## 7.6 Construction Materials

### 7.6.1 Test Quantities and Test Items

The locations where samples for laboratory tests were collected with regard to borrow materials and concrete aggregates required for the Upper Kihansi Project and the Lower Kihansi Project are shown in Fig. 7-15. The quantities of testing and items of test related to the samples collected are given in Table 7-6.

As shown in Fig. 7-15, laboratory test samples were collected from 3 points on the surface, namely the surface of Test Pit P-1 in the vicinity of the Upper Kihansi dam site, the Upper Kihansi Project Site and the Lower Kihansi Project Site with regard to borrow materials, and from Test Pits P-2 to P-4, and Q-1 and Q-2 already sampled for other work, with regard to concrete aggregates.

The laboratory tests mentioned here except above 3 samples collected from the surface (the laboratory tests are performed in Japan) were all performed at the University of Dar Es Salaam.

### 7.6.2 Test Results and Considerations

(1) Borrow Materials

As shown in Table 7-7 and Figs. 7-16 (1), (2) and 7-17, the samples on which soil tests were performed had high contents of clay and silt on the whole. Compared with borrow materials that have been used in rockfill dams of heights about 100 m in Japan and elsewhere in the world, these samples are too high in content of fine-grained materials such as clay and silt, while the densities are also low.

Judging from the test results, it is expected that these samples will lack sufficient strength for use in a high dam. However, it will be possible to use them if mixed with other coarser materials.

The alluvial deposits occurring in small quantity on Kihansi River (Samples No. L-1 and L-2) mainly consists of fine sand of similar grain sizes and main minerals composing the sand are quartzo and feldspar. The quantity of the deposits is not sufficient to use for construction material of fill-type dam. These test results should be used for basic data concerning the sedimentation in the reservoir and abrasion of the turbine.

(2) Concrete Aggregates

The test results are shown in Table 7-7 and Fig. 7-18.

The samples collected from Test Pits P-2, P-3 and P-4 are judged to be unsuitable for application as concrete aggregates as a result of grain-size analyses, clay lumps tests, and alkali-aggregate reaction tests, and from the fact that quantities available are small. On the other hand, according to the results of tests on samples from existing quarry sites and boring cores, it is judged that fresh rock distributed in this area can generally be used as concrete aggregates.

(3) Considerations

With regard to borrow materials, it is considered most suitable for the residual soil and weathered rock widely distributed at the ground surface in the vicinity of the Upper Kihansi dam site to be used mixed with coarse material.

With regard to concrete aggregates, there are no sediments to be seen in this area which are available in large quantities. As a result of testing, the gneisses widely distributed as basement rocks in this area can be used as concrete aggregates from the standpoints of specific gravity, absorption, alkali-aggregate reaction, and further, strength. Consequently, it is thought the best method for this Project is to select a quarry at the optimum location and obtain concrete aggregates in the form of crushed stone.

The location of the quarry site selected taking into account geological conditions and the locations of the principal structures of the Upper and Lower Kihansi projects is shown in Fig. 7-15.

Furthermore, it will be necessary hereafter to ascertain at the quarry site the quantity and properties of the weathered layer that can be furnished as borrow material, and to investigate the distribution of fresh rock that can be considered for blending with borrow material and that can be used as concrete aggregates. Particularly, with regard to fresh rock, confirmation of rock type will be an important point in investigations since in the case of gneiss containing large amounts of biotite, "abrasion loss" will be high and it is conceivable that securing the necessary quantity of coarse aggregates may be difficult.

## Table 7-6 Quantity and Items of Laboratory Test for Construction Materials

[CORE MATERIAL 1]

| Sample No. | Pit No. in Depth(m) | Specific Gravity | Grain-size Analysis | Liquid Limit | Plastic Limit | Compaction | Permea-bility |
|---|---|---|---|---|---|---|---|
| KB-1a | P-1, 0.5 | - | 1 | 1 | 1 | - | - |
| KB-1b | 〃, 1.0 | - | 1 | 1 | 1 | - | - |
| KB-1c | 〃, 1.5 | - | 1 | 1 | 1 | - | - |
| KB-1d | 〃, 2.0 | - | 1 | 1 | 1 | - | - |
| KB-1e | 〃, Mixed | 1 | 1 | 1 | 1 | 1 | 1 |
| Total | | 1 | 5 | 5 | 5 | 1 | 1 |

[CORE MATERIAL 2]

| Sample No. | Sampling Location | Specific Gravity | Water Content | Grain-size Analysis | X-ray Analysis | Mineral Composition Analysis |
|---|---|---|---|---|---|---|
| L-1 | Lower Dam Site(River Deposit) | 1 | 1 | 1 | 1 | 1 |
| L-2 | Lower P/S Site( 〃 ) | 1 | 1 | 1 | 1 | 1 |
| U-1 | Upper Dam Site(Residual Soil) | 1 | 1 | 1 | 1 | - |
| Total | | 3 | 3 | 3 | 3 | 2 |

[CONCRETE AGGREGATE (Fine Aggregate)]

| Sample No. | Pit No. in Depth(m) | Specific Gravity | Absorp-tion | Unit Weight | Grain-size Analysis | Organic Impurities | Clay Lumps | Soundness | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|---|---|---|
| KQ-2 | P-2, 1.6 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| KQ-3 | P-3, 1.1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| KQ-4 | P-4, 1.2 | 1 | 1 | 1 | 1 | - | 1 | 1 | 1 |
| KQ-5 | (Q-1 Site) | 1 | 1 | 1 | 1 | - | 1 | 1 | 1 |
| Total | | 4 | 4 | 4 | 4 | 2 | 4 | 4 | 4 |

[CONCRETE AGGREGATE (Crashed Coarse Aggregate)]

| Sample No. | Location | Specific Gravity | Absorp-tion | Unit Weight | Clay Lumps | Soundness | Abrasion Loss | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|---|---|
| KQ-6b | Q-2 Site | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| Total | | 1 | 1 | 1 | 1 | 1 | 1 | 1 |

[CRASHED AGGREGETE (Drilled Core Test)]

| Sample No. | Hole No. in Depth (m) | Specific Gravity | Absorption | Unconfined Compression | Tensile Splitting | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|
| KQ- 7 | KU-1 33.2-33.5 | 1 | 1 | 1 | 1 | 1 |
| KQ- 8 | 〃 35.4-35.6 | 1 | 1 | 1 | - | 1 |
| 〃 | 〃 35.7-35.9 | 1 | 1 | 1 | - | 1 |
| KQ- 9 | KU-3 30.5-30.8 | 1 | 1 | 1 | 1 | 1 |
| KQ-10 | 〃 36.7-37.0 | 1 | 1 | 1 | 1 | 1 |
| KQ-11 | KL-1 5.5- 5.8 | 1 | 1 | 1 | - | 1 |
| KQ-12 | 〃 15.0-15.3 | 1 | 1 | 1 | 1 | 1 |
| KQ-13 | 〃 17.5-18.0 | 1 | 1 | 1 | 1 | 1 |
| KQ-14 | KL-2 9.7-10.0 | 1 | 1 | 1 | 1 | 1 |
| KQ-15 | 〃 15.5-16.0 | 1 | 1 | 1 | 1 | 1 |
| KQ-16 | KL-5 6.8- 7.0 | 1 | 1 | 1 | 1 | 1 |
| 〃 | 〃 7.5- 7.8 | 1 | 1 | 1 | 1 | 1 |
| KQ-17 | 〃 11.5-11.8 | 1 | 1 | 1 | 1 | 1 |
| KQ-18 | 〃 18.5-19.0 | 1 | 1 | 1 | - | 1 |
| KQ-19 | KM-1 18.3-18.7 | 1 | 1 | 1 | 1 | 1 |
| KQ-20 | KM-2 17.2-17.4 | 1 | 1 | 1 | - | 1 |
| KQ-6a | (Site Q-2) | 1 | 1 | 1 | 1 | 1 |
| Total | | 17 | 17 | 17 | 12 | 17 |

