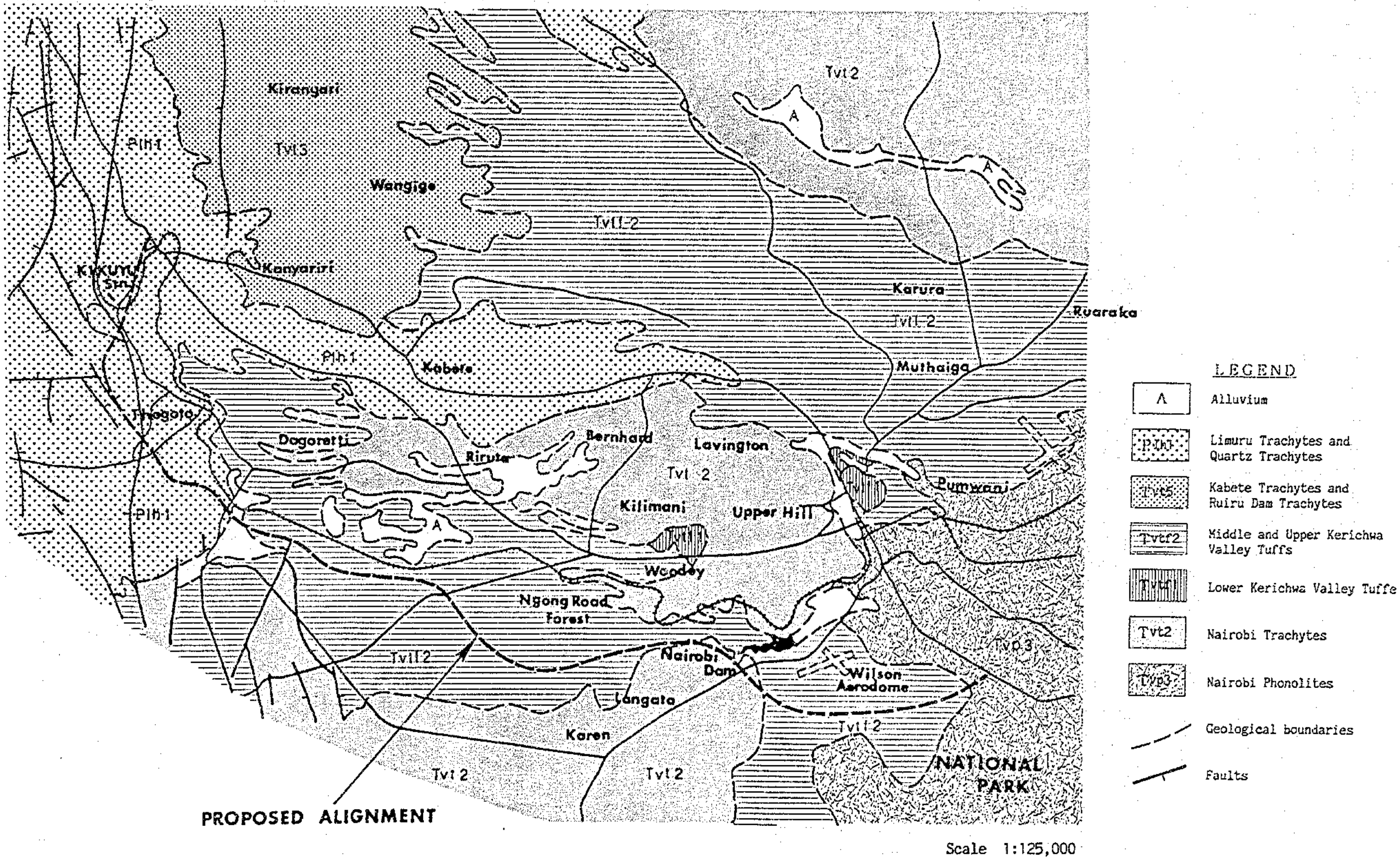


Fig. 2.2.1 Geological Map of Nairobi

3. INVESTIGATIONS ALONG THE ALIGNMENT

3.1 SUBSURFACE GROUND CONDITIONS

The subsurface geological conditions are as shown in APPENDIX 2. They can roughly be divided into the following areas:

– Black cotton soil areas

– Red Soil areas

– Pyroclastic tuff areas

– Trachyte areas

– Other areas

(1) Black cotton soil areas

These areas are found in flat topography mainly in Nairobi National Park (KM 0 + 00 m - 5 + 700 m) but they can also be found, sporadically, in the Ngong Road Forest (KM 11 + 00 m - 11 +220 m, KM 11 + 494 m - 11 + 614 m, KM 11 + 781 m - 12 + 773 m, KM 13 + 26 m - 13 + 748 m, KM 13 + 936 m - 13 + 996 m, KM 14 + 916 m - KM 14 + 962 m KM 16+ 350 m - 16 + 990 m).

Black cotton soil is an expansive clay which is black to gray in color and includes weathered gravel (tuff, phonolite and trachyte) in places.

Its thickness ranges from 0.3 m to 1.0 m but is about 0.5 m in most areas. The bed rock lies at a shallow depth in Nairobi National Park and it consists of phonolitic agglomerate (hard rock). The bed rock's surface layers (0.5 m to 1.0 m) have been affected by high weathering and consequently they are similar to soft rock or soil. In contrast, the bed rock in the Ngong Road Forest consists of pyroclastic tuff which is a soft to medium-hard rock.

(2) Red soil areas

These areas are found in the hills. Most alignment sections, with exception of the flat area in the National park, can be geologicaly classified into this soil type. Red soil is reddish brown, as its name indicates, and it's a stiff clay of uniform grading. In dry conditions it is

vulnerable to erosion. Its thickness varies from 3 m to 6 m in most places. Areas with a greater thickness of 5 m to 10 m extend km 26 +600 m where road C63 joins its terminal point.

The bed rock consists of pyroclastic tuff (soft to medium hard rock) in areas from KM 9 + 600 m to 20 + 200 m. Generally, a highly weathered zone with soil exists from the surface layers to about 1.5 m down and below this a weathered zone is found at a depth of about 2.0 m. Bed rock found from Dagoretti Forest to Kikuyu consists of trachyte (hard rock). Its weathering condition can roughly be divided in two. The area from Dagoretti Forest to the vicinity of the Alliance Girls School is fresh. It is composed of a highly weathered zone, from the surface layer of bed rock to a depth of about 1.0 m, with the area underneath it being nearly fresh. On the other side of the Alliance Girls School to the terminal point, the weathering zone is thick and the surface layer goes to a depth of between 3.0 m and 11.0 m. It is a highly weathered zone.

(3) Pyroclastic tuff areas

These areas lie between the Moi Otiende Estate in Langata (KM 7 + 900 m), Langata Prison (KM 9 + 300 m), and the Ngong Road Forest (KM 11 + 200 m - 12 + 50 m and 12 + 300 m - 12 + 900 m). The pyroclastic tuff can be classified as soft to medium hard rock. The surface layer, to a depth of a about 1.5 m, consists of a highly weathered zone and has a weathers zone for about 2 m underneath it. In this area, the tuff is covered with less than 1.1 m of red soil (a partly lateritic gravel).

(4) Trachyty area

These areas is distributed from Carnivore Restaurant (KM 5 + 700 m) to Moi Otiende Estate (KM 7 + 800 m) as well as in Dagoretti Forest (KM 21 +00 m to 21 + 500 m). Trachyte can be classified as a hard rock which is generally fresh, apart from about 1 m of highly weathered rock. In this area a layer, about 0.5 m thick, of lateritic gravel covers the trachyte.

(5) Other areas

A garbage dumping site is located near KM 9 + 00m in the vicinity of Langata Prison. Alluvium soil is distributed near KM 15 + 560 m which

crosses C63 KM 20 + 200 m to 20 + 900 m in Dagoretti and near KM 26 + 350 m in the vicinity of Ondiri Swamp. Alluvium soil is generally stiff, but a very soft peat is found near Ondiri Swamp. The banking for C63 Road is distributed in the vicinity of KM 27 + 600 m to 27 + 850 m and 28 + 50 m to 28 + 300 m in Kikuyu.
Banking material consists mainly of red soil.

3.2. FOUNDATION OF BRIDGES

(1) Mombasa road Junction Bridge (KM 0+369 m - 0+426 m)

Boring BF-1 and -2 were made at the Mombasa road junction. The geological sections are shown in APPENDIX 3 . Black cotton soil is distributed on the surface while bed rock starts at a depth of 0.5 m to 1.0 m or deeper. Bed rock consists of phonolitic agglomerates. BF-1 showed that a highly weathered zone exists from the surface of bed rock down to a depth of 1.1 m, with a weathered zone lying underneath this. Since the weathered zone is hard, with a boring core of average length 15 cm, it is believed that this zone can be used as the bearing layer for the proposed bridge.

(2) Uhuru Monument Junction Bridge (KM 6+676 m - 6+714 m)

The geological sections and geological conditions are as shown in the F/S Report.

(3) Railway Bridge (KM 27+00 m)

The geological sections and geological conditions are as shown in the F/S Report.

(4) Kikuyu Town Overbridge (KM 27+920 m)

Boring BF-4 was conducted as a study for the proposed Kikuyu Town overbridge. The geological sections obtained are as shown in APPENDIX 3. Red soil is distributed from the surface of Road C63 to a depth of 5.00 m and a highly weathered Trachyte lies below it. The red soil shows an N-values of 4 to 8 with soft to medium consitency. Highly weathered trachyte has turned to soil. In particular, the upper 4 m zone has a low N-value between 4 and 24. Below this depth N-values are N > 30 and the consistency becomes hard offering a good bearing layer.

3.3. FOUNDATION OF EMBANKMENTS

3.3.1 BLACK COTTON SOIL

The distribution of black cotton soil is shown in Chapter 3. This soil is mainly found in the National Park area and in the Ngong Road Forest area.

Results of soil tests, on samples collected from each area, are shown in Table 3.3.2, Fig. 3.3.1, Fig. 3.3.2 and Appendix 9.

The test results are given below.

		National Park	Ngong Road Forest
Liquid Limit(%) LL	=	39 ~ 83	37 ~ 59
Plasticity Index PI	=	14 ~ 43	16 ~ 30
Moisture Content(%) W	=	16.3 ~ 40.5	20 ~ 26
Shrinkage Limit (%) SL	=	8 ~ 20	9 ~ 14
Grading Passing 75μm (%)	=	89 ~ 95	76 ~ 90
Free Swell (%)	=	80 ~ 165	68 ~ 98
Swell at 100% MDD 4days Soak (%)	=	2.7 ~ 5.5	3.6 ~ 5.2
Swelling Pressure (kN/m^2)	=	0 ~ 80	32.2 ~ 101.9
Is = LL - SL (%)	=	31 ~ 64	29 ~ 45

(1) Characteristics of soil in the National Park

The classification of black cotton soil based on these results is shown in Table 3.3.1, which is based on the report by F.J. Gichaga, B.K. Sahu and T.G. Visweswaraiya (Prediction of swell of black cotton soil in Nairobi, 19 June 1987). The table shows that black cotton soil along the alignment can be classified as having "High to Very High Swellability". However, swelling pressure is generally small and σs = O - 80KN/m^2 has been measured. The above report proposes the following equation on swelling pressure:

$$\sigma_s = 4.4 \times 10^{-7} \times \gamma d2.75 \times (LL - W)^{3.27}$$

where:

σ_s : Swelling Pressure (KN/m^2)

γd : Dry Density (KN/m^3)

LL : Light Limit

W : Moisture Content (%)

The relationsip between the value calcualated by this equation and the measured value are as shown in Fig. 3.3.2. There is good agreement between them. Points not in agreement cannot be considered as $\sigma s = 0$ and only swelling pressures approximately the same as the calculated value should be taken into consideration.

As of July 1990, conspicuous shrinkage cracks occurred in the field. Based on this, it can be concluded that the ground consists of expansive clay and that the swelling pressure is about equal to that calculated by the above equation.

As a measure against the effects of this swelling pressure, the soil should be replaced with suitable fill material since the layer is as thin as 0.5 m. The sections subject for such replacement are as follows:

- KM. 0 to KM. 5 + 300 m

Where the thickness of filling is greater than 5 m (in case of $\gamma t = 1.75$ t/m^3), no replacement is required since the fill-up load can effectively prevent expansion. As of July 1990, the unconsolidatedundrained shear strength is as shown below and consolidation is very small.

- Angle of shear resistance : $\phi = 5.3 \sim 32.8$ degrees

- Cohesion : $C = 37 \sim 51$ KN/m^2

(2) Characteristics of soil in the Ngong Road Forest

Characteristics of the black cotton soil in this area indicate the influence of lateritic gravel which is not found in the National Park area.

Therfore, this soil has lower values of liquid limit and moisture content and a higher value of swelling presure despite the swell potential being slightly less. The swellability of soil in the area can be classified as medium to high swellability, judging from Table 3.3.1.

To deal with the problem of black cotton soil, the adoption of the replacement method, suggested for the National Park area, will be recommended. The sections subject for such replacement are as follows:

- KM.11 + 100 m to KM.11 + 200 m

- KM.12 + 50 m to KM.12 + 300 m

- KM.13 to KM.13 + 350 m

3.3.2 SOFT GROUND

As described in Section 3.1, alluvium soil is found mainly at the Ngong Road Junction, the Dagoretti Forest Junction and at the high bank of Alliance Boys High School (For geological sections, refer to Appendix 3). These Fig.s show that alluvium soil is generally stiff to hard. Peat forms soft ground in the area of the proposed high bank. However, the thickness of this peat layer is negliable, only 1.5 m, and the layer is not too wide spread. Consequently, it is believed that the soil condition will present little or no problem, in stability or settlement, for the foundation of the high bank.

3.3.3 OTHERS

Other foundation soils are mostly red soil. These is suitable as a foundations for the embankment because reds soil is stiff to hard.