## Table 7-7 (1) Results of Laboratory Test for Construction Materials

[CORE MATERIAL 1]

| Sample No. | Pit No. in Depth(m) | Specific Gravity | Grain-size Analysis | Liquid Limit | Plastic Limit | Compaction | Permea-bility |
|---|---|---|---|---|---|---|---|
| KB-1a | P-1, 0.5 | - | Shown in Fig. 7-16 | Slightly plastic | | - | - |
| KB-1b | 〃, 1.0 | - | | 〃 | | - | - |
| KB-1c | 〃, 1.5 | - | | 〃 | | - | - |
| KB-1d | 〃, 2.0 | - | | 〃 | | - | - |
| KB-1e | 〃, Mixed | 2.65 | | 〃 | | Shown in Fig. 7-17 | |

BS or ASTM

[CORE MATERIAL 2]

| Sample No. | | L-1 | L-2 | U-1 |
|---|---|---|---|---|
| Sampling Location | | Lower Dam Site (River Deposit) | Lower P/S Site (River Deposit) | Upper Dam Site (Residual Soil) |
| Specific Gravity 1) | | 2.65 | 2.67 | 2.77 |
| Water Content(%) 2) | | 32.6 | 5.5 | 24.2 |
| Grain-size Analysis 3) | | Shown in Fig. 7-16 | | |
| X-ray Analysis | Quartz | B | A | B |
| | Plagioclase | D | D | - |
| | Orthoclase | E | E | - |
| | Muscovite | E | E | - |
| | Biotite | E | E | E |
| | Magnetite | D | E | E |
| | Chlorite | E | E | E |
| | Chamosite | - | - | E |
| | Kaolinite | - | - | D |
| Mineral Composition Analysis | Quartz (%) | 62.0 | 55.3 | —— |
| | Feldspar (%) | 16.0 | 12.7 | |
| | Mica (%) | 13.0 | 13.6 | |
| | Amphibole (%) | 4.0 | 7.7 | |
| | Garnet (%) | 3.0 | 3.9 | |
| | Iron Mineral (%) | 2.0 | 2.9 | |
| | Clay Mineral (%) | 0.0 | 3.9 | |

X-ray Analysis ; A:Strong B:Medium C:Weak D:Very Weak E:Trace

1) JIS A 1202
2) JIS A 1203
3) JIS A 1204

## Table 7-7 (2) Results of Laboratory Test for Construction Materials

[CONCRETE AGGREGATE(Fine Aggregate)]

| Sample No. | Pit No. in Depth(m) | | Specific Gravity | Absorption (%) | Unit Weight (g/ cm³) | Grain-size Analysis | Organic Impurities | Clay Lumps (%) | Soundness (% Wt loss) | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|---|---|---|---|
| KQ-2 | P-2, 1.6 | | 2.55 | 0.32 | 1.51 | Shown in Fig. 7-18 | Not-Present | 6.4 | 4.8 | Limited Acceptable |
| KQ-3 | P-3, 1.1 | | 2.61 | 0.52 | 1.54 | | 〃 | 5.6 | 5.2 | Harmful |
| KQ-4 | P-4, 1.2 | | 2.51 | 0.29 | 1.34 | | - | 3.6 | 8.2 | 〃 |
| KQ-5 | (Q-1 Site) | | 2.66 | 2.4 | 1.51 | | - | 1.6 | 3.9 | Limited Acceptable |
| Maximum Allowance | | ASTM | | | | | | less than 3.0 | less than 10.0 | |
| | | JIS | | | | | | less than 1.0 | 〃 | |

[CONCRETE AGGREGATE(Crashed Coarse Aggregate)]

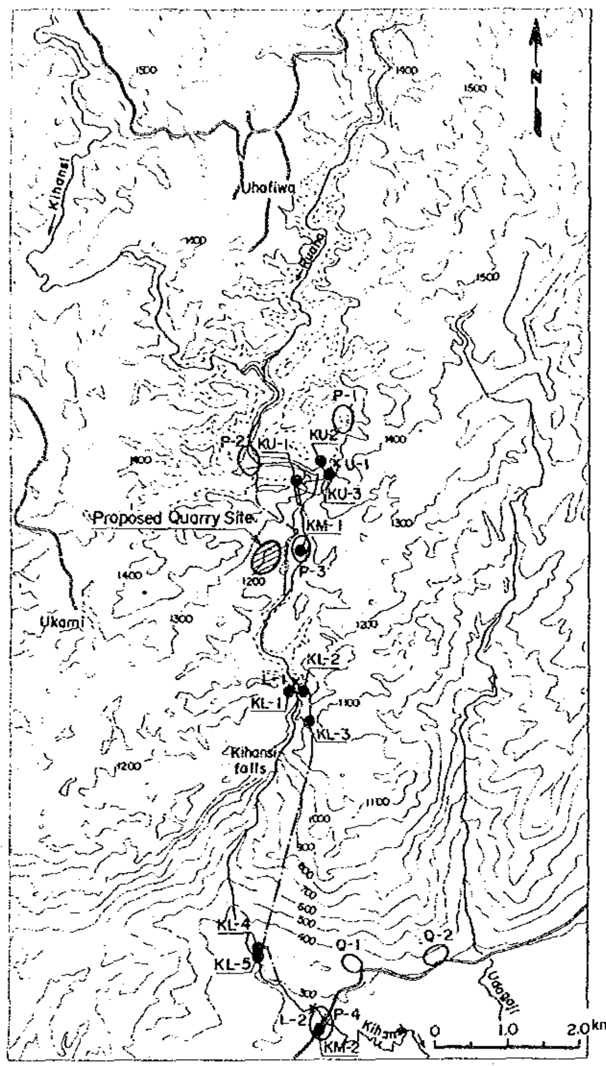
| Sample No. | Location | Specific Gravity | Absorption (%) | Unit Weight (g/ cm³) | Clay Lumps (%) | Soundness (% Wt less) | Abrasion Loss (%) | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|---|---|
| KQ-6b | Q-2 Site | 2.42 | 0.68 | 1.59 | 0.1 | 2.4 | 58 | Innocuous |
| Maximum Allowance | ASTM | | | | less than 2.0 | less than 18.0 | less than 50.0 | |
| | JIS | more than 2.5 | less than 3.0 | | less than 0.25 | less than 12.0 | less than 40.0 | |

[CRASHED AGGREGATE (Drilled Core Test)]