Table 3.3.1 Classification of Black Cotton Soil

Soil Parameter	Classification		
	Moderate Swellability	High Swellability	Very High Swellability
1. Dry Density γd (KN/m^3)	< 15	$15 \leq \gamma d \leq 15.75$	> 15.75
2. Clay Content < 0.002mm (%)	< 40	40≤0.002≤55	> 55
3. Liquid Limit W_{LL} (%)	< 48	48≤LL≤65	>65
4. Plasticity Index PI	< 30	30≤P.I≤40	>40
5. Shrinkage Index $I_s = W_{LL} - W_{SL}$	0~20 (Small) 20~30 (Moderate)	30~60	> 60
6. Swell Pressure σs (KN/m^2)	<120	$120 \leq \sigma s \leq 600$	>600
7. Swell Potential $\frac{\Delta h}{ho} = h'$ (%)	< 4.5	h' > 4.5 < 13	h' > 20 < 13

Source: F.J.Gichaga, B.K.Sahu and T.G. Visweswaraya
University of Nairobi 19 June 1987.
"Prediction of Swell of black cotton soil in Nairobi"

Table 3.3.2 Soil Test Results Summary Sheet of Black Cotton Soil

	Pit No.	Station	Liquid Limit L.L (%)	Plasticity Index P I	Moisture Content W (%)	Specific Gravity	Bulk Density (kg/m³)	Shrinkage Limit S L (%)	Free Swell (%)	Swell at 100% MDD, 4days Soak (%)	Swelling Pressure (KN/m²)	I s= LL−SL (%)	* Swellability
NATIONAL PARK	Tb−1	0+900m	78	33	34.2	2.40	1,545	16	130	4.5	80	62	High~Very high
	Tb−2	1+435m	73	30	—	—	—	—	—	—	—	—	—
	Tb−3	1+900m	80	42	40.5	2.50	1,507	20	160	5.0	44	60	High~Very high
	Tb−4	2+400m	73	40	—	—	—	—	—	—	—	—	—
	Tb−5	2+900m	78	30	40.3	2.44	1,568	14	165	4.4	0	64	High~Very high
	Tb−6	3+400m	79	42	—	—	—	—	—	—	—	—	—
	Tb−7	3+900m	74	43	30.5	2.40	1,619	20	140	5.5	0	54	High~Very high
	Tb−8	4+400m	73	32	—	—	—	—	—	—	—	—	—
	Tb−9	4+895m	39	14	16.3	2.48	1,658	8	80	—	0	31	Moderate~High
	Tb−10	5+400m	83	30	—	—	—	—	—	5.2	—	—	—
	Tc−8	12+100m	80	38	—	—	—	—	—	4.6	—	—	—
	Tc−10	13+80m	64	26	—	—	—	—	—	4.4	—	—	—
NGONG	Tc−33	11+880m	37	16	21.0	—	—	8	98	3.6	101.9	29	Moderate
	Tc−37	12+575m	40	18	20.0	—	—	9	68	4.2	32.2	31	Moderate
	Tc−34	13+660m	59	30	26.0	—	—	14	78	5.2	72.4	45	High
	Tc−14	16+700m	50	21	—	—	—	—	—	0.5	—	—	Low
	Tc−17	19+305m	39	14	—	—	—	—	—	0.7	—	—	Low

* See Table 3.3.1

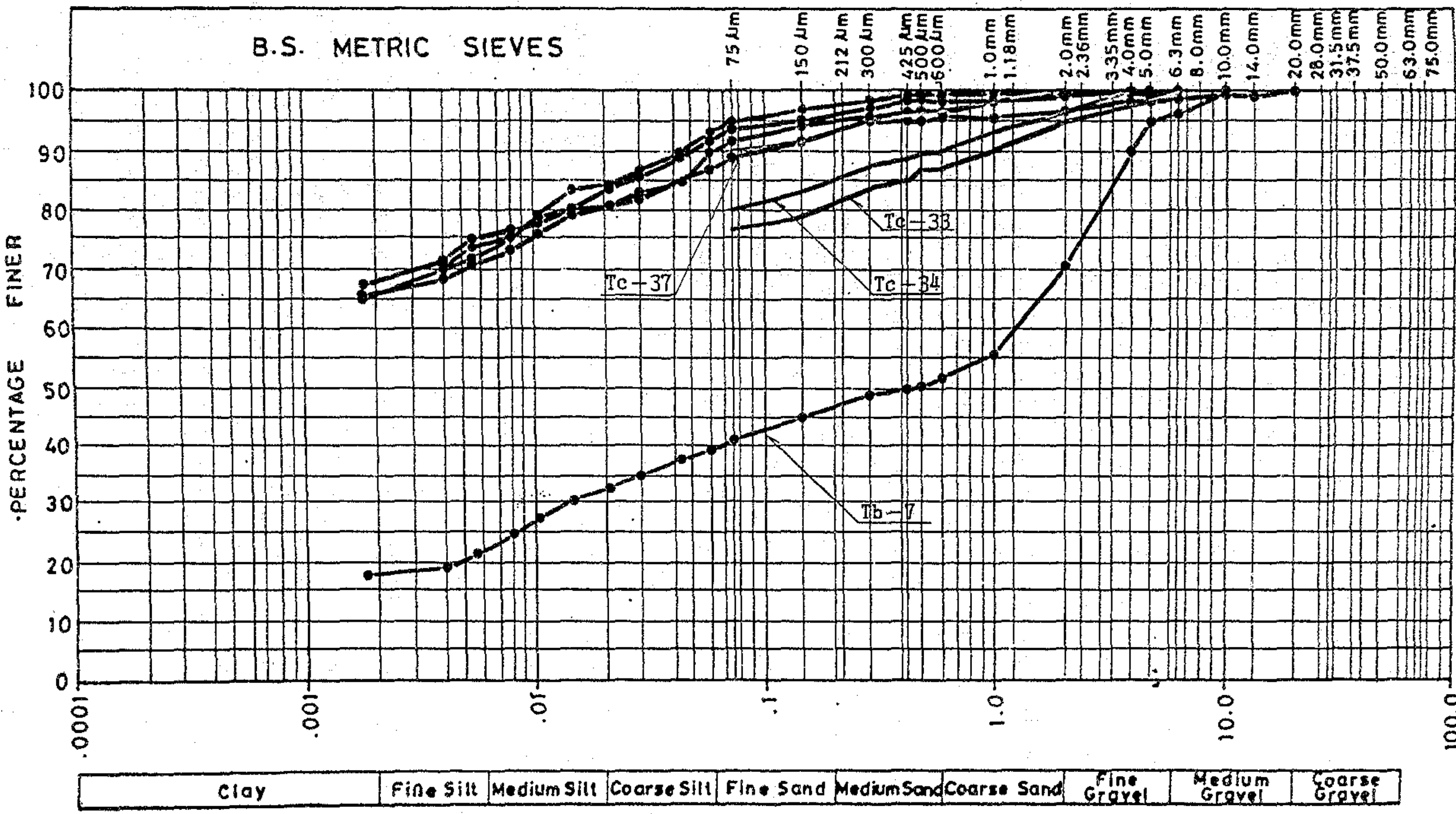


Fig. 3.3.1 Particle Size Distribution of Black Cotton Soil

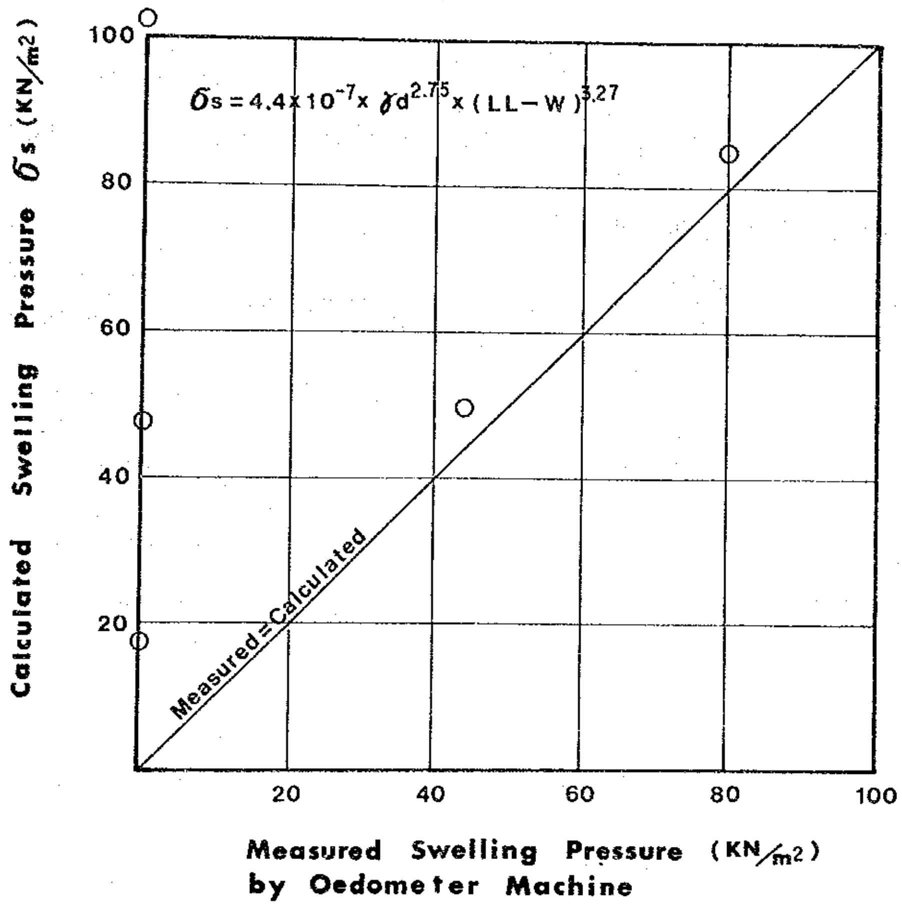


Fig. 3.3.2 Measured-Calculated Swelling Pressure in National Park area

3.4 EARTHWORKS

3.4.1. CUTTING

The layers subject to cutting are mostly red soil, pyroclastic tuff and trachyte. For geological sections at major cutting areas, refer to APPENDIX 3. The table below shows the observations made at boring cores and the relationship between the type of work and geology.

Table 3.4.1 Cutting work

Classification	Cutting work	Geology
Soil	Bulldozer work	Red soil High weathered tuff High weathered trachyte
Weathered rock	Ripper work	Weathered tuff
Hard rock	Blast work	Weathered and sound Trachyte tuff

3.4.2 EMBANKMENTS

(1) Fill materials

Embankments will all be built using a fill material which will be obtained from the alignment cutting operation. Consequently, fill material consists of red soil, pyroclasitc tuff, trachyte and some lateritic gravel. Most of the fill material however, will be read soil. Results of the soil test are shown in APPENDIX 8 and Fig.s 3.4.1 to Fig. 3.4.5.

The test results are given below.

		Red soil	High weathered pyroclastic tuff and trachyte	Lateritic gravel
Group Index	=	11.9 ~ 39.3	-4.4 ~ 17.6	-3.2 ~ 15.0
Liquid Limit (%)	LL =	48 ~ 67	NP ~ 55	46 ~ 55
Plasticity Index	PI =	13 ~ 30	NP ~ 21	15 ~ 24
Shrinkage Limit (%)	SL =	9 ~ 11	2 ~ 10	—
Grading Passing				
75μm (%)	=	68 ~ 99	6 ~ 76	20 ~ 82
Plasticity Modulus	=	1,224 ~ 2,970	0 ~ 1,743	360 ~ 1,408
Moisture Content (%)	=	23.6 ~ 27.4	10.2 ~ 15.0	—
M.D.D (T99) (kg/m^2)	=	1,300 ~ 1,510	1,240 ~ 1,620	1,476 ~ 1,780
O.M.C (T99) (%)	=	26 ~ 38	23 ~ 34	20 ~ 39
Swell at 100% MDD				
4days Soak(%)	=	0.1 ~ 1.7	0.2 ~ 1.4	0.1 ~ 0.9

The MOPW's Road Design Manual Part III gives the following judgement criteria for suitable fill materials.

Swell	Max 3%
Plasticity Index	Max 50%
Moisture Content ≧	O.M.C. (T99) × 1.05

In addition, the AASHTO indicates that a material with a Group Index ≥ 20 is unsuitable for use as a fill material.

As red soil, Pyroclastic tuff, trachyte, and lateritic gravel not only satisfy the above criteria but, in addition, do not contain organic matter, they are judged to be good fill marterials. The fact that a fill up to 15 m using red soil has been successfully used for existing roads supports this judgement.

OMC will be required for the compaction of the fill materials pursuant to the MOPW's Standard Specifications for Road and Bridge Construction 1986.

As the respective moisture content of red soil and weathered rock is 3 ~ 6 % and 13 ~ 15 % lower than that of OMC, it should be regulated by sprinkling.

According to the interview, survey results, and the soil characteristics of the subject area the use of a sheepsfoot roller is considered appropriate for the compaction work.

(2) Shear Strength and Consolidation

Construction of a high bank is planned near Alliance Boys High School. Soil tests were conducted on samples taken from points TC23, TC24 and TC25, which are expected to become major supply sources of fill materials.

The subject soil is a red soil whose shear strength, when compacted to a dry density of at least 95% MDD with OMC, is as follows:

Bulk Density (kg/m^3)	Moisture Content (%)	Angle of Shear Resistance ø (degrees)	Cohesion C (KN/m^2)
1,703 - 1,752	32.8 - 37.6	12.6 - 22.2	198 - 389
(1,733)	(34.9)	(17.5)	(281)

note: ( ) = average

Shear Strength valves were obtained using a unconsolidated-undrained triaxial test.

Fig. 3.4.4 and 3.4.5 show consolidation when red soil is compacted

under the above conditions.

The settlement prediction of fill material in the high bank can be calculated, as shown in Table 3.4.2, using the results of the consolidation test and the following equation:

$$S = \frac{e_o - e}{1 + e_o} H$$

S = Settlement

e_o = Initial void ratio

e = Void ratio for P in e-log P curve

H = Thickness of fill (30 m)

Table 3.4.2 Calculation of Settlement

Filling Depth (m)	Effective Stress P (KN/m^2)	Final Void Ratio (e)	Total Settlement S (m)	
0 – 5	42.4	1.02	0.02	e = 1.03
5 – 10	127.2	1.010	0.05	γt 1.733 kg/m^3
10 – 15	212.1	1.005	0.06	H = 5 m
15 – 20	296.9	0.999	0.08	$P = \gamma e \times H$
20 – 25	381.7	0.993	0.09	
25 – 30	466.6	0.987	0.11	
		Total	0.41	

Table 3.4.2. shows that when the filling thickness is 30m, the maximum total settlement is 0.41m. Thus, it is predicted that soil will settle by 1.4% of the filling thickness.

(3) Bulking Factor of Fill Material

A study of bulking factor of the fill material was carried out in accordance with the following criterion;

a. The Standard Specification for Road and Bridge Construction of the MOPW.

b. The Highway earthwork series of the Japan Road Association (refer to Table 3.4.3).

c. The Bulking Factor Estimated by the Density Test

According to the MOPW's Standard, the Bulking factor is C = 0.8 (cuttining 1 m^3, fill 0.8 m^3). This can be used with any material, and common value, but it is smaller than the Bulking factor of the Japan Road Association. It seems that the main reason for this is the small scale earthwork and cutting of the shallow layer in Kenya relative to that in Japan

That is ;

a. The field denstity of the shallow layer is less than that of the deep layer due to weathering. Therefore, the margin of density is large comparing field density and hill density.

b. The earthwork has very little hard material of high bulking factor.

c. The rate of loss of material as a result of transportation is higher than for large scale earthwork.

d. The layer of fill material shall be compacted throughout to a dry density of at least 95% MDD, except for the upper 300 mm of the subgrade which shall be compacted to a dry density of at least 100% MDD. Therefore, if the height of the fill is low then the average degree of compaction is high.

The results of the field soil density test are shown in Table 3.4.4. Using these results and the compaction test results, the bulking factor is calculated according to the following equation.

$$\text{Bulking Factor (C)} = \frac{\text{Embankment Volume}}{\text{Cutting Volume}} = \frac{\text{Field Dry Density}}{\text{Compacted Dry Density}}$$

According to the MOPW's Standard Specifications for Road and Bridge Construction, compacting must be conducted for all fill materials in embankments except the 300 mm below formation: 95% MDD (AASHTO T99). The bulking factor calculation results required to meet this condition are shown in Table 3.4.5. Bulking factors for red soil and weathered rock are as follows;

Red soil : C = 0.80 ~ 0.89

Weathered Rock : C = 0.98 ~ 1.0

As shown in Fig. 3.4.6 red soil tends to change its bulking factor in relation to its depth. The Fig.s in Table 3.4.5 represent the test results on a highly weathered surface and, therefore, the general bulking factor is assumed to be higher than the test results. Based on the above observations and their implications, the following values are judged to be appropriate to indicate the bulking factors for the Nairobi Bypass.

– Soil : C = 0.85

(red soil, high weathered tuff high weathered trachyte)

– Soft rock : C = 1.0 (weathered trachyte and tuff)

– Hard rock : C = 1.2 (weathered and sound trachyte and tuff)

As it is practically impossible to determine precisely the bulking factor at the design stage, a test fill should be conducted during the construction period to determine the true value.