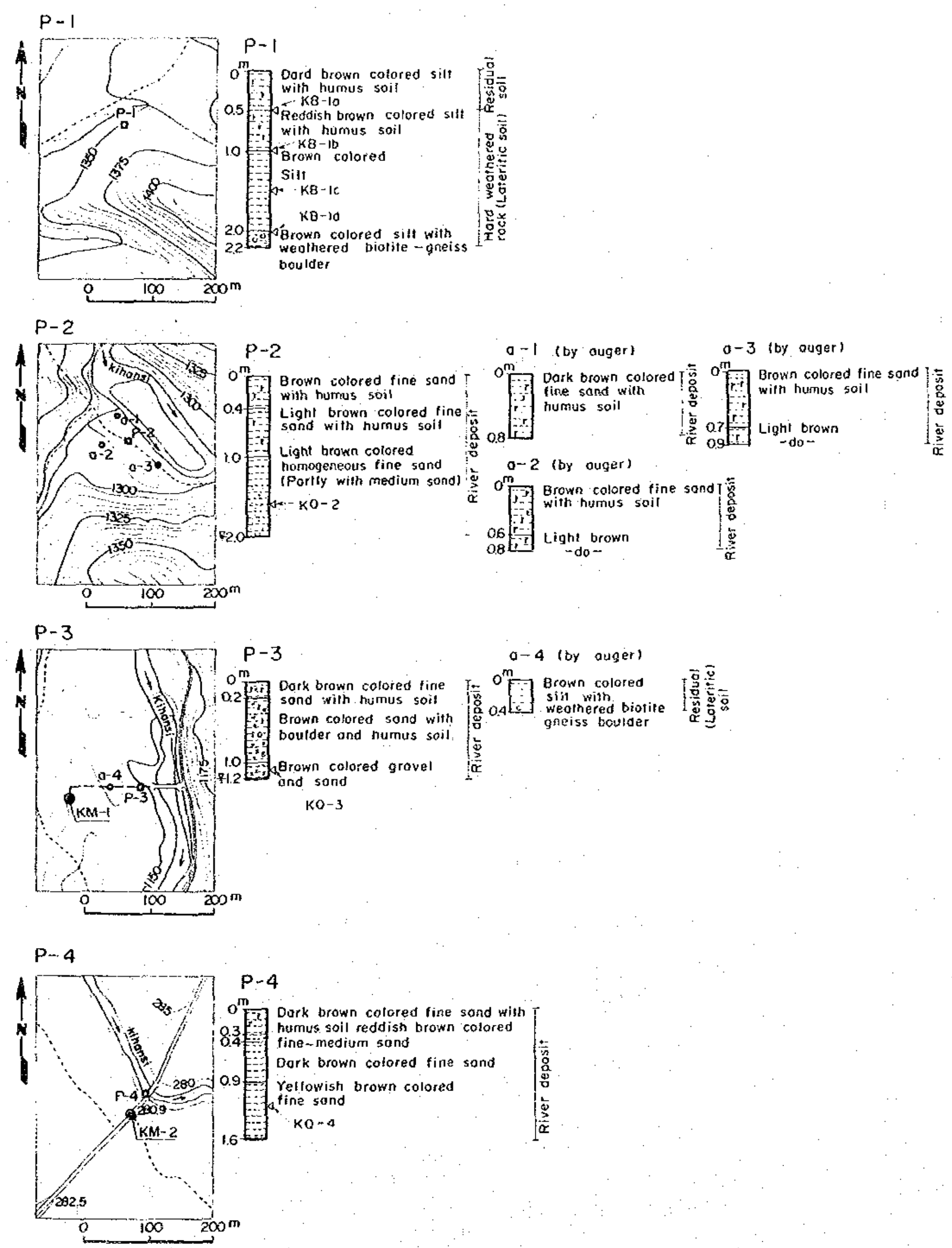
| Sample No. | Hole No. in Depth (m) | Specific Gravity | Absorption (%) | Unconfined Compression (kgf/cm²) | Tensile Splitting (kgf/ cm²) | Alkali-aggregate Reaction |
|---|---|---|---|---|---|---|
| KQ- 7 | KU-1 33.2-33.5 | 2.1 | 2.8 | 110 | 10 | Innocuous |
| KQ- 8 | 〃 35.4-35.6 | 2.3 | 0.4 | 701 | - | 〃 |
| 〃 | 〃 35.7-35.9 | 2.2 | 0.6 | 799 | - | Limited Acceptable |
| KQ- 9 | KU-3 30.5-30.8 | 2.4 | 0.6 | 571 | 25 | Innocuous |
| KQ-10 | 〃 36.7-37.0 | 2.1 | 0.6 | 784 | 40 | 〃 |
| KQ-11 | KL-1 5.5- 5.8 | 2.2 | 3.9 | 80 | - | 〃 |
| KQ-12 | 〃 15.0-15.3 | 2.3 | 1.2 | 154 | 17 | 〃 |
| KQ-13 | 〃 17.5-18.0 | 2.9 | 0.03 | 2335 | 105 | 〃 |
| KQ-14 | KL-2 9.7-10.0 | 2.1 | 2.6 | 138 | 15 | Limited Acceptable |
| KQ-15 | 〃 15.5-16.0 | 2.9 | 0.03 | 1844 | 152 | Innocuous |
| KQ-16 | KL-5 6.8- 7.0 | 2.3 | 0.6 | 424 | 32 | 〃 |
| 〃 | 〃 7.5- 7.8 | 2.3 | 0.5 | 543 | 23 | 〃 |
| KQ-17 | 〃 11.5-11.8 | 3.0 | 1.3 | 108 | 22 | 〃 |
| KQ-18 | 〃 18.5-19.0 | 2.3 | 0.5 | 559 | - | 〃 |
| KQ-19 | KM-1 18.3-18.7 | 2.3 | 0.3 | 1109 | 68 | 〃 |
| KQ-20 | KM-2 17.2-17.4 | 2.5 | 0.3 | 252 | - | 〃 |
| KQ-6a | (Site Q-2) | 2.4 | 0.3 | 807 | 31 | 〃 |

(Note) Alkali-aggregate Reaction : TZS (Tanzanian Standard) 58
Other Test Items : BS or ASTM

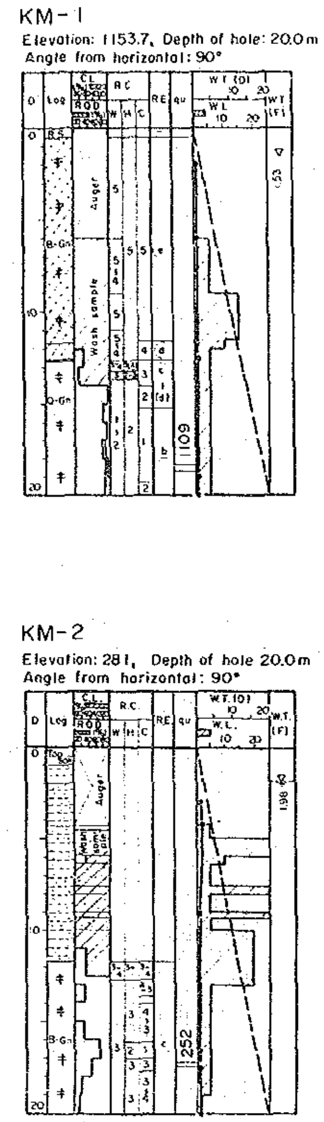
## LOCATION MAP OF QUARRY SITES




## LOCATION MAP OF TEST PITS AND GEOLOGICAL LOGS




## GEOLOGICAL LOGS OF DRILLHOLE



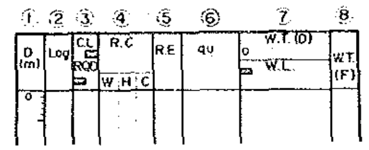

LEGEND

- Location of Test Pit
- Location of Auger Boring
- Location of Drill Hole

Geological Log of Test Pit and Auger Boring

- Silt ~ Fine Sand with Humus Soil
- Silt ~ Fine Sand
- Medium Sand
- Gravel
- Sampling Depth for Laboratory Test (Sample No.: KB- and KO- )
- Sampling Point for Laboratory Test (Sample No.: L-1, L-2, U-1)