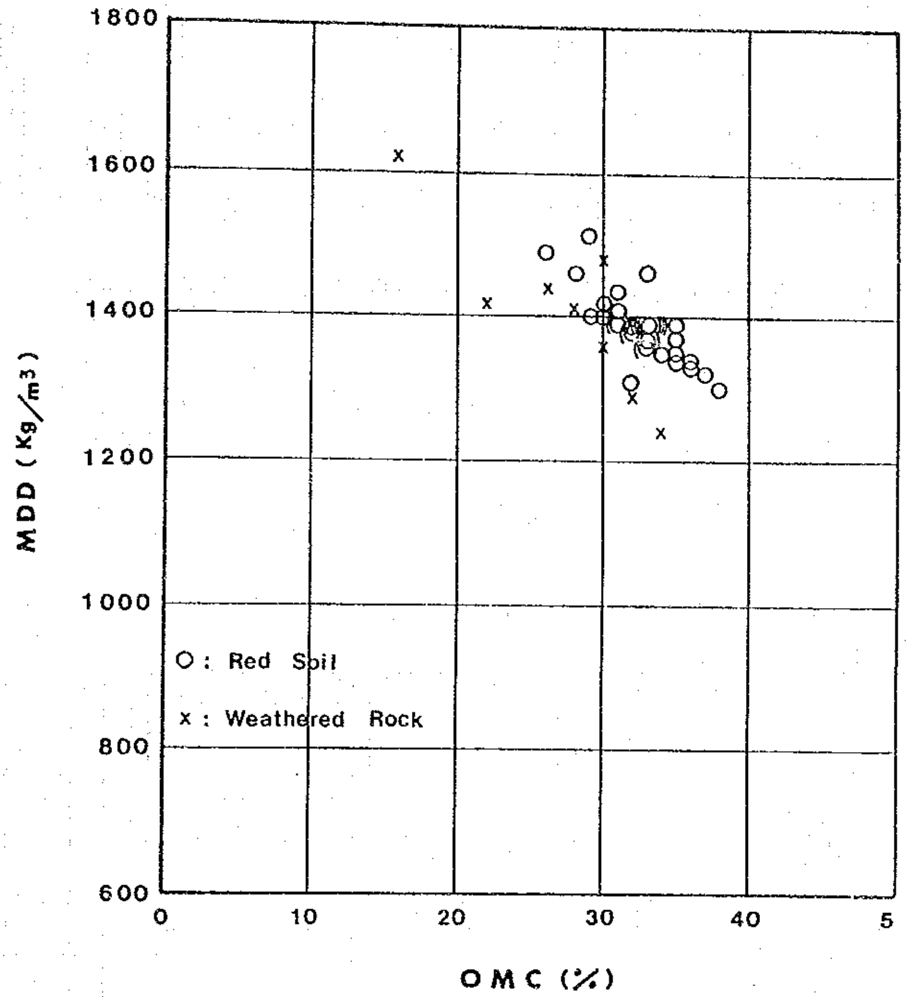


Fig. 3.4.1 OMC-MDD at Standard Compaction

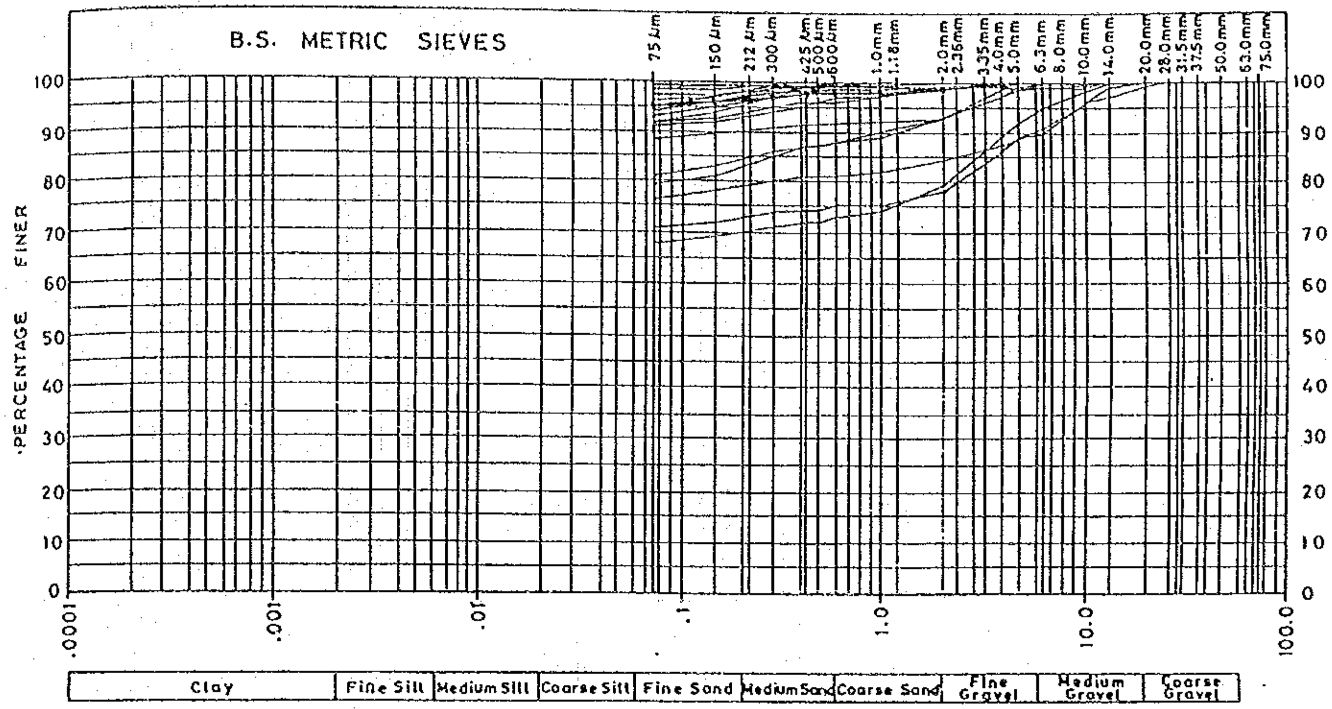


Fig. 3.4.2 Particle Size Distribution of Red Soil

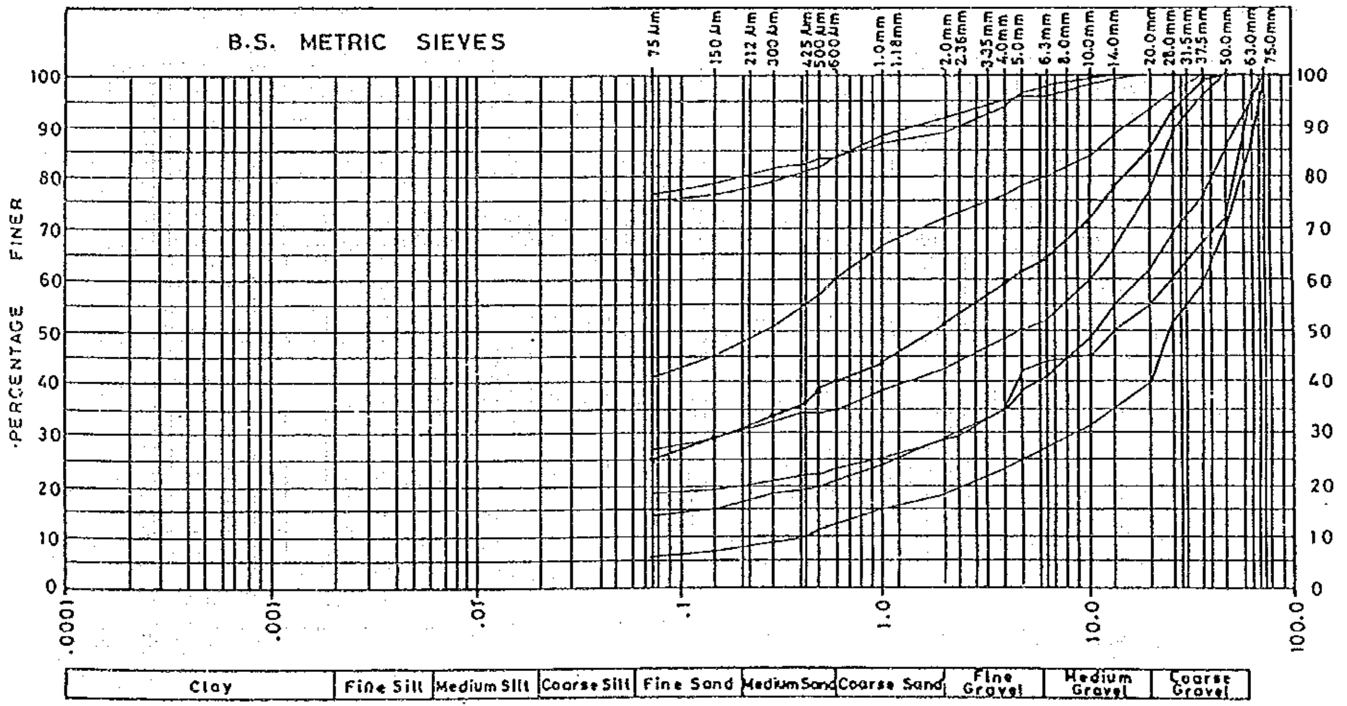


Fig. 3.4.3 Particle Size Distributio of Weathered Pyroclastic Tuff and Trachyte

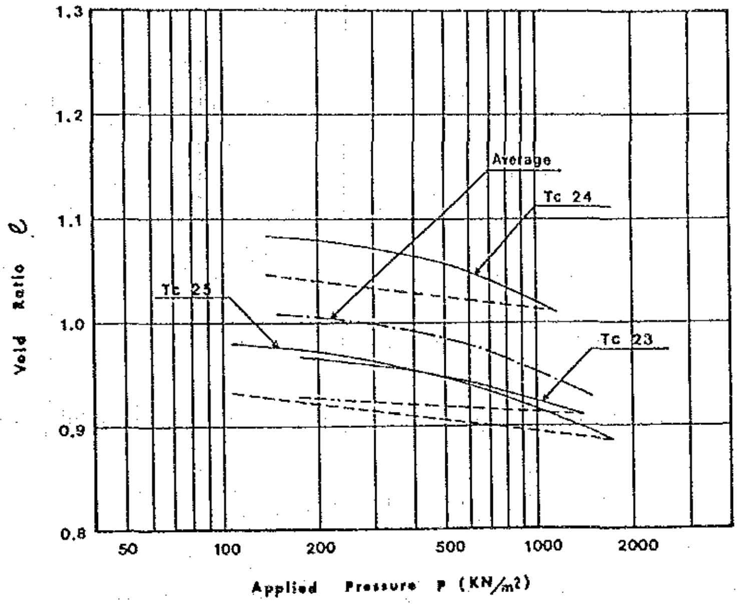


Fig. 3.4.4 e-log P Curve of Red Soil

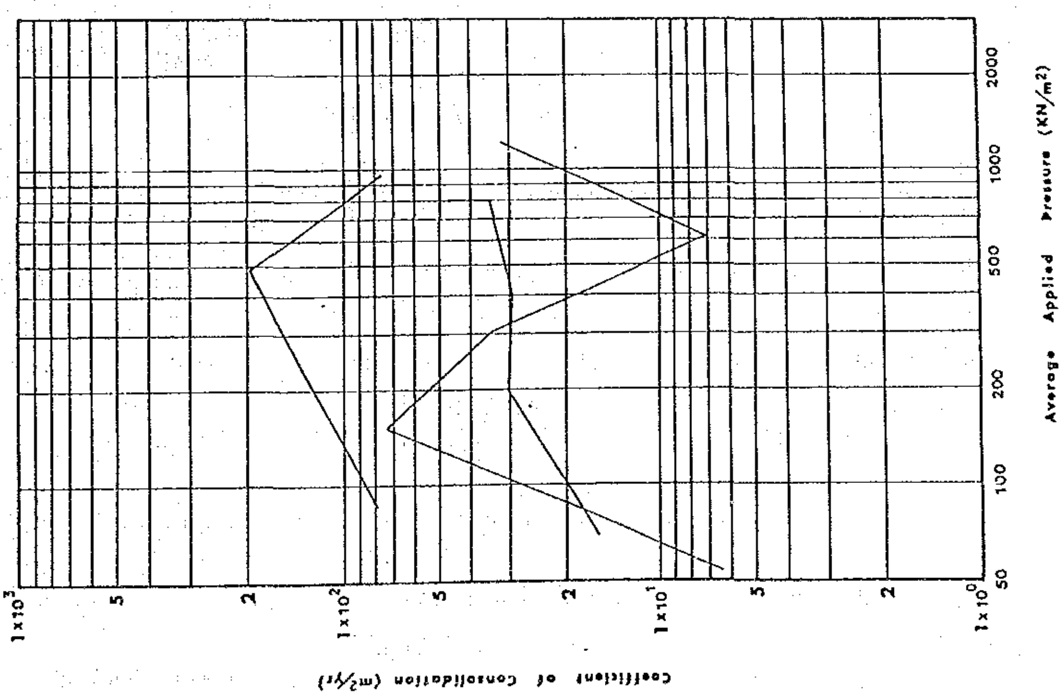


Fig. 3.4.5 log Cv - log P Curve of Red Soil $\overline{P}$

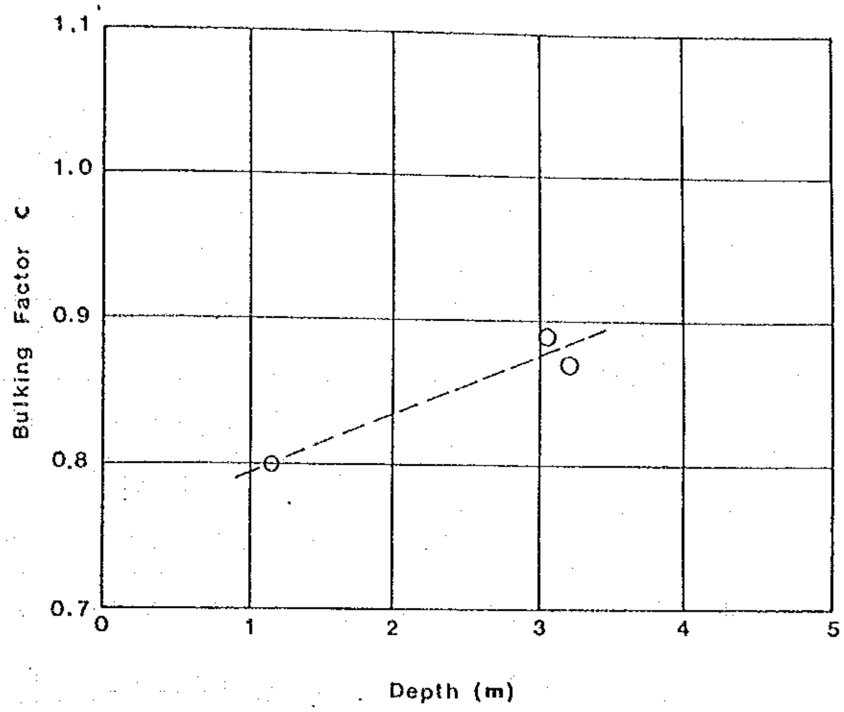


Fig. 3.4.6 Bulking Factor and Depth Relationship

Table 3.4.3 Bulking Factor of Soil

	Soil Type	C
Rock or Stone	Hard Rock	1.30 ~ 1.50
	Medium Hard Rock	1.20 ~ 1.40
	Soft Rock	1.00 ~ 1.30
	Boulder	0.95 ~ 1.05
Soil with Gravel	Gravel	0.85 ~ 1.05
	Gravelly Soil	0.85 ~ 1.00
	Hard Gravelly Soil	1.10 ~ 1.30
Sand	Sand	0.85 ~ 0.95
	Sand with Boulder	0.90 ~ 1.30
Soil	Sandy Soil	0.85 ~ 0.95
	Sandy Soil with Bolder	0.90 ~ 1.00
Clay	Clayly Soil	0.85 ~ 0.95
	Clayly Soil with Gravel	0.90 ~ 1.00
	Clayly Soil with Bolder	0.90 ~ 1.00

Highway earthwork series of the Japan Road Association

Table 3.4.4 Field Soil Density Test Results

Pit No.	Station	Depth (m)	Soil Type	Dry Density (kg/m3)	Moisture Content (%)
Tc-31	KM10+400	2.5~2.7	W.Rock	1,290	15.0
Tc-32	KM11+380	3.0~3.4	R.S.	1,080	26.0
Tc-33	KM11+880	0.8~1.0	W.Rock	1,320	10.2
Tc-35	KM14+155	1.0~1.3	R.S.	1,080	27.4
Tc-36	KM14+650	2.9~3.2	R.S.	1,270	23.6

W.Rock : Weathered Roak
R.S. : Red Soil

Table 3.4.5 Bulking Factor Calculation

Pit No.	Station	Depth (m)	Soil Type	① Field Dry Density (kg/m^3)	M.D.D. (T99) (kg/m^3)	② 95% M.D.D (kg/m^3)	Bulking Factor C = ① / ②
Tc-31	KM10+400	2.5~2.7	W.Rock	1,290	1,360	1,292	1.00
Tc-32	KM11+380	3.0~3.4	R.S.	1,080	1,310	1,245	0.87
Tc-33	KM11+880	0.8~1.0	W.Rock	1,320	1,420	1,349	0.98
Tc-35	KM14+155	1.0~1.3	R.S.	1,080	1,420	1,349	0.80
Tc-36	KM14+650	2.9~3.2	R.S.	1,270	1,510	1,435	0.89

W.Rock : Weathered Roak
R.S. : Red Soil

3.5. SLOPES

3.5.1 GENERAL

Existing slopes, in the vicinity of the proposed alignment, were examined. The items studied are shown in Appendix 5. All investigations took into consideration features including the environment of slope, slope structure, geology and type of failure. The locations of the investigated slopes are also given in Appendix 5. In total, there are three cut slopes and five hill slopes.