DRILLING LOGS



1. Depth of Drillhole (m)
2. Geological Log
3. R.Q.D = Rock Quality Designation (%)
   C.L. = Core Loss (%)
4. R.C. = Rock Classification
   W: Weathering 1 (Very Fresh) ~ 5 (Strongly Weathered)
   H: Hardness 1 (Very Hard) ~ 5 (Soft)
   C: Interval of Cracks 1 (Over 30cm) ~ 5 (Under 1cm)
5. R.E. = Rock Evaluation
   Very Good ~ Very Bad
6. qu = Unconfined Compression Strength of Core (kgf/cm²)
7. W.T.(D) = Water Table in Drillhole during Drilling
   W.L. = Water Leakage during Drilling (ℓ/min)
8. W.T.(F) = Final Water Table (m)

| KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT | |
|---|---|
| LOCATION AND GEOLOGICAL LOG OF CONSTRUCTION MATERIAL SITE | |
| Fig. 7-15 | DATE; |

Fig. 7-16 (1) Grain-size Analysis of Core Material (KB-1a, -b, -1c, -1d)

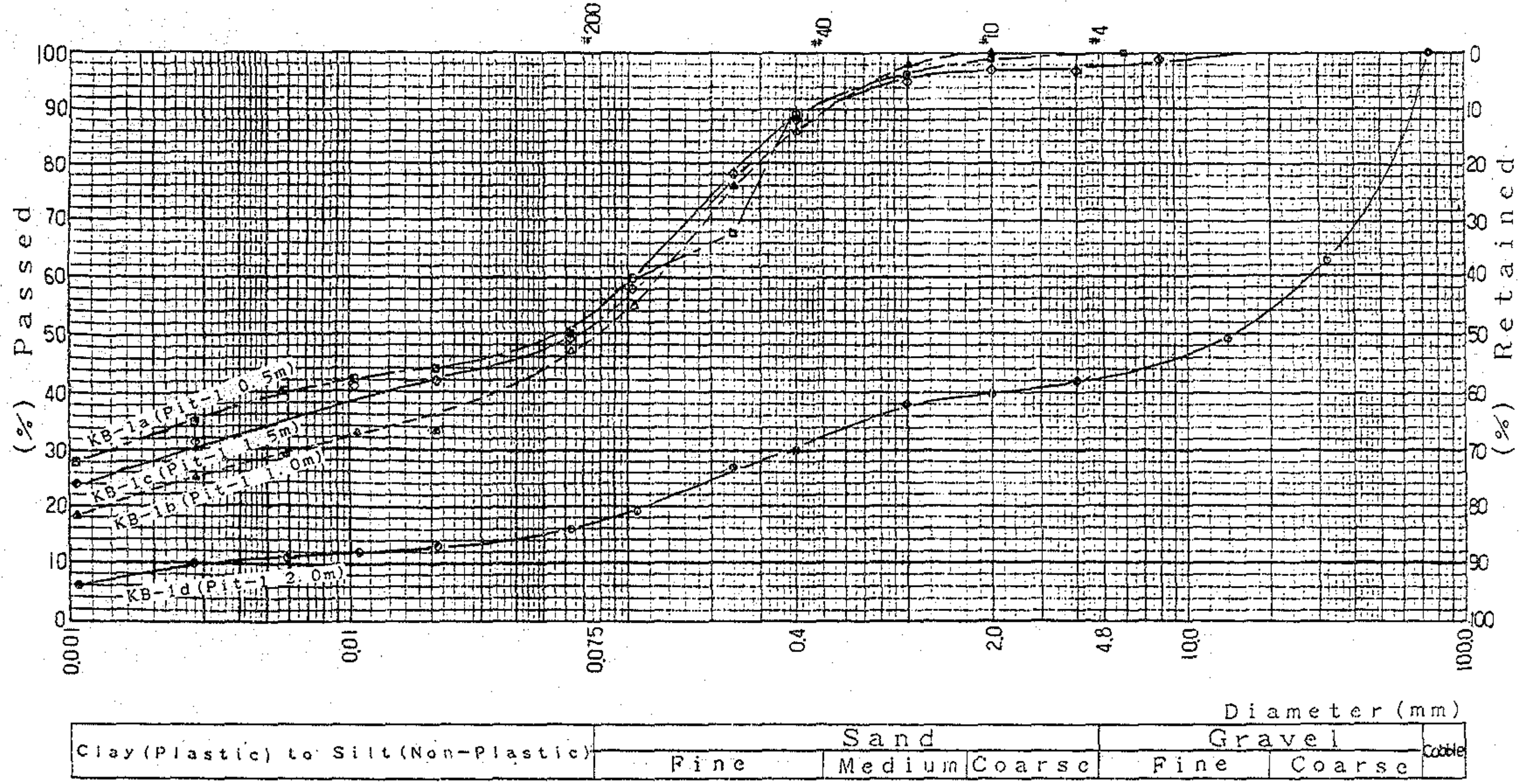


Fig. 7-16 (2) Grain-size Analysis of Core Material (KB-1e, L-1, L-2, U-1)

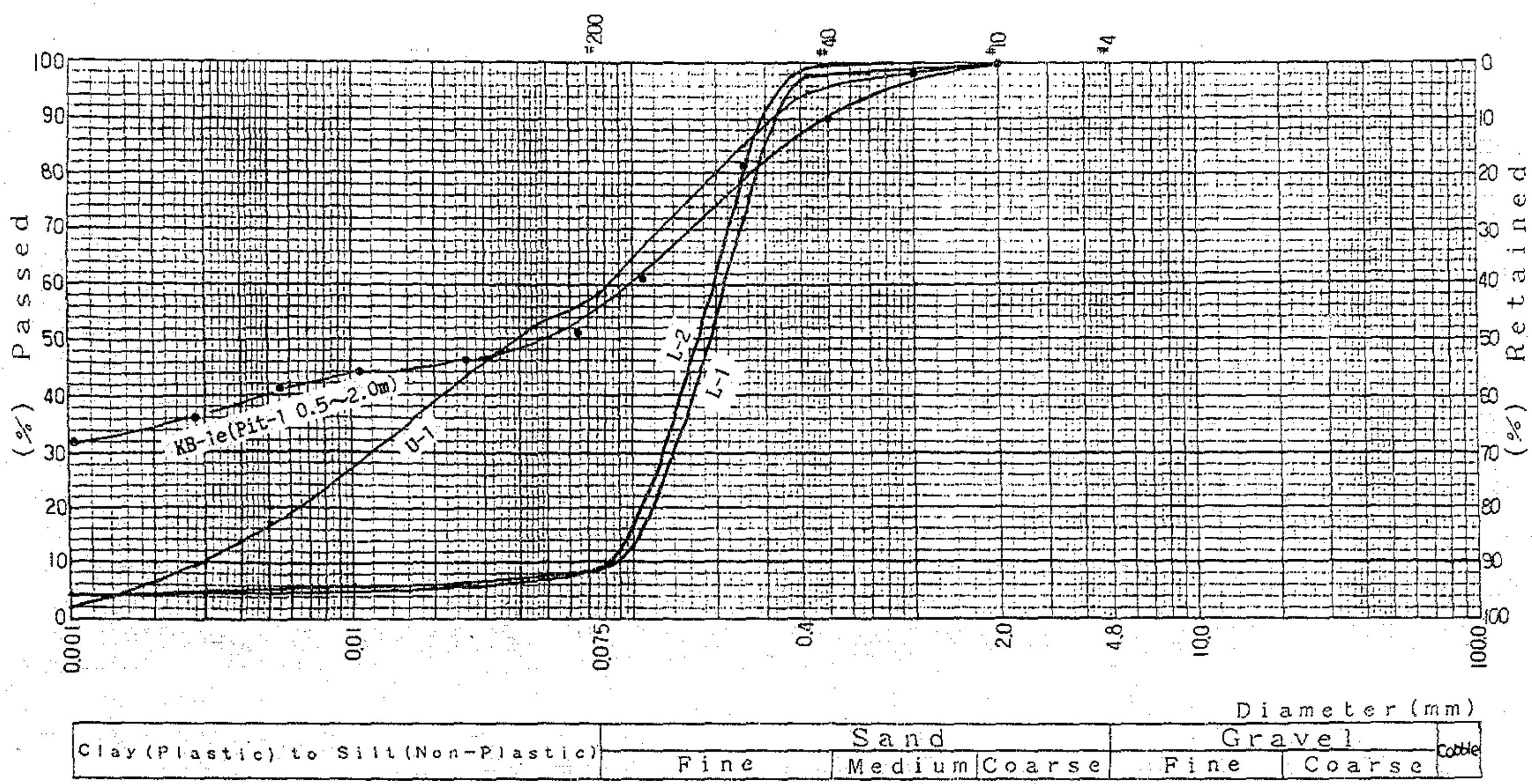


Fig. 7-17 Procter Test of Core Material (KB-1e)

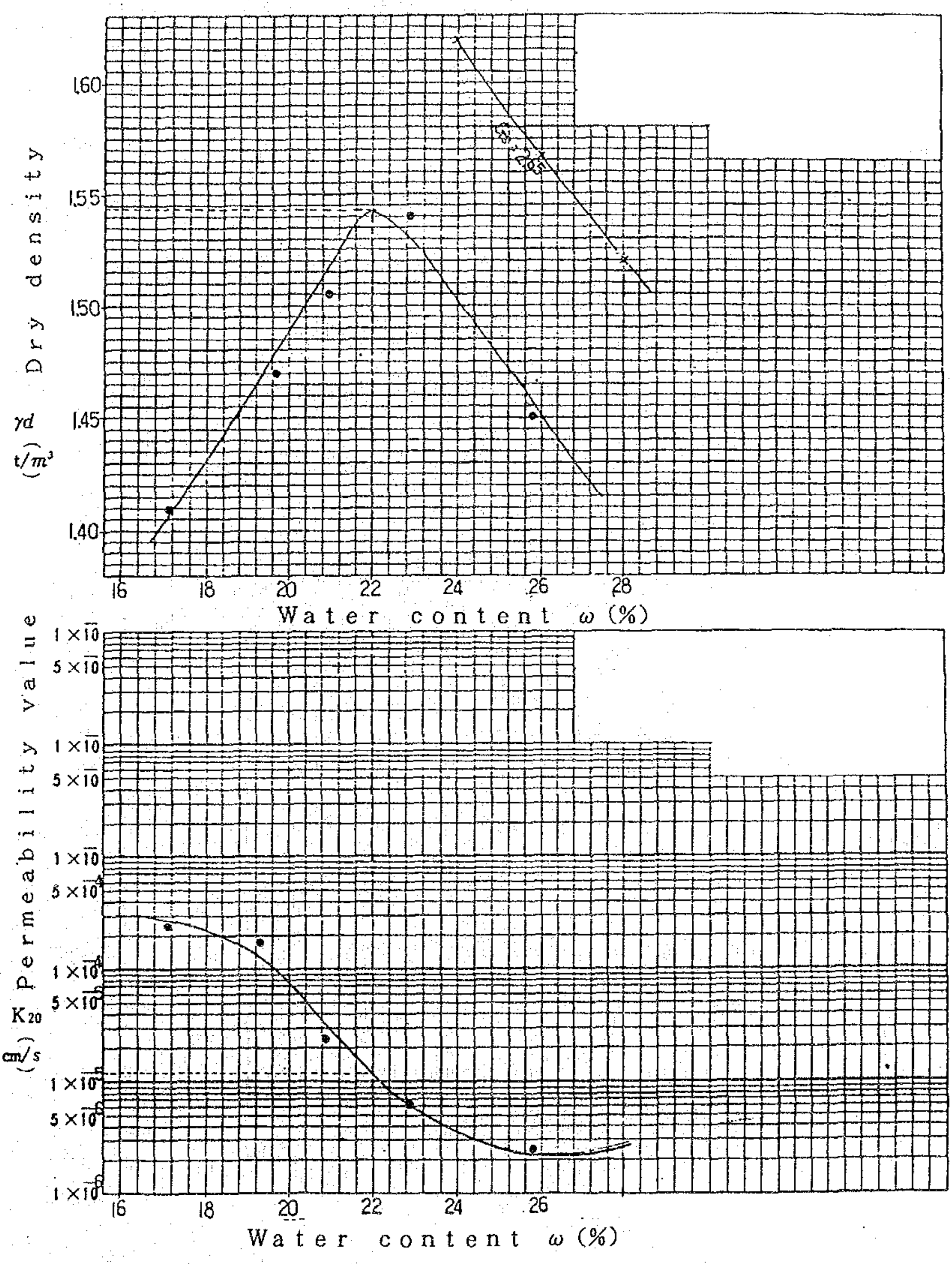


Fig. 7-18 (1) Grain-size Analysis of Concrete Aggregate (KQ-2, -3)