3.5.2 RESULTS OF THE INVESTIGATIONS

The data are given in Appendix 5. The results of investigations are as follows;

(1) Cut slope

1) Cut slope on the Red Soil

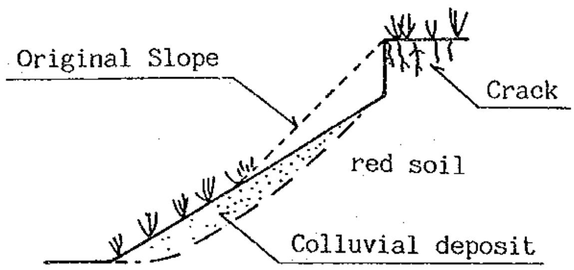
The outline of this slope structure is as follows;

- Slope angle : 28° ~ 45°
- Depth of cut : 3 m ~ 8 m
- Protection : Soding
- Drainage : None

Red soil is stable at shallow depths, however, if the depth of a cut is less than 5 m the stability angle is 90°.

It was found that erosion occured on slopes of angle 40° to 50°.

Fig. 3.5.1. illustrates the nature of the erosion.



This erosion was due to characteristic of red soil to be friable, like sand, when it is dry. (When red soil is wet, it has good cohesion).

Fig. 3.5.1 The Erosion of a slope is due both to rain and to its own weight when it is dry and has low cohesion. The depth of cut and

the angle of slope relationships to stability and erosion of the slope are plotted in Fig. 3.5.3.

The stability angle of colluvial deposits and the depth of cut relationships are also plotted in the same Fig.. This Fig. shows that the boundary angle between stability and unstability is approximately 37°. However, erosion also occured on some slopes, such as slopes No. 17 and 18 where the slope was gentle, because rainwater gathered and flowed from the shoulder of the slope berm to toe slope. Gully erosion occured in the locations where there was heavy flow. This clearly shows that drainage of surface water from a slope is important.

2) Cut Slope on Rock

Cut slope on rock investigations were carried out slopes No. 4 and 5. These slopes are formed from soft rock (Kerichwa Valley Tuff) where the rock, from the surface down to a depth of about 1 m, is weatherd and crumbled. As result, the degree of wearthering needs to be taken into account when deciding the angle of the slope.

(2) Fill Slope on Red Soil

The outline of this slope structure is as follows:

- Slope angle : 24° ~ 40°
- Height of fill : 4 m ~ 15 m
- Protection : Soding
- Drainage : None

There are both stable and unstable slopes in this category. Representative of the unstable slopes is slope No. 10 which has an angle of 40°. The mode of failure is shown in Fig 3.5.2. It appears that the reason for surface failure is the steep angle and the decrease in cohesion resulting from repeated wetting and drying of the soil.

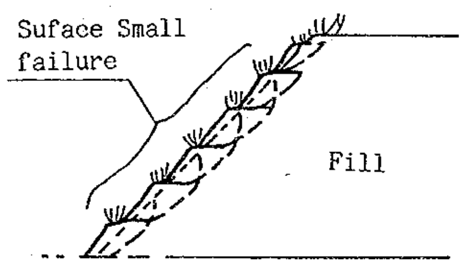


Fig 3.5.2. The slope of Failure in No.10

At other slopes of 40° angle surface failure or erosion occurs in some places. This, in conjunction with the stability angle of colluvial deposits, indicates that the stability angle of a red soil fill is 30° ~ 35°.

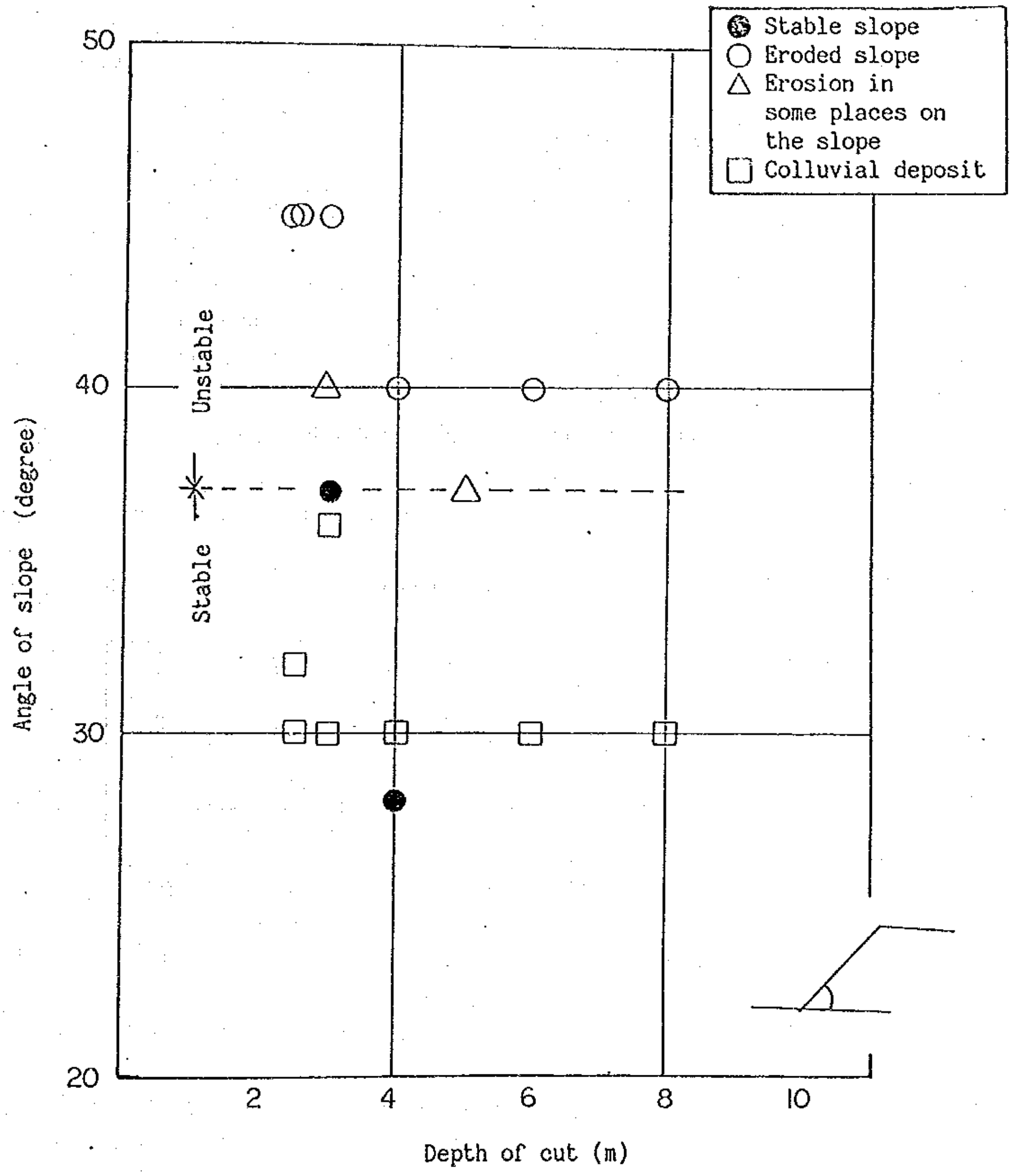


Fig. 3.5.3 Depth of cut-Angle of Slope Relationship

3.5.3 SLOPE STABILITY ANALYSIS OF HIGH BANKS

The slope stability of the high bank near the Alliance Boys High School was analysed because its height of 30 m gives rise to concern about its stability.

The stability anaylsis was made using Taylor's stability chart as shown in Fig. 3.5.4.

Conditions for the analysis are as follows:

- Cohesion of Fill $C = 281\ KN/m^2$
- Angle of Shear Resistancc of Fill $\phi = 17.5°$
- Bulk Density $\gamma t = 17.5\ KN/m^3$
- Height of Fill $H = 30\ m$
- Slope angle $\beta = 33.7°$ (1:1.5)

Fc and Fø are calculated as follows and calculation results are illustrated in Fig. 3.5.5. Since the safety factor is Fs = 4.1 it can be concluded that the 30 m high fill will be stable if built with red soil at a slope of 1:1.5.

Table 3.5.1 Calculations of Fc and Fø

ør	1/Ns	Cr=1/Ns H	Fc=Cu/Cr	F ø = tanøu/tan ø r
5	0.118	61.95	4.54	3.60
10	0.085	44.63	6.30	1.79
15	0.058	30.45	9.23	1.18
20	0.036	18.90	14.87	0.87

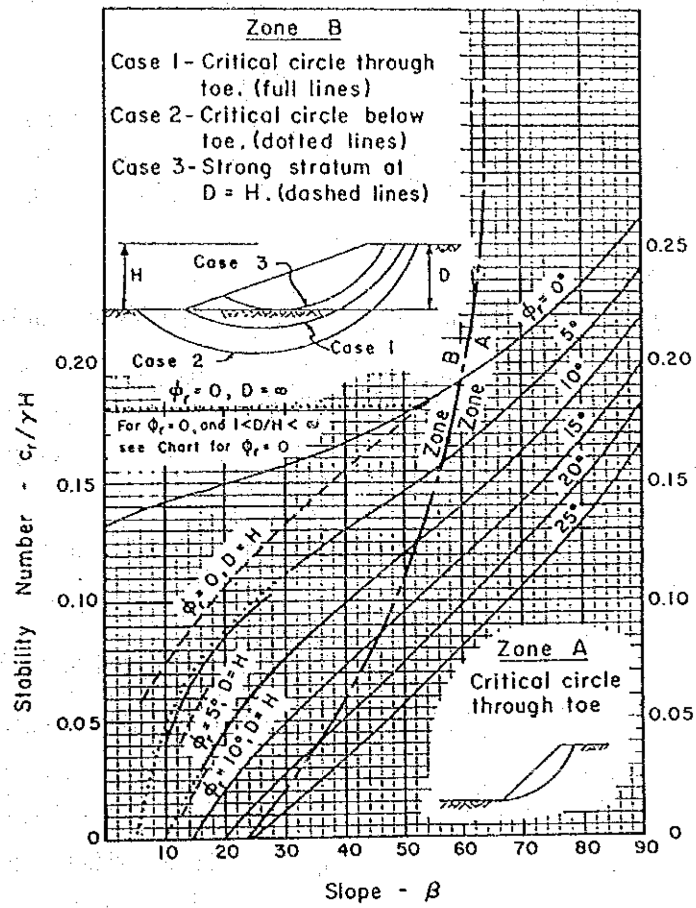


Fig. 3.5.4 Stability Numbers for Homogeneous Simple Slopes. (After Taylor, 1948.)

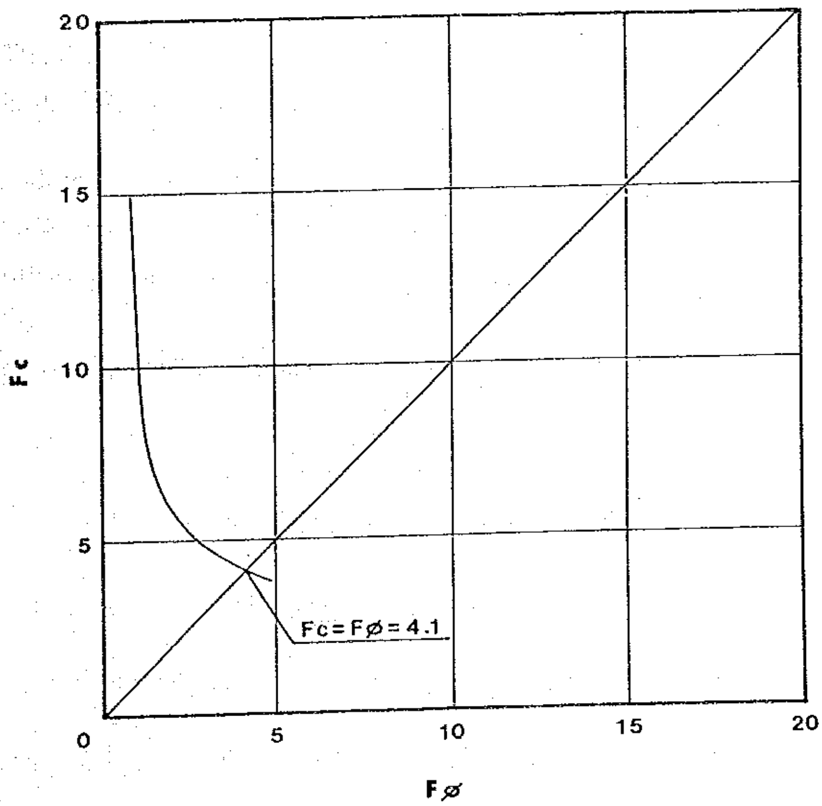


Fig. 3.5.5 Fø — Fc

3.6 SUBGRADE CONDITIONS

Test pits were dug at Sixty-one points to investigate the subgrade conditions.

As described earlier, in areas along the road alignment the subgrade is predominantly red soil with a partial presence of weathered rock, black cotton soil and lateritic gravel. APPENDIX 8 shows the results of the soil test on samples obtained from each test pit. The types of materials and PI as well as CBR values of each layer are shown below.

	Red soil	Weathered Rock	Black cotton soil	Lateritic gravel
*1Type of Material =	4	13 ~ 15	1	17
Plasticity Index =	13 ~ 30	NP ~ 21	14 ~ 43	15 ~ 24
CBR at 100% MDD 4 days Soak (%) =	6 ~ 30	9 ~ 65	1 ~ 4	5 ~ 70
Swell at 100% MDD 4 days Soak (%) =	0.1 ~ 1.7	0.2 ~ 1.4	2.7 ~ 5.5	0.1 ~ 0.9

*1 : see Table 3.6.2

The decision on the number of material types is based on the categories given in the MOPW's Road Design Manual Part III (Table 3.6.2).

Judging from Table 3.6.2 and the relationship between the soil class in the CBR range (also shown in Part III), the CBR values of red soil, pyroclastic tuff, black cotton soil, and lateritic gravel are 7 ~ 13 (S3), 10 ~ 30 < (S4 ~ S6), 2~5 (S1) and 15~30 < (S5 ~ S6) respectively. The MOPW's soil classifications are as follows:

Soil Class	CBR Range
S1	2 - 5
S2	5 - 10
S3	7 - 13
S4	10 - 18
S5	15 - 30
S6	> 30

The correlation between the plasticity index and the CBR value is confirmed by Table 3, ROAD NOTE 29 (Her Majesty's Stationary office, 1977) and ROAD NOTE 31 (Her Majesty's Stationery Office, 1977).