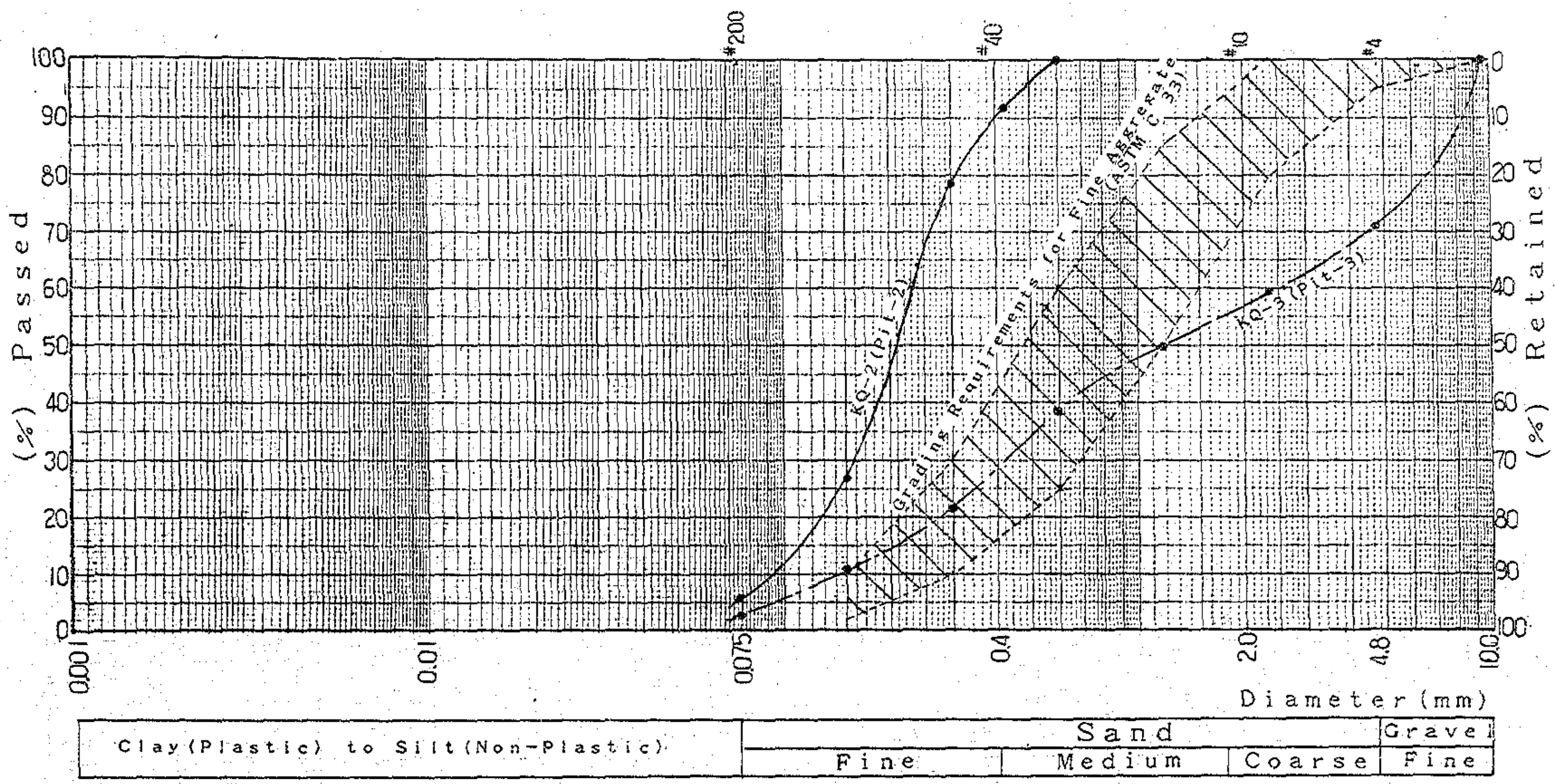
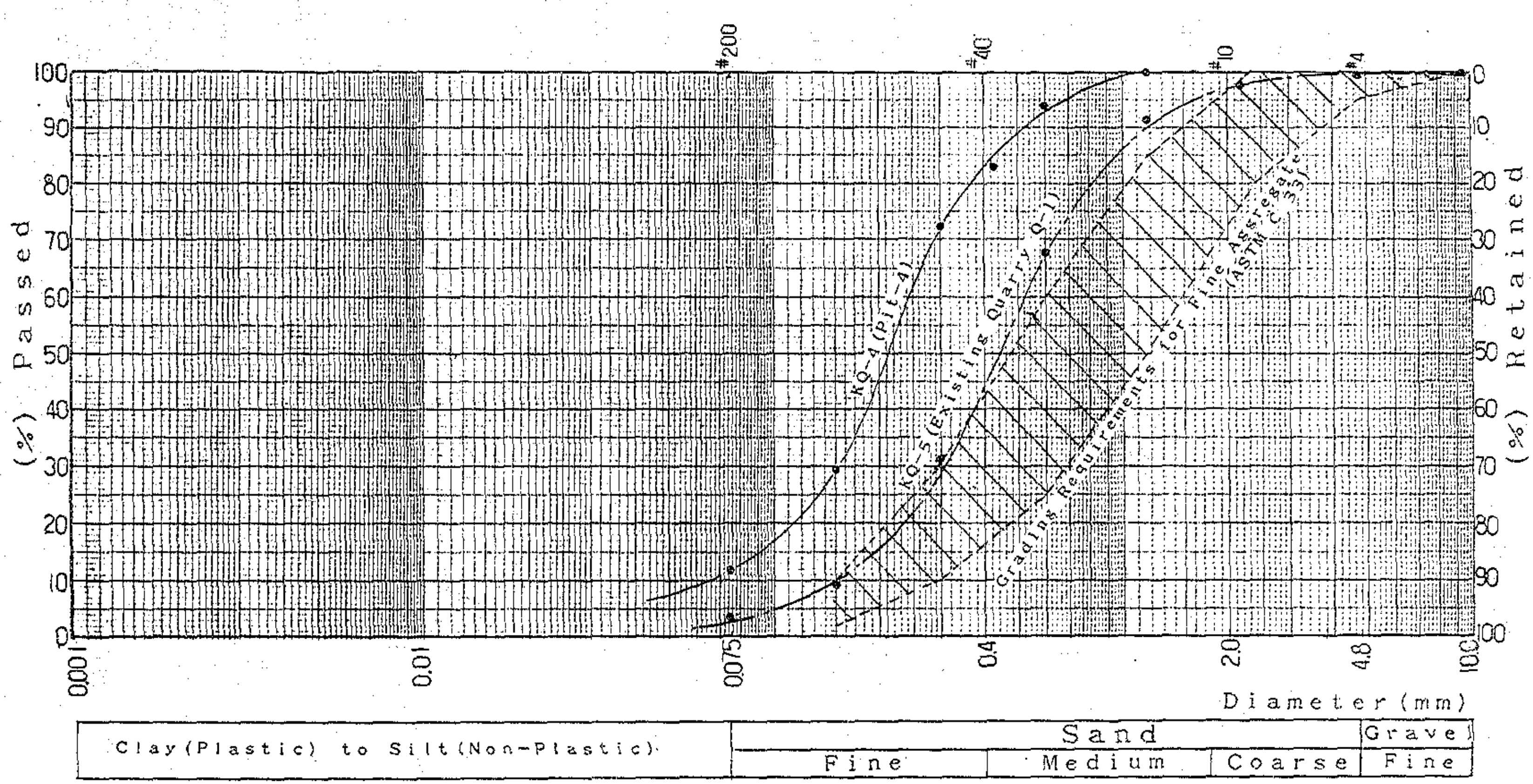


Fig. 7-18 (2) Grain-size Analysis of Concrete Aggregate (KQ-4, -5)



## 7.7 Seismicity

### 7.7.1 Outline of Seismicity in Tanzania

(1) Background of Earthquake Occurrence

The East African Rift Valleys comprise a topographic troughs and grabens system 4,000 km in length running north-south in the eastern part of Africa starting from Egypt and going through Sudan and Kenya to reach Tanzania.

It has been found that these rift valleys comprise the boundary of plates caused by separation of the Nubia Plate and the Somalia Plate.

A part of the abovementioned East African Rift Valleys passes through Tanzania. In detail, as shown in Fig. 7-3, the rift valleys may be divided into the Eastern Rift running at the west side of Arusha to Dodoma, and the Western Rift continuing from Lake Tanganyika to Lake Malawi.

It is considered that regarding occurrences of earthquakes in Tanzania separation of plates at the abovementioned Eastern Rift and Western Rift is an extremely important matter.

(2) Seismicity

The epicenters of 349 earthquakes which occurred during the 78-year period from 1910 to 1987 in a 1,000-km radius circle with the project site at the center are shown in Fig. 7-19. The epicenters of the earthquakes during this period grouped according to magnitudes are shown in Fig. 7-20.

As is clearly seen in these figures, seismicity is prominent along the Western Rift passing through the vicinity of Mbeya, while along the Eastern Rift, it is considered to be less than half in comparison.

It is thought that epicenters indicating prominent seismicity have been extremely rare in the vicinity of the project area.

Fig. 7-19 Seismicity Around the Project Site

during 1910 - 1987, and Δ < 1,000 km

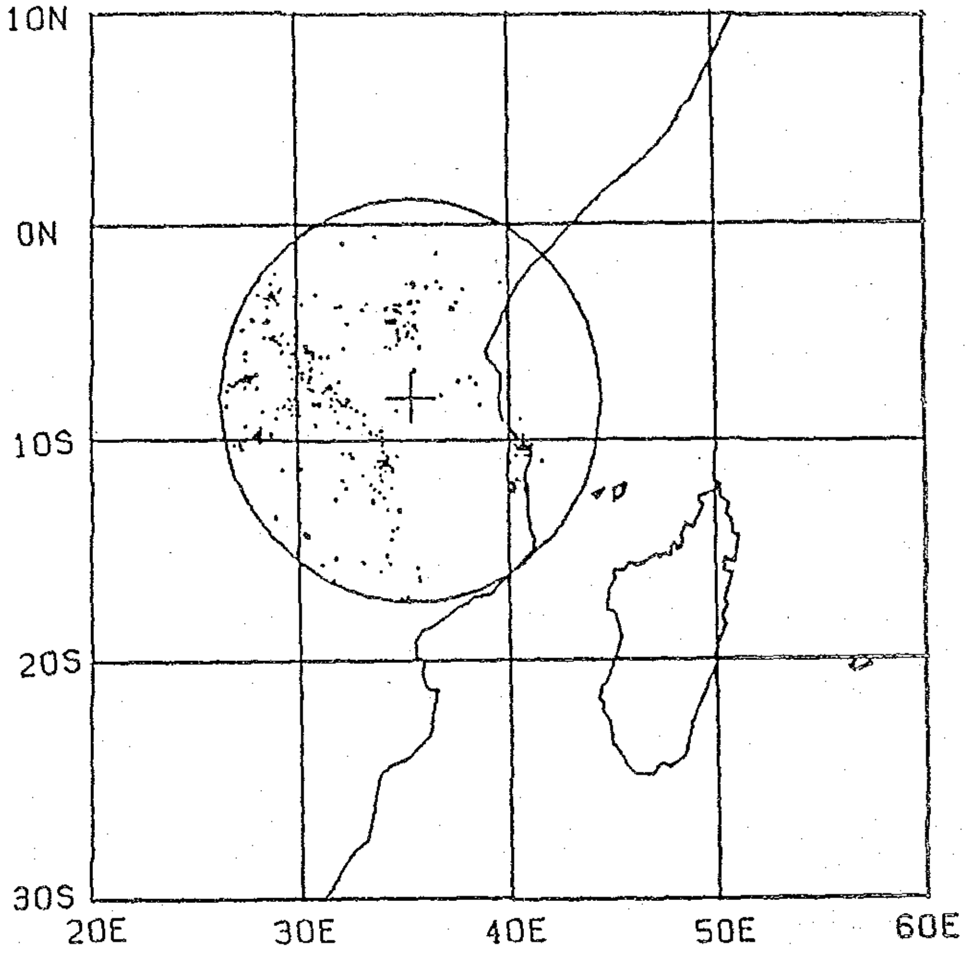


Fig. 7-20 (1) Locations of Magnitudes (3 ≤ M < 4 during 1910 - 1987)

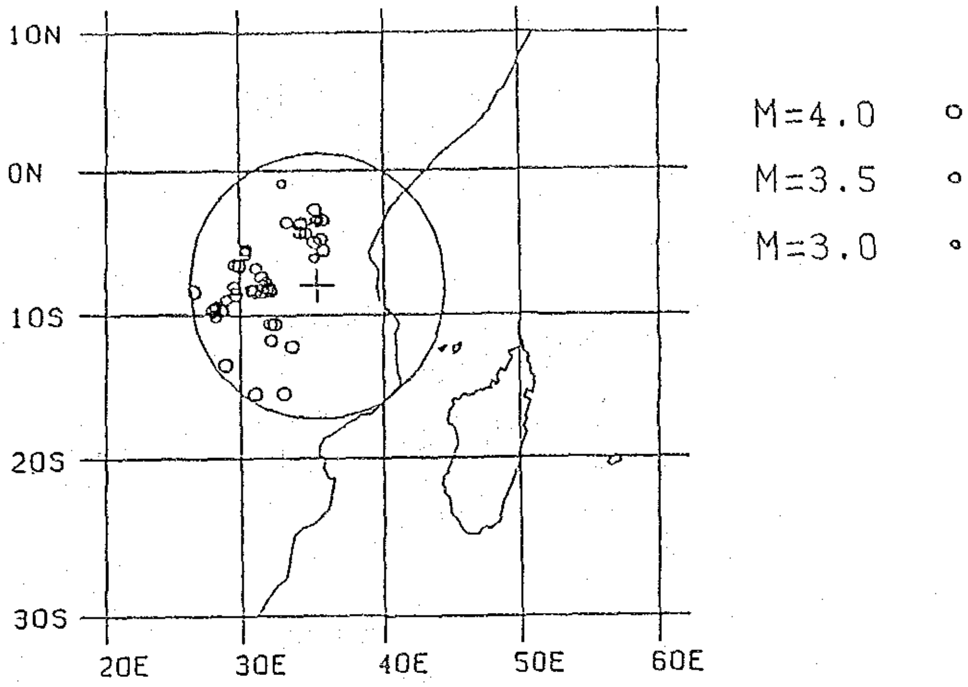



Fig. 7-20 (2) Locations of Magnitudes (4 ≤ M < 5 during 1910 - 1987)

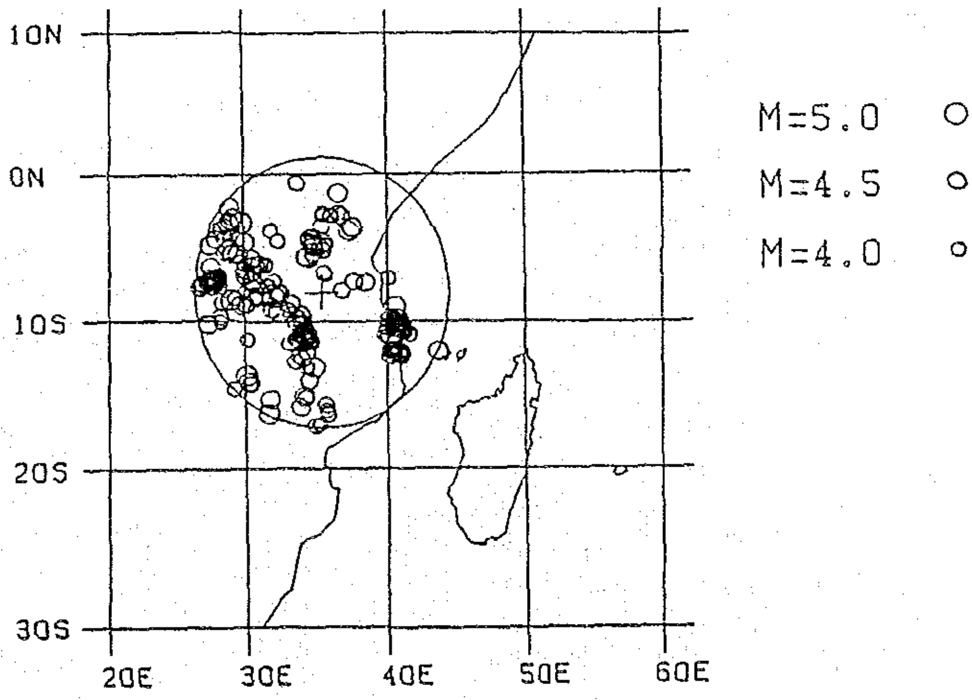


Fig. 7-20 (3) Locations of Magnitudes (5 ≤ M < 6 during 1910 - 1987)