The correlation between red soil and the CBR value shown in Fig. 3.6.1 is assumed for the present survey. Despite scattering, a certain correlation can be observed and those CBR values outside the correlation range may be caused by the presence of gravel or unavoidable test errors.

The soil class and design CBR of the subgrade along the proposed alignment were studied, taking into account both the results of the soil tests and the major types of earthwork. Fig. 3.6.2 gives the results of this study, concurrently showing CBR (55 measured CBR, 4 inference CBR from PI), swell at 100% MDD 4 days soak and major types of earthwork. In the establishment of soil class and design CBR, the plans for hauling of earth has also been taken into consideration.

Since it was necessary to consider the dispersion of data from the total, the design CBR value was obtained using the following calculation method from the Japan Road Association's Manual for Asphalt Pavement.

Section CBR value = Average CBR value of individual locations - (CBR max - CBR min)/C Where C is a coefficient, values of which are listed in Table 3.6.1.

Table 3.6.1 Values of C for Calculating Section CBR Value

Number of values available	2	3	4	5	6	7	8	9	10 or more
C	1.41	1.91	2.24	2.48	2.67	2.83	2.96	3.08	3.18

The subgrade of black cotton soil is distributed in the section of KM0 + 00 ~ 5 + 600 and KM 11 + 00 ~ 13 + 748. This soil could not satisfy the following subgrade requirements given in the MOPW's Road Design Manual Part III (May 1981) and consequently, it needs to be replaced;

- CBR at 100% MDD (Standard Compaction) and 4 days soak: more than 5%

- Swell at 100% MDD (Standard Compaction) and 4 days soak: less than 2%

The soil class, and design CBR of the subgrade, was determined to be by soil type. Soil where the subgrade is predominantly red soil is classified as being

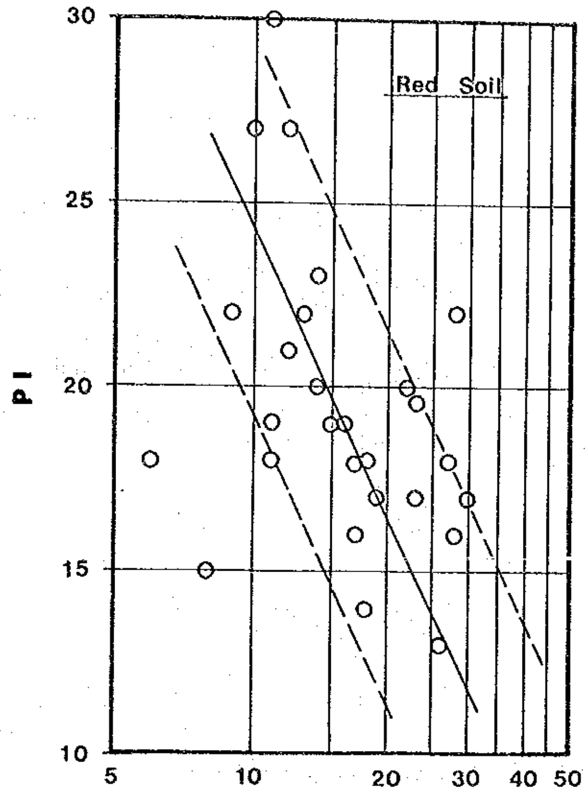
S4 while highly waethered rock is S5 and slightly weathered rock is S6. Soil class and design CBR by section were set, as follows based, on Fig. 3.6.2.

	Section	Soil Class	Design CBR
1	KM0 + 00 m – 6+ 300 m	S5	15
2	KM6 + 300 m – 15 + 400 m	S4	10
	(except. KM11 + 340 m – 13 + 700 m)	S6	30<
3	KM15 + 400 m – 20 + 800 m	S4	10
4	KM20 + 800 m – 23 + 400 m	S4	13
	(except. KM21 + 130 m – 21 + 600 m)	S6	30<
5	KM23 + 130 m – 28 + 700 m	S4	10

Table 3.6.2 Classification of Kenya Subgrade Materials

Type of material		Bearing Strength Class	
		After 4 days soak	At O.M.C.(Standard)
1	Black cotton soils	S1	S5
2	Micaceus silts (decomp.rock)	S1	S3
3	Other eluvial silts (decomp.rock)	S2	S4
4	Red friable clays	S3	S5
5	Sandy clays on volcanics	S3 or S4	S5
6	Ash and pumice soils *	S3 or S4	S5
7	Silty loams on gneiss and granite	S4	S5
8	Calcareous sandy soils	S4	S5
9	Sandy clays on basement	S4	S5
10	Clayey sands on basement	S4 or S5	S5 or S6
11	Dune sands	S4	S4 – S5
12	Coastal sands	S4	S5
13	Weathered lava	S4 or S5	S5 or S6
14	Quartzitic gravels	S4 – S6	S5 or S6
15	Soft (weathered) tuffs	S4 – S6	S5 or S6
16	Calcareous gravels	S4 – S6	S5 or S6
17	Lateritic gravels	S5 or S6	S6
18	Coral gravels	S5 or S6	S6

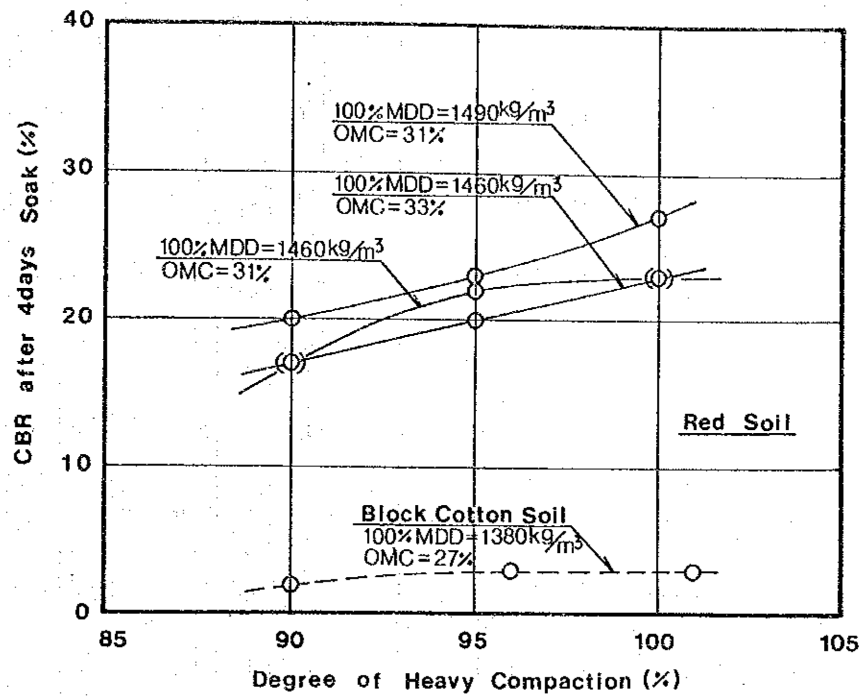
* Some of the ash pumice soils have a very low maximum dry density and a lower Young's Modulus than might be expected from the measured CBR values. Such soils (Standard Compaction MDD Less than 1.4 Mg/m^3) cannot be classified for pavement design purposes on the basis of CBR only.



CBR at 100% MDD 4 days Soak (%)

Fig. 3.6.1 CBR—PI Relationship

Fig. 3.6.2 Degree of Heavy Compaction — CBR Relationship



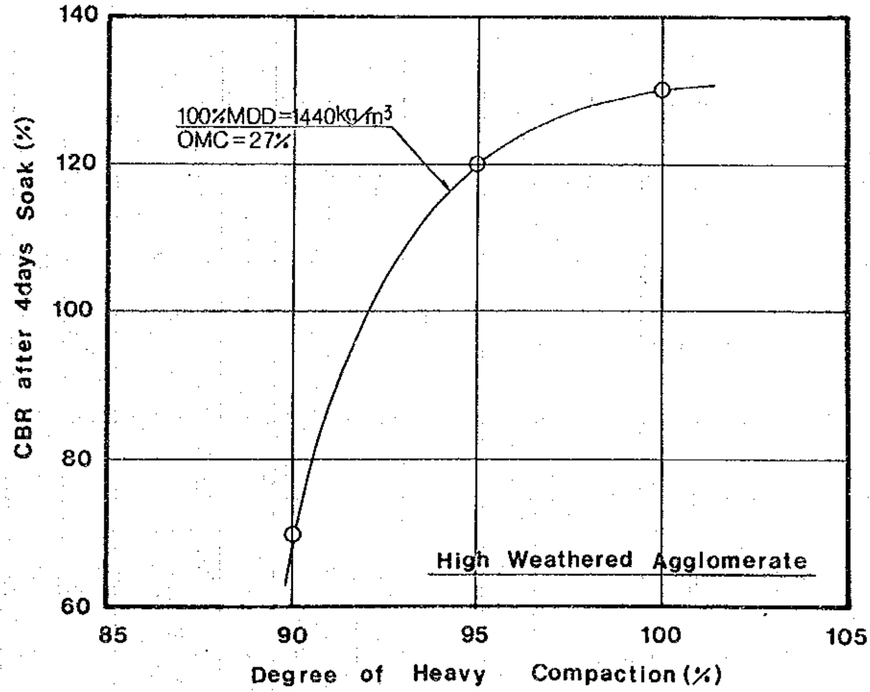


Fig. 3.6.3 Subgrade Conditions

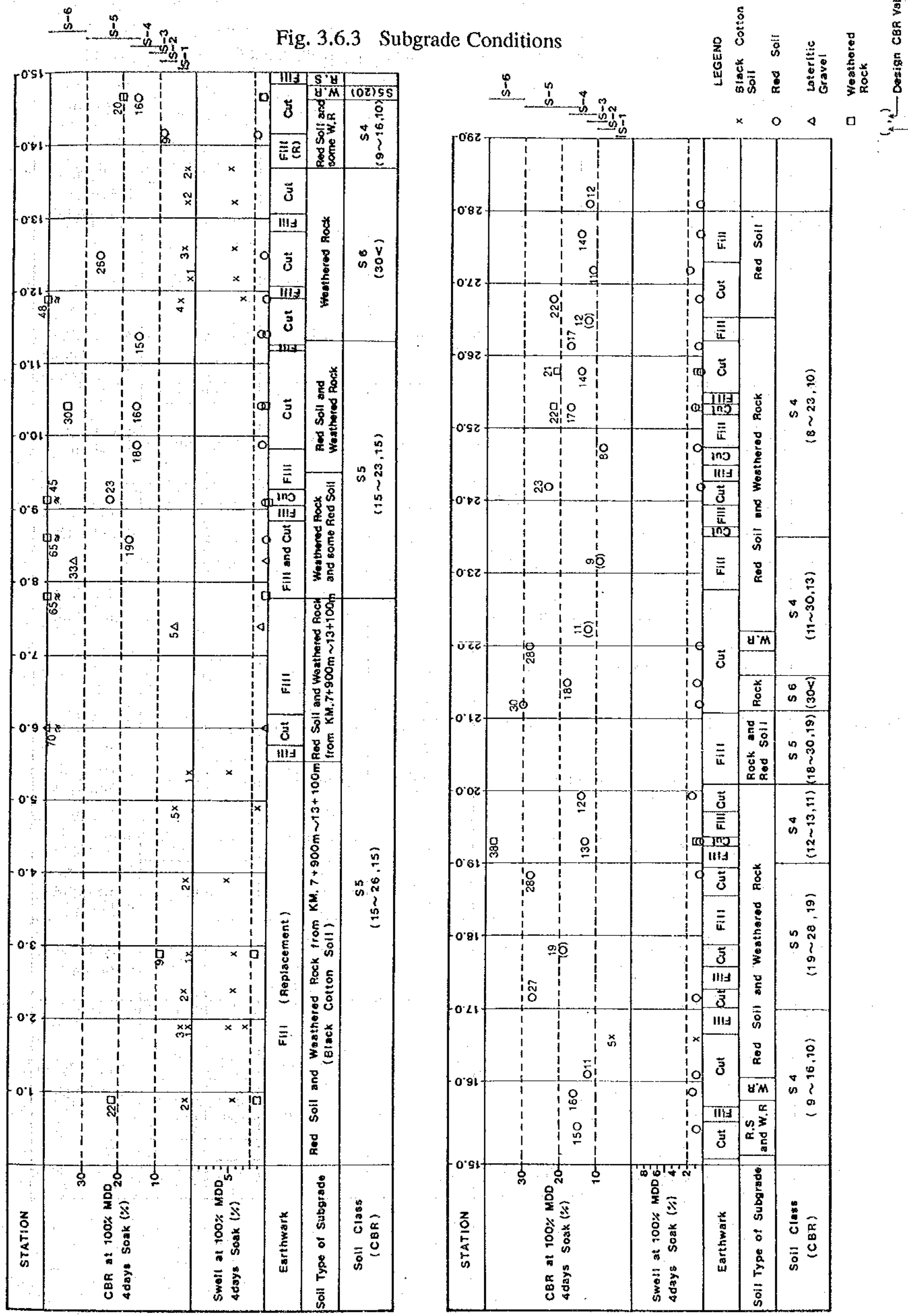


Table 3.6.3 Design CBR value based on calculation method of the Japan Road Association's Manual for Asphalt Pavement

Station	CBR (measured)	Average CBR value	CBR max	CBR min	C	Design CBR value
KMO+00 ~KM5+600 (Original)	2 2 2 1 2 5 1 5	2.5	5	1	2.96	1.1
KM0+00 ~KM5+600 (after replacement)	23 18 16 15 26	19.6	26	15	2.48	15.2
KM5+600 ~KM7+780 (Fill)	23 18 16 15 26	19.6	26	15	2.48	15.2
KM7+780 ~KM11+340	19 23 18 16 15	18.2	23	15	2.48	15.0
KM11+340 ~KM13+700	30<					
KM13+700 ~KM14+625 and KM14+825 ~KM17+00	9 16 15 16 11	13.4	16	9	2.48	10.6
KM17+00 ~KM19+00	27 19 28	24.7	28	19	1.91	19.9
KM19+00 ~KM20+110	13 12	12.5	13	12	1.41	11.8
KM20+110 ~KM21+100	30 18 28	25.3	30	18	2.24	19.9
KM21+100 ~KM21+100	30<					
KM21+580 ~KM28+500	30 18 28 11	21.8	30	18	2.24	13.3
KM23+500 ~KM28+500	23 8 17 14 17 12 22 11 14 12	15.0	23	8	3.18	10.3

3.7 GROUND CONDITION OF SOAK PIT

According to plans, the soak pit will be located near KM.24 + 200m (see Appendix 13). Two drillings and an in-situ permeability test were conducted at the site.

The soil formation consists of red soil from 0 m to 10 m in depth and, at greater depth, of Trachyte.

Red soil is a stiff to hard silty clay while the trachyte is highly weathered in the upper 4 m to 5 m layer and is essentially a clayey gravel. Further below this the trachyte is only slightly weathered and the RQD becomes about 45%.

This survey found no ground water (down to 15 m) and little or no see page water was observed although an injection method permeability test was performed at boring BS2.

Red soil is considered to be an impermeable layer but the existence of small cracks results in it having some permeability. Despite the fact that the terrain in this region forms a sort of basin, without a water outlet, surface run off is not normally observed.

The trachyte in general is considered to have poor permeability when it is in the advanced stage of weathering and is clayey.

However, in this slightly weathered formation, the trachyte is believed to have some permeability since it has cracks. Sewage soak pits, for schools in this basin, are under construction using this formation as the permeability layer which indicates that it has been found to be permeable.

4. MATERIAL INVESTIGATION

4.1 GENERAL

The feasibility study called for the purchase of pavement materials and concrete aggregate. Its purpose was to locate new sources of materials and to investigate existing quarries because of the planned relocation of these quarries and rising market prices of rocks.

The survey was carried out by conducting site investigations for both hard rock and gravel sites. Samples have been collected from some sites and taken for testing. At other sites no samples were collected, either because the material was obviously poor or because there was not enough available. With both the rock and gravel samples various tests were carried out in accordance with the MOPW's Road Design Manual Part III.

4.2 GRAVEL

4.2.1 GRAVEL MATERIAL SITES

The location, site plans and test results of existing and new sites are shown in Fig. 1.1.2 and Appendix 6.

(1) Existing Gravel Material sites

1) Karai Material Site

The gravel extracted from this site was used in construction of the Rural Access road serving the area. The material is mainly composed of a whitish decomposed trachyte. This site cannot be used so there were no further investigations made.

2) Lusigeti Material Site

This site is about 10 km from the Proposed road. The material from the site had been used to improve the access roads serving the area especially for the newly created settlement area.

3) Mutuini Material Site

Material extracted from this site had been used to improve the roads serving the vicinity of Dagoretti Market and other private roads. The site showed good lateritic gravel, but digging of trial pits showed

that the material site was not extendible. Estimated available quantity is 400 m^3.

4) Kiriba Material Site

The exposed faces of this site showed good lateritice gravel. However, trial pits indicated that the material is not wide spread.

Material extracted from the site has been used for private developments and localized communal access roads. Estimated available quantity is 12,600 m^3.

5) Kabiria Material Site

This site also showed good lateritice gravel. However, the site looked exploited owing to the proximity of the exposed face and the buildings there-in. This site was the source of some materials used to construct the University Graduation grounds.

(2) New Gravel Material sites

1) Thandi Material Site

This is a virgin site and has an estimated volume of 5,530 m^3. It has an average of 0.5 m overburden.

2) Dagoretti Material Site

This site has an estimated volume of 6,480 m^3. The average overburden is 0.7 m.

3) Galsheet Material Site

This site is very large and has a very high potential. The average overburden is 0.4 m. The estimated volume of gravel in excess of 39,000 m^3.

4) Carnivore Material Site

During the trial pits excavation, it was revealed that the gravel was only an upper layer of 0.3 m in depth and the underlying bed was of soft rock. The volume of gravel is estimated to be 100 m^3.

5) Forest Guard Camp Material Site

This site has very good accessibility in relation to the proposed road. The average overburden is 0.4 m. The volume of gravel is estimated to be greater than 21,200 m^3.

6) Karen Material Site

The average overburden is 0.8 m. The estimated volume of gravel is in excess of 3,600 m^3.

7) Church Material Site

The average overburden is 0.4 m. The estimated volume of gravel is in excess of 22,050 m^3.

8) Waiganjo Material Site

Trial pits revealed that the site consists of only pockets of gravel. The estimated volume of gavel is 100 m^3.

9) Race course Material Site

The trial pits revealed, on inspection, that the gravel quantity was poor.

10) Kibera Material Site

Though the surface indicated the presence of gravel, excavation of trial pits revealed that the site has a greater abundance of soft rock. Thus, no further investigations were carried out.

The estimated volume of gravel is 6,160 m^3.

4.2.2 QUALITY

Items for soil tests at each material site are as given below and test results are as shown in Table 4.2.1 and in APPENDIX 10.

- Grading to 0.075 mm sieve
- Atterberg Limtis (LL. PL. LS. PM)
- Compaction test (Heavy Compaction: 4.5 kg)
- CBR and swell at 4 days soak on specimens mounded at O.M.C. (Heavy Compaction) at 90, 95 and 100% MDD.

- Cement Stabilization 2, 4 and 6% (Compaction test (Heavy Compaction), CBR at 7 days cure plus 7 days soak on specimens moulded at OMC and 95% MDD)
- Lime Stabilization 2, 4 and 6% (Same cement method)

The results of stabilization tests with cement and lime are as shown in Fig. 4.2.1 and Fig. 4.2.2.

The test results can be summarized as follows;

Gravel that is distributed in this region is all lateritic gravel containing clay and silt (10% to 30%), and sand (10% to 20%). Consequently, depending on clay and silt content, the characteristics of the gravel change and CBR substantially varies from 3% to 190% with 95% MDD.

For stabilization treatment, cement is more effective than lime.

4.2.3 ANALYSIS AND RECOMMENDATIONS

The gravel was evaluated as a pavement material based on the results obtained.

Gravel may be used in any of the following three cases: natural materials for sub-base, cement or lime improved material for sub-base and cement stabilized gravel for base. According to the Road Design Manual Part III, major requirements for each of these materials are as follows:

- Natural Materials for Sub-base

• CBR at 95% MDD (Modified AASHTO) and 4 days soak	Min. 30
• Plasticity Index	Max. 15
• Plasticity Modules	Max. 250

- Cement or Lime improved Material for sub-base

• Maximum size	10 - 50 mm
• Passing 0.075 mm sieve	Max. 40%
• Plasticity Index:	Max. 30
• Plasticity Modulus:	Max. 2,500
• CBR of Laboratory mix at 95% MDD (Modified AASHTO) and 7 days cure + 7 days soak after treated	Min. 60

- Cement Stabilized Materials for Base

• Maximum size	2 - 40 mm	
• Passing 0.075 mm sieve	Max. 35%	
• Plasticity Index	Max. 25	
• Plasticity Modulus:	Mix in place	Max. 1,500
	Min. in plant	Max. 700
• UCS of Laboratory mix at 95% MDD (Modified AASHTO) and 7 days cure + 7 days soak	Min. 1,800 KN/m^2 (18kg/cm^2)≒ CBR 180	

Refer to Table 4.2.2 for the area of each site, overburden, thickness, volume available for use, and acceptability as above materials.

Material sites must meet the following conditions:

- Prescribed quality must be obtainable.
- Required quantity can be obtained (about 190,000 m^3 for sub-base and shoulder, and about 110,000 m^3 for base)
- Easy to obtain

The available volume at each site is generally small, and no single site can supply the required quantity. Sites that can offer the possibility of supplying more than 10,000 m^3 are limited to Kiriba, Galsheet, Forest Guard, Karen and Church.

However, generally these sites have only a thin gravel layer about 0.3 m to 1.0 m in depth. Overburden is not thick, being 0.7 m to 0.8 m in depth. Sampling efficiency is generally poor and Galsheet in particular, could not be used. The results showed that gravel cannot qualitatively satisfy the requirements for either sub-base or base.

The following are sites which offer possibility of exploitation. A summary of application for each type of material is also given:

Site	Sub base	Base
Kiriba	• Cement or lime improved material	• Cement stabilized gravel
Forest Guard	• Cement or lime improved material	• Cement stabilized gravel
Karen	• Cement of lime improved material	• Cement stabilized gravel
Church	• Natural material • Cement or lime improved material	• Cement stabilized gravel

Unfortunately, about half of the Forest Guard's are private plots and land owners have not agreed to allow them to be used. In addition, Karen and Church are located within the Ngong Road Forest. It is conceivable that, because of protectionist movements, development will not be possible.

Table 4.2.1 Laboratory Test Results of Gravel

Site	Atterberg Limits		Linear Shrinkage (%)	Plasticty Modulus (%)	Grading (%)			Compaction (heavy)		CBR (%)		
	LL(%)	PI			Clay & Silt	Sand	Gravel	M.D.D. (kg/m³)	O.M.C. (%)	% of M.D.D.	4 day Soak	Swell
Mutuini	45~49	13~19	7~9	312~722	19~34	13~16	50~68	1540~1760	20~24	95	30~35	0.1 ~0.4
Kiriba	44~51	16~20	9~10	204~860	9~38	7~11	51~81	1750~1960	15~27	90~91	14~160	0.02~0.04
										95~96	19~190	0.04~0.07
										100~101	22~209	0.05~0.15
Thandi	35~51	13~19	7~10	260~836	16~39	12~15	49~69	1730~1890	15~20	90	7~26	0.04~0.2
										95	20~30	0.07~0.33
										100	27~45	0.04~0.47
Galsheet	46~72	11~31	6~15	143~558	7~20	3~14	73~79	1540~1710	20~28	90~91	3	0.5 ~0.7
										95~96	3~30	0.4 ~1.8
										100	5	1.1 ~3.4
Garnivore	41~51	13~18	6~9	65~615	3~39	8~14	53~86	1630~2000	14~20	90~91	6~40	0.1 ~0.3
										95	7~50	0.2 ~0.6
										100~101	10~170	0.2 ~1.1
Forest Guard	44	17	8	110	5	16	79	2030	15	96	90	0.1
Karen	34~41	11~13	5~7	240	10~17	9~12	71~78	1710~1970	15~17	90	12~26	0.11~0.16
										95	21~65	0.22~0.28
										100	30~110	0.28~0.39
Church	37~51	10~15	5~8	180~465	15~26	11~14	60~74	1710~1910	18~21	90~91	16~85	0.05~0.09
										95~96	35~100	0.13~0.18
										100~101	75~170	0.18~0.23
Waiganjo	NP	NP	2	0	2	19	79	1240	29	90	90	0.2
										95	90	0.3
										101	190	0.4
Kibera	50	19	9	437	20	8	72	1510	24	89	11	0.1
										95	13	0.1
										100	24	0.2

Table 4.2.2 Quantity and Quantity of Gravel

Site	Area (m^2)	Average Over burden (m)	Average Thickness (m)	Expected Volume (m^3)	Natural Materials for Subbase	Cement or Lime improred material for Subbase			Cement Stabilized Gravel for Base		Remarks
							Cement (%)	Lime (%)		Cement (%)	
Mytuini	—	—	—	400	△	○	2.0	2.0～4.4	△	2.2～4.5	
Kiriba	12,600	0.8	1.0	12,600	△	○	2.0	2.0～2.1	△	3.4～5.0	
Thandi	9,220	0.7	0.6	5,530	×	○	2.0	2.8～6.0	△	2.0～4.5	
Galsheet	115,200	0.7	0.3	34,500	×	○	2.0～2.1	2.0～2.4	△	3.1～5.0	
Carnivore	—	—	0.3	100	△	○	2.0	2.0	△	2.0	
Forest Guard	23,550	0.5	0.9	21,200	△	○	—	—	○	—	Owner could not agree
Karen	39,630	0.7	0.8	31,700	△	○	2.0	2.0	○	2.3	Ngong Road Forest
Church	35,630	0.7	0.8	28,500	○	○	2.0	2.0	○	2.0	Ngong Road Forest
Waiganjo	—	—	—	100	○	○	2.0	2.0	○	2.0	
Kibera	—	—	—	6,160	×	○	2.0	2.5	○	3.4	Owner could not agree
Dagoretti	10,800	0.5	0.6	6,480	×	×	—	—	×	—	
Karai	—	—	—	—	—	—	—	—	—	—	Owner could not agree
Kabira	—	—	—	—	—	—	—	—	—	—	
Race Course	—	—	—	100	—	—	—	—	—	—	

Note : ○ Appropriate
△ Unjustified on some part
× Unjustified

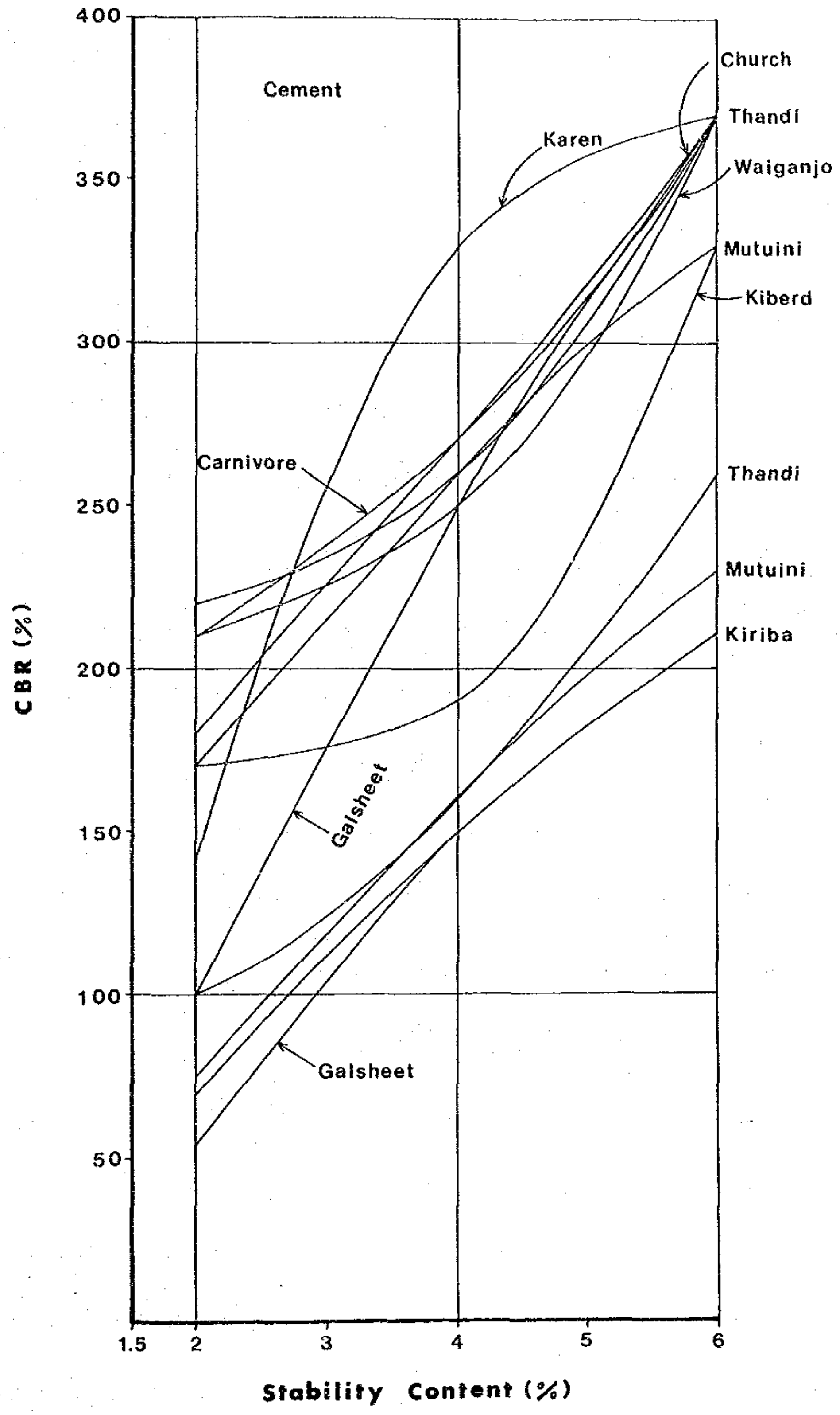


Fig. 4.2.1 Cement Stability Content — CBR Relationship

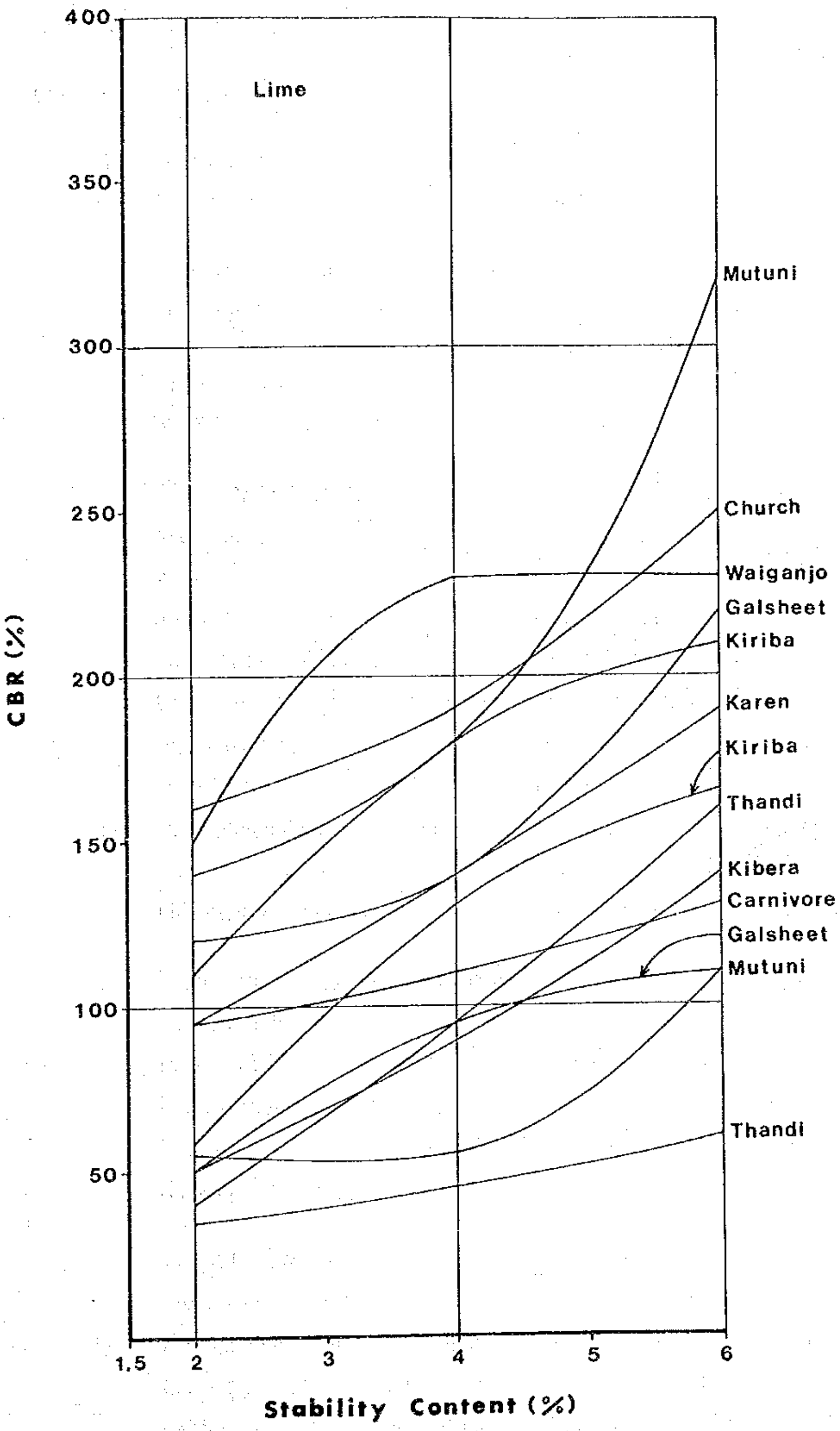


Fig. 4.2.2 Lime Stability Content — CBR Relationship

4.3 HARD STONE

4.3.1 HAND STONE QUARRY SITES

(1) Existing Quarries

1) Nairobi Eastern Quarries

Seventeen existing commercial rock quarries were identified. Sixteen of them are concentrated in the Eastern side of Nairobi in the vicinity of the Kariobangi, Dandora and Kayole housing estates.

They exist as a chain close to each other but are spread over an area of approximately 20 sq. km.

The rock in this area is Nairobi phonolite which is hard and can be used for all the layers of the road pavement (i.e. sub-base, base and wearing surface). Each one of the quarries has abundant material and the proprietors are willing to enter into negotiation with the road construction contractor. Indeed, some quarry owners are themselves road contractors and, no doubt, shall seek the opportunity to make a tender for the Nairobi Bypass. More significantly, however, is the fact that the material has been used in the construction of roads in the vicinity including the Kangundo-Dandora road which was constructed in mid 1970's for the Government of Kenya.

These quarries are fairly large areas of land which are pieces either owned or hired on long leases. At the time of the study most of the leases still had an average of 40 years before they expire. The land is leased, for the most part, from the Nairobi City Commission. It is otherwise useless land along the banks of the Nairobi and Ngong rivers and/or lying on the Electrical Power Wayleaves.

The sites appear potentially expansive but there is a problem since these quarries have been recently issued a stoppage notice by the Kenya Power and Lighting Co. The notice is the result of dust from the quarries causing serious damage to the high tension electric cables in the Vicinity. This notice will expire at the end of the year. The Quarry operators have launched an appeal against the notice however, it is unknown it will be successful.

2) DBT Quarry

This is the seventeenth existing commercial rock quarry. It is situated about 25 km from the starting point of the proposed road. The existing crushing plant produces an aggregate of reasonable quality which can be considered as a possible source. A sample was collected and taken for testing.

3) Rungiri Quarry

This rock quarry was in the process of being opened by the contractor for the Kabete-Limuru Road Project. A sample was collected and taken for testing.

(2) New Quarry Sites

1) Kitengela Quarry

This quarry is located off Namanga-Arusha (A 104) about 2 km from Athi River Town. The site is extensive and lies on Government Land. Visual inspection indicates that it has very high quality rocks. A sample from this site was collected by drilling for testing of its quality characteristics. This source is approximately 23 km away from the project. Average overburden is 1.1 m. The estimated quantity in this site is in excess of 324,000 m^3.

2) Muthiga Rock Quarry

This site is about 2 km from the Kikuyu Turn off on the Nairobi-Nakuru Road (A 104) which is near the western end of the proposed road. The site is located in a highly agriculture productive area which is served by both electricity and piped water. The quarry site is on the banks of the Nairobi river. A rock sample was collected from the surface and taken for testing. (see Appendix) Drilling was carried out, on the basis of the results obtained, to determine both the quantity and quality available.

3) Max.-Inn Rock Quarry (National Park)

This site is off Mombasa Road opposite the existing weigh-bridge site. The site is located within the National Park. A rock sample was collected and taken for testing.

4) Kibera Rock Quarry

This quarry can only be regarded as a source of soft rock so further investigations were not necessary.

5) Carnivore Rock Quarry

This quarry was observed to contain medium hard rock. A rock sample was extracted and taken for testing of its quality characteristics. The site is just next to the proposed road.

4.3.2 QUALITY

(1) New Quarry site

Items for the soil tests conducted at each site are as follows with the results given in Table 4.3.1 and in APPENDIX 11:

- Los Angeles Abrasion (LAA)
- Aggregate Crushing Value (ACV)
- Sodium Sulphate Soundness (S.S.S)
- Specific Gravity (oven-dry method)
- Plasticity Index on L.A.A. fines
- Bitumin Affinity (Binder: MC3000, KI-60, 80/100)
- Grading to 0.075 mm sieve *
- Flakiness Index * (FI)

Note: * Grading and FI sample should be crushed with a small Jaw crusher, to a size which depends on the proposed use of the stone (20 mm to 40 mm).

The test results can be summarized as follows:

The sites consists of Trachyte, Tuff and Phonolites. Both Trachyte and Tuff are soft and their ACV (31% - 48%), LAA (46% - 63%), S.S.S. (22.0% - 59.5%) and Water Absorption (4.9% - 24.3%) are too large for aggregate.

Phonolite includes Kapiti Phonolite and Nairobi Phonolite. Both of which are hard and of high quality as aggregate with small values of

ACV (17% - 21%), LAA (18% - 22%), S.S.S. (2.0% - 5.1%), and Water Absorption (2.0% - 4.4%).

However, FI of Phonolite is as high as 26% - 39% and liable to be broken in flat.

(2) Existing Quarry Sites

Tests were conducted at the quarry site (Rungiri) on the Kabete-Limuru Road. DBT, which is not scheduled for relocation, was also tested in the same manner as the new quarry sites. The results are given in Table 4.3.1 and APPENDIX 11. The quality of aggregate that can be supplied from the existing quarries in the area included in the relocation plan is shown in Table 4.3.2. Rungiri produces Trachyte which is too soft for aggregate as is the Trachyte available from the new quarries.

Other existing quarries can supply Phonolite which is hard and suitable as aggregate with ACV (16% - 22%), LAA (12% - 27%), Bitumen affinity (good). However, FI gives a large value of more than 30% at DBT and Diamond and is liable to be broken in flat. At the other sites, FI = 15 to 20 which is believed to be due to the difference in crushing methods used since any substantial change in rock characteristics is inconceivable.

4.3.3 ANALYSIS AND RECOMMENDATIONS

Based on these results, each site was evaluated as a possible source of pavement materials and concrete aggregate. Regarding the use of pavement materials, the following four cases can be considered: graded crushed stone for sub-base and base, dense bitumen and macadam for base, lean concrete for base and asphalt concrete.

According to the Road Design Manual Part III, the major requirements for each application are as follows:

- Graded Crushed Stone for Sub-base

• LAA	Max. 45%
• ACV	Max. 32%
• S.S.S.	Max. 20%
• FI	Max. 35%

- PI on Fines NP

- Graded Crushed Stone for Base
  - LAA Max. 30%
  - ACV Max. 25%
  - S.S.S. Max. 12%
  - FI Max. 25%
  - PI on Fines NP

- Dense Bitumen Macadam for Base
  - FI Max. 25%
  - LAA Max. 35%
  - ACV Max. 28%
  - S.S.S. Max. 12%

- Lean Concrete for Base
  - FI Max. 25%
  - LAA Max. 35%
  - ACV Max. 28%
  - PI on Fines NP
  - S.S.S. Max. 12%

- Asphalt Concrete
  - LAA Max. 35%
  - ACV Max. 28%
  - S.S.S. Max. 12%
  - FI Max. 25%

- Coarse Aggregate for Concrete
  - Passing 0.075 mm sieve Max. 1%
  - S.S.S. Max. 12%
  - FI Max. 35%

- Water absorption Max. 2.5%
- ACV Max. 35%
- LAA Max. 50%

The following sites almost satisfy these requirements:

- Kitengela
- Max.-in
- Existing quarries (except for Rungiri)

However, FI slightly exceeds the standard value at Kitengela, Max.-in and Diamond. It is believed materials tend to become flat because, for testing, rocks are crushed with jaw crushers. Thus, it is necessary to develop an appropriate crushing method. For example, an impact crusher could be used, decreasing the possibility of rocks being broken.

If a jaw crusher is used, the size of the rocks should be adjusted by further crushing.

Regarding the available rock production at the Max.-in site, there is a high possibility that there is a thick weathered layer present. In view of these topographical conditions, even if rocks are obtained, it is believed there will be considerable loss.

At Kitengela site, rocks of good quality are widely exposed in the surface layers. Drilling results also indicate that at least 324,000m^3 can be mined to fully meet the requirements for both pavement materials and concrete aggregate.

The problem is the site's remote location, 23 km away from the projected alignment. Even with this disadvantage, Kitengela site is considered the best suited as a new quarry for the following reasons; its close proximity to A104 means it requires only a very short access road, A104 runs to the projected alignment and a good road is available. Max.-in site, on the other hand, requires a long access road which needs to cross the railway. In addition, the site is located in the national park.

The current survey shows that there is a high possibility that, with the exception of DBT, the existing quarries will be relocated. Also, according to a questionnaire survey directed at several firms (as detailed in APPENDIX 12)

the price of rock has been rising at an abnormal rate this year. The quarries are believed to be moved to sites to the East of Max.-in. It is anticipated that the relocation will begin by 1991.

For these reasons, Kitengela site is considered to be the best suited as the source of hard stone.

Table 4.3.1 Laboratory Test Results of Quarry Site

Site	Rock Type	Grading % passing								ACV	LAA	PI on LAA Fines	FI	Bitumen Afinity (%)			S.S.S.	Specific Gravity	Water Absorption (%)
		37.5 mm	20 mm	10 mm	6.3 mm	2.36 mm	1.18 mm	425 μm	75 μm	(%)	(%)		(%)	KI-60	MC 300	80/100	(%)		
Kitengela	Kapiti Phonolite	90~100	55~69	29~31	16~17	7~8	4	0~2	0~1	17~18	18~22	NP	26~35	>95	>95	>95	2.0~5.1	2.35~2.71	2.0~4.4
Muthiga	Limuru Trachyte	100	79	35	23	12	8	4	1	37	55	NP	20	>95	>95	>95	28.1~39.5	2.45~2.82	5.1
Kibera	Middle and Upper Kerichwo Valley Tuff	100	81~88	43~53	31~41	18~27	12~20	6~12	1~2	42~48	47~63	NP	10~20	<95	<95	<95	22.0~48.2	1.53~2.82	16.4~24.3
Max-inn	Nairobi Phonolite	100	67	27	15	7	4	2	0	21	22	NP	39	>95	>95	>95	2.9~3.1	2.43~2.65	2.9
Carnivore	Nairobi Trachyte	100	81	41	30	18	13	9	3	31	46	NP	22	>95	>95	>95	37.7~59.5	2.40~2.77	4.9
Existing DBT	Kapiti Phonolite	100	73	31	20	10	7	4	1	22	27	NP	34	>95	>95	>95	4.3~4.9	2.42~2.60	3.2
Existing Rungiri	Limuru Trachyte	100	80	33	21	11	8	5	3	34	45	NP	17	>95	>95	>95	21.7~30.8	2.40~2.68	4.6

Table 4.3.2 Test Result from Existing Hardstone Quarries
(availed from quarry sites)

QUARRY NAME	SAMPLE No.	ACV	LAA	FI	BITUMEN AFFINITY				
					80/100	MC 3000	KI-60	A 360	MC 5
H.Z & COMPANY	1	17	24	18	——	——	——	——	——
	2	19	12	15	——	——	——	——	——
	3	21	19	18	——	——	——	——	——
BHIMJI RAMJI	4	18	22.6	20	Good	Good	Good	Good	——
DIAMOND	5	15.9	18.8	16	Good	——	Good	Good	Good
	6	19	22	32	Good	——	Good	Good	Good

4.4 SAND

No sand is found in the vicinity of the projected alignment. Sand available in Nairobi is mostly supplied from Machakos. Consequently, sand must be obtained as purchased material.

For information on sand at Machakos see the Feasibility Study Report.

4.5 WATER SUPPLY

Water can be easily obtained from the Motone River, which the projected alignment runs along. To use this water, it is first necessary to secure an agreement with the holder of the water rights.

5. PAVEMENT STRUCTURE DESIGN

5.1 MAIN ROAD

The pavement was designed based mainly, on the Road Design Manual Part III and discussions with the Department of Material and Testing of the MOPW in the Preliminary Design stage in 1990. In 1991, a concrete plan was set for the construction of the Oil Pipeline from Nairobi to Kisumu/Eldoret. Use of the pipeline, by KPC, for oil will commence in 1993, before thee completion of Nairobi Bypass. The pavement design of the main road of the bypass took this into consideration..

5.1.1 TRAFFIC

A major factor in pavement design is the cumulative number of equivalent standard axles (ESA) in the design period. The cumulative number of equivalent standard axles during the design period was calculated from the forecast of future traffic in the feasibility study which was reviewed by the JICA study team and which was accepted by the Planning Department of the MOPW in the end of 1989.

(1) Cumulative number of equivalent standard axles during the design period

1) Initial daily number of equivalent standard axles (ESA) in the opening year 1997 :

The initial daily number of ESA in the opening year of the bypass were obtained from the number of medium goods vehicles (M.V.), heavy goods vehicles (H.V.), oil tankers (O.T. and buses (B) in Table VI-4-4 "Future Traffic Growth of the Bypass by Links" in the Feasibility Study Report. However, the number of oil-tankers classified in the heavy vehicles class category was estimated by referring to "MATERIALS BRANCH REPORT NO.333 AND NO. 455".

FUTURE TRAFFIC GROWTH OF THE BYPASS BY LINK

Link			LINK-1			LINK-2			LINK-3			LINK-4			
		pcu rate	AADT	AADT in pcu	AGR(%)	AADT	AADT in pcu	AGR(%)	AADT	AADT in pcu	AGR(%)	pcu rate	AADT	AADT in pcu	AGR(%)
1986	C/T	1.0	898	898	21	1,208	1,208	18	3,468	3,468	10	1.0	3,812	3,812	8
	L.V	1.0	414	414	24	934	934	19	1,675	1,675	11	1.0	1,846	1,846	10
	M.V	2.0	588	1,176	4	575	1,150	3	224	448	4	3.8	295	1,112	4
	H.V	2.0	491	982	6	465	930	6	374	748	7	3.8	412	1,583	5
	B	2.0	313	626	-1	172	344	-4	104	208	-9	3.8	112	429	-8
		pcu rate	AADT	AADT in pcu	CRV(%)	AADT	AADT in pcu	CRV(%)	AADT	AADT in pcu	CRV(%)	pcu rate	AADT	AADT in pcu	CRV(%)
2000	C/T	1.0	12,750	12,750	50	8,780	8,780	48	9,371	9,371	49	1.0	10,921	10,921	48
	L.V	1.0	8,272	8,272	32	6,547	6,547	33	5,220	5,220	32	1.0	6,588	6,588	29
	M.V	2.0	965	1,930	8	771	1,542	8	659	1,318	8	3.8	683	2,050	9
	H.V	2.0	1,066	2,131	8	999	1,998	9	1,027	2,054	11	3.8	1,101	3,138	13
	B	2.0	255	510	2	37	74	2	41	82	0	3.8	48	134	1
				25,593			18,941			18,045				22,831	

Note: C/T = Car, Taxi L.V. = Light Goods Vehicle M.V. = Medium Goods Vehicle H.V. = Heavy Goods Vehicle
B = Bus
AGR = Annual growth rate by vehicle type
CRV = Composit ratio by vehicle types

2) Ratio of Heavy Goods Vehicles to Oil Tankers

According to "Summary Chapter 16, page 13, Material Branch Report No. 333" and "Table 9.2.11 (a) Nairobi / Kisumu / Mal aba Roads (A104, B1, A1) 12 hr ADT. Daytime Number of Vehicles by class listed Time and Road Section", the ratio of heavy goods vehicles and oil-tankers is 2 : 1.

This ratio applies to the number of heavy vehicles including oil tankers in the heavy vehicles category.

3) Commercial vehicles distribution on the Bypass

The commercial vehicles distribution by section (link) is as follows;

AADT (Both Directions), 1986

Section	1	2	3	4
M.V.	588	575	224	295
H.V.	327	310	249	275
O.T.	164	155	249	275
B.	313	172	104	112

AADT (Both Directions), 2000

Section	1	2	3	4
M.V.	965	771	659	683
H.V.	711	666	658	734
O.T.	355	333	342	367
B.	255	37	41	48

4) Equivalence Factors of Vehicles

Referring to Table 9.2.1 (Mean Equivalence Factors, Material Branch Report No. 455), the Equivalence Factors on A104 near Nairobi City are as follows.

Vehicle	Equivalence Factor
M.V	0.9
H.V	8.57
O.T	12.8
Bus	0.45

5) The average daily number of Equivalent Standard Axles (ESA) in the base years 1986 and 2000

<u>Average daily number ESA in 1986</u>

EF		1		2		3		4	
		AADT	ESA	AADT	ESA	AADT	ESA	AADT	ESA
M.V.	0.9	588	529	575	517	224	201	295	265
H.V.	8.57	327	2802	310	2656	249	2133	275	2356
O.T.	12.8	164	2099	155	1984	125	1600	137	1753
B.	0.45	313	140	172	77	104	46	112	50
Total:			5570		5234		3980		4424

<u>Average daily number ESA in 2000</u>

EF		1		2		3		4	
		AADT	ESA	AADT	ESA	AADT	ESA	AADT	ESA
M.V.	0.9	965	868	771	694	659	593	683	615
H.V.	8.57	711	6090	666	5708	685	5870	734	6290
O.T.	12.8	355	4548	333	4262	342	4378	367	4698
B.	0.45	255	114	37	17	41	18	48	22
Total:			11620		10681		10859		11625

6) Annual growth rate of ESA (1986 - 2000)

$t_{2000} = t_{1986} \times (1+i)^{14}$

$i = (t_{2000}/t_{1986})^{1/14} - 1$ ...... Annual growth rate

Section	1	2	3	4	
	5570	5234	3980	44424	1986
	11620	10681	10859	11625	2000
i (%)	5	5	7	7	

7) Daily number of standard axles in the Bypass opening year, 1997.

$t_{1997} = t_{1986} \times (1 + i)^n$

where,

t = The average daily number of ESA in the base year (1988)

i = Annual growth rate

n = years (1997-1986)

Section	T_{1986}	i (%)	n	ESA/day in 1997
1	5570	5	11	9526
2	5234	5	11	8951
3	3980	7	11	8377
4	4424	7	11	9311

8) Numbers of oil-tankers which are equivalent to the volume of oil transport by pipeline:

The volume of oil transported by pipeline was planned by KPC is as follows;

1993 $1{,}080{,}000^{kl} / _{year}$

2008 $1{,}815{,}000^{kl} / _{year}$

Equivalent number of oil-tanker to the oil transport by pipeline is;

1993 $1{,}080{,}000^{kl} \div 20^{kl} / _{UNIT} = 54000^{kl}$ UNIT/year

2008 $1{,}815{,}000^{kl} \div 20^{kl} / _{UNIT} = 90750^{kl}$ UNIT/year

As: Capacity of an oil-tanker is 20 kl

Number of ESA (Oil-tankers)

1993	$54{,}000 \times 12.8 = 691{,}200/_{year}$
2008	$90{,}750 \times 12.8 = 2{,}161{,}600/_{year}$

Average daily number of ESA (Oil-tankers)

1993	$691{,}200 \div 365 = 1{,}893$
2008	$1{,}161{,}600 \div 365 = 3{,}182$

Decrease rate of ESA (1993 ~ 2008) of oil-tankers : i

$$i = (3{,}182/1{,}893)^{1/15}\text{-}1$$

$$= 3.5\%$$

Decrease number of ESA/day (Oil-tankers) in 1997 (Opening year of the Bypass)

$$t_{1997} = 1{,}893 \times (1+0.035)^4$$

$$= 2{,}172$$

Decrease number of ESA/day (Oil-tanker) in 2000

$$t_{2000} = 1{,}893 \times (1+0.035)^7$$

$$= 2{,}408$$

9) Cumulative number of Equivalent Standard Axles (ESA) during the design period (T).

Average daily number of ESA (t1)

Section	1997	2000
1	19,526	11,028
2	28,951	10,363
3	38,377	10,263
4	9,311	11,407

Cumulative number of Equivalent Standard Axles (ESA) during the design period (T) is obtained in accordance with the MOPW method as follows:

$$T = 365 \times t_1 \times \frac{(1+i)n - 1}{i}$$

where:

t_1 : The average daily number of standard axles in the first year.

i : The annual growth rate expressed as a decimal fraction

n : Design period: 4 years from 1997 to 2000

6 years from 2000 to 2007

Cumulative decrease number of Equivalent Standard Axles (ESA) of Oil-tankers during the design period (1997~2007)

1997 - 2000 (3 years)

$$T_1 = 365 \times 2,172 \times \frac{(1+0.035)^3 - 1}{0.035} = 2.4 \times 10^6$$

2000 - 2007 (7 years)

$$T_2 = 365 \times 2,408 \times \frac{(1+0.035)^7 - 1}{0.035} = 6.8 \times 10^6$$

Cumulative number (T) of ESA by section (Link)

1997 - 2000 (3 years)

Section	T_{1987}	i (%)	T	$(T/2-T_1)x0.8$
1	9,526	5	10,961,210	2.4×10^6
2	8,951	5	10,299,580	2.1×10^6
3	8,377	7	9,829,894	1.9×10^6
4	9,311	7	10,925,885	2.4×10^6

2000 - 2007 (7 years)

Section	T_{2000}	i (%)*	n	$(T/2-T_2)x0.8$
1	11,028	3	30,843,096	6.9×10^6
2	10,363	3	28,983,224	6.2×10^6
3	10,263	5	30,499,922	6.7×10^6
4	11,407	5	33,899,700	8.1×10^6

* : Growth rate is reduced after the year 2000 refering to Traffic Demand Forecast in the Feasibility Study Report. (Table VI-2-2)

Cumulative ESA (1997 - 2006)

Section :

1 $2.4 + 6.9 =$ 9.3×10^6

2 $2.1 + 6.2 =$ 8.3×10^6

3 $1.9 + 6.7 =$ 8.5×10^6

4 $2.4 + 8.1 =$ 10.5×10^6

5.1.2 PAVEMENT DESIGN

(1) The MOPW Method

- Traffic class: T2 ... Refer to the Road Design Manual, Part III
- Subgrade soil class ... S4, S6 (Refer to Fig. 5-1-2)
- Proposed Pavement Structure by subgrade condition are as follows;

Type	Subgrade	Pavement Structure	
Type 15	S4	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Lean concrete (200 mm)
		Subbase	: Graded crushed stone (175 mm)
	S5	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Lean concrete (200 mm)
		Subbase	: Graded crushed stone (150 mm)
	S6	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Lean concrete (200 mm)

Type	Subgrade	Pavement Structure	
Type 12	S4	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Dense bitumen macadam (125 mm)
		Subbase	: Graded crushed stone (175 mm)
	S5	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Dense bitumen macadam (125 mm)
		Subbase	: Graded crushed stone (125 mm)
	S6	Surface	: Asphalt concrete Type I, (50 mm)
		Base	: Lean concrete (125 mm)

(2) Road Note No. 29 Method

- Traffic: 10.5×10^6 ... Cumulative number of ESA (10 years)

- Proposed Pavement Structure by subbase condition are as follows:

Case 1

Subbase	Pavement Structure		
CBR < 30	Surfacing	:	Asphalt concrete (120 mm)
	Base	:	Lean concrete (200 mm)
	Subbase	:	Graded crushed stone (150 mm)
CBR > 30	Surfacing	:	Asphalt concrete (120 mm)
	Base	:	Lean concrete (200 mm)

* Note: Refer to Figs. 6 and 9 in Road Note No. 29

Case 2

Subbase	Pavement Structure		
CBR < 30	Surfacing	:	Asphalt concrete (100 mm)
	Base	:	Dense bitumen macadam (150 mm)
	Subbase	:	Graded crushed stone (150 mm)
CBR > 30	Surfacing	:	Asphalt concrete (100 mm)
	Base	:	Dense bitumen macadam (150 mm)

* Note: Refer to Figs. 6 and 8 in Road Note No. 29

(3) ASSHTO Method

- Traffic: 10.5×10^6 ... Cumulative number of ESA (10 years)

- Serviceability Index

$$Po = 4.2$$

$$Pt = 2.5$$

$$Psi = 4.2 - 2.5$$

$$= 1.7$$

- Required Structure Number (SN)

  Reliability (assumed)....................R = 95

  Overall Standard deviation.............So = 0.35

  Effective resilient modulus of roadbed material

  ....Mr = 15000

  Required Structure Number (SN) .....SN = 4.3

  (Refer to Fig. 3.1 in the AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES)

- Layer Thickness (Pavement Structure)

  $SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$

  ai : Layer coefficient

  D : Layer thickness

  mi : Drainage coefficient for base and subbase layers, respectively.

Layer	ai	mi
Surfacing	0.44	---
Base	0.25 (Lean concrete)	1.35
	0.40 (Dense bitumen macadam)	1.35
Subbase	0.14	1.35

Case 1 (Base is Lean concrete base)

$$SN = \frac{1}{2.54}(0.44 \times 10 + 0.25 \times 20 \times 1.35 + 0.14 \times 15 \times 1.35)$$

$$= 5.5 > 4.3$$

Case 2 (Base is Dense bitumen macadam)

$$SN = \frac{1}{2.54}(0.44 \times 10 + 0.4 \times 15 \times 1.35 + 0.14 \times 15 \times 1.35)$$

$$= 6.0 > 4.3$$

(4) JRA Method

1) Traffic (A) at five years after the opening of the Project road.

Traffic in the 2000

(Both directions) : 965 + 711 + 355 + 255 = 2,286/day Traffic (as 1997 is opening year) in 2002 : 2,286 × (1+i)2 = 2,452

Traffic increase rate : i = 0.03 (= 3%)

Reduction of oil tankers in the 2000 (see Decrease number of ESA/day (oil-tanker) in the year 2000)

T_{ot} = 2,408 ÷ 12.8

= 188 vehicles/day

∴ Heavy vehicle traffic in 2002 : t_{2002}(one direction)

t_{2002} = 2,452/2 - 188

= 1,038 vehicles/day .... C traffic

2) Pavement design

As:	Traffic class	CLASS C	
	CBR of subgrade	CBR	= 10
	Target value of the pavement	TA	= 25
	Target total depth of pavement		= 41 cm

Pavement structure

Case I

Surface (Asphalt concrete)	= 15 cm
Base (High qualitative cement stabilized material)	= 15 cm
Subbase (Graded crushed stone)	= 15 cm

$\therefore$ Actual T_A $= 15 \times 1.0 + 15 \times 0.55 + 15 \times 0.35$

$= 28.5 > 25$

Case II

Surface (Asphalt concrete)	= 10 cm
Base (High qualitative cement stabilized material)	= 15 cm
Subbase (Graded crushed stone)	= 15 cm

$\therefore$ Actual T_A $= 10 \times 1.0 + 15 \times 0.8 + 15 \times 0.35$

$= 27.0 > 25$

(5) Summary of Pavement Structures Using Various Design Methods

Unit: mm

Design Method	Material	MOPW	RN-NO.29	AASHOTO	JRA
Case I.					
CBR < 30					
Surfacing	Asphalt concrete	50	120	100	150
Base	Lean concrete	200	200	200	150
Subbase	Graded crushed stone	175 (150)	150	150	150
CBR > 30					
Surfacing	Asphalt concrete	50	120	100	150
Base	Lean concrete	200	200	200	200
Subbase		—	—	—	—
Case II.					
CBR < 30					
Surfacing	Asphalt concrete	50	100	100	100
Base	Dense bitumen macadam	125	150	150	150
Subbase	Graded crushed stone	175 (150)	150	150	150
CBR > 30					
Surfacing	Asphalt concrete	50	100	100	100
Base	Dense bitumen macadam	125	150	150	150
Subbase		—	—	—	—

5.1.3 COMPARISON OF PAVEMENT STRUCTURE

The pavement structure was designed in accordance with the results of the Feasibility Study and in close consultation with the relevant engineers of MOPW refering to MOPW's DESIGN MANUAL Part III, ROAD NOTE NO.29, the AASHTO Guide for Pavement Structures and the DESIGN MANUAL FOR ROAD PAVEMENT by the Japan Road Association.

The pavement structures, especially the types of base were studied and compared. The results of the comparison as follows.

Case A: Lean concrete base(A kind of cement stabilized material).

Case B: Dense bitumen macadam base.

	Case A	Case B
Pavement structure		
Surfacing (mm)	120	100
Base (mm)	200	150
Subbase (mm)	150	150
Base Materials	Lean concrete – cement – aggregate	Dense bitumen macadam – bitumen – aggregate
Availability of Base Materials	Cement : local aggregate : local	bitumen: imported aggregate : local
Construction		
Mixing	Stationary Plant	Central Plant Method
Laying	Paver	Paver
Curing	Necessary	Not necessary
Cost (Price level: Jun 1990)	Kshs 529.8	Kshs 667.6
Foreign currency portion	small	big
Effect by oxygen	yes	yes

It is recommended to adopt case A in view of availability local material and saving foreign currency.

Refering to "Main conclusion from the asphaltic pavement investigations and documentation study for pavement rehabilitation Jomo Kenyatta International Airport Nairobi", it said that:-

— Asphaltic pavement was excessively hardened by the physical and chemical nature of the Nairobi phnolite stones and of the airblown bitumen from the Kenya Petroleum Refinery.