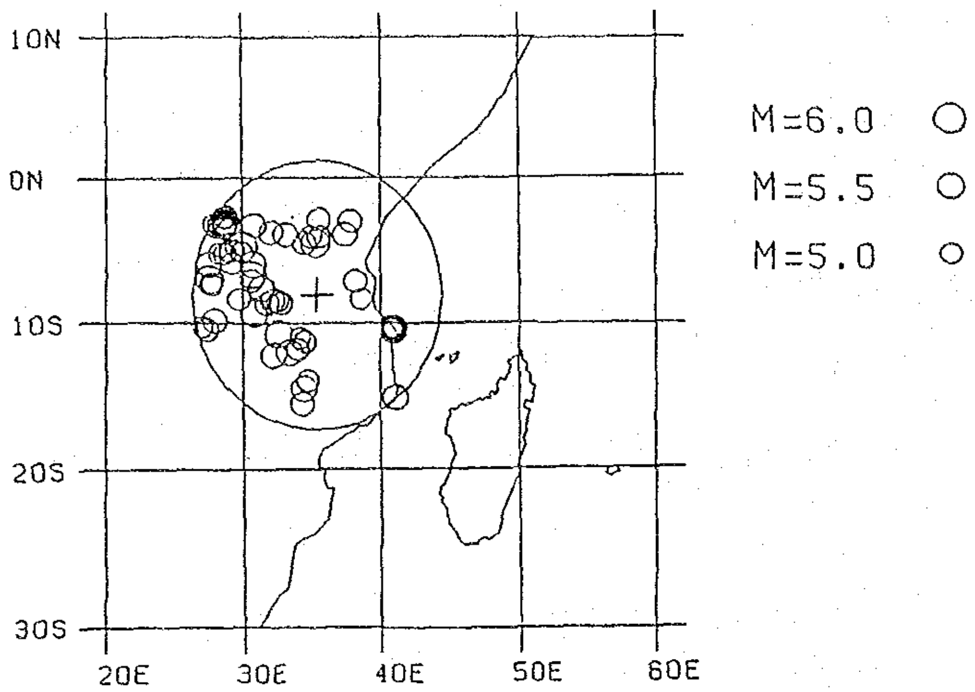
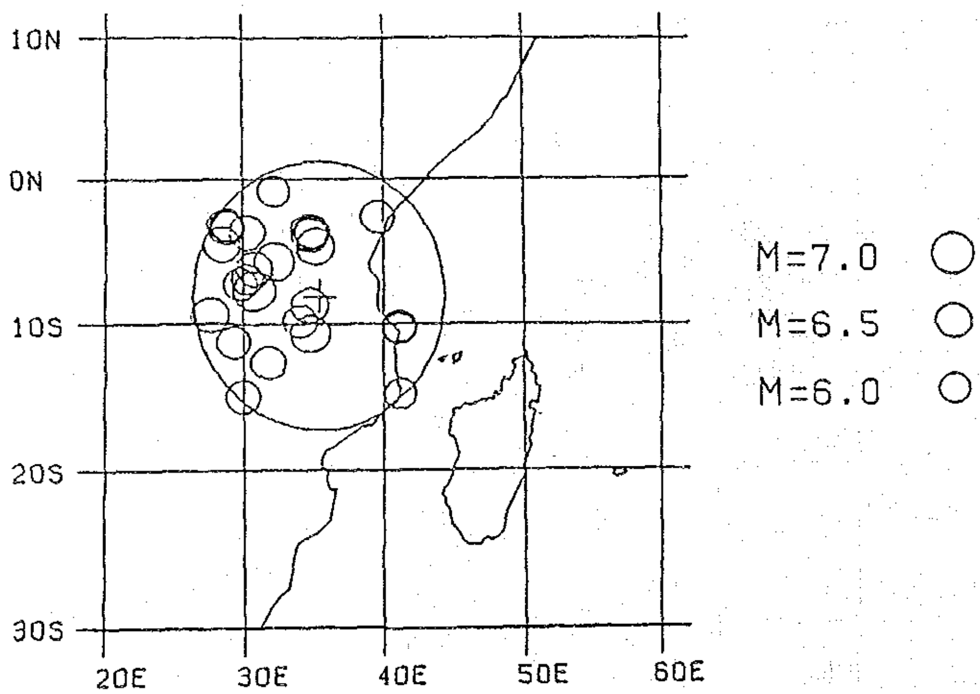



Fig. 7-20 (4) Locations of Magnitudes (6 ≤ M < 7 during 1910 - 1987)



### 7.7.2 Design Seismic Coefficient

(1) Estimation of Maximum Acceleration at Project Site

In order to decide what the design seismic coefficient should be, predictions of peak accelerations at the ground surface in the project area were made by statistical analyses. The earthquake data used in these predictions were those which had been collected by NOAA (National Oceanic and Atmospheric Administration) of the U.S.A. The number of earthquake data amounts to 349 during the 78-year period from 1910 to 1987. The attenuation models applied in prediction and evaluation of maximum acceleration were the five below from among those proposed in the past. In these models, "A" indicates maximum acceleration (gal), "M" magnitude of earthquake, "R" hypocentral distance (km), and "D" epicentral distance.

$$\log A = 3.090 + 0.347M - 2 \log (R + 25) \quad (1)$$

proposed by C. Oliveira

$$\log A = 2.674 + 0.278M - 1.301 \log (R + 25) \quad (2)$$

proposed by R. K. McGuire

$$\log A = 2.041 + 0.347M - 1.6 \log D \quad (3)$$

proposed by L. Esteva and E. Rosenblueth

$$\log A = 2.308 + 0.411M - 1.637 \log (R + 30) \quad (4)$$

proposed by T. Katayama

$$\log (A/640) = (D+40)(-7.6+1.72M-0.1036M^2)/100 \quad (5)$$

proposed by S. Okamoto

The corresponding types of ground and methods of calculation differ for these equations (1) to (5). The features of the models are given below.

(1): Firm ground
(2): Firm ground

(3): Modification from firm ground to surface of bedrock

(4): Gives maximum acceleration at ground surface not considering ground characteristics. Results in average of (1) and (2)

(5): Based on observation data obtained using vertical shaft at Kinugawa Power Station

The maximum acceleration for any return period was calculated based on the theory of extreme values and applying Gumbel Type III (Gumbel, 1958). The distribution of earthquake data used in seismic risk analysis is as shown in Fig. 7-19. The results of calculation of maximum accelerations are given in Tables 7-8, 7-9 and 7-10.

**Table 7-8 Distribution of Magnitude and Epicentral Distance of the Seismic Data during 1910 - 1987**

| | 0 ≦ Δ < 50 | < 100 | < 200 | < 300 | < 400 | < 500 | < 500 | < 700 | < 800 | < 900 | ≦ 1000 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3.0 ≦ M < 3.5 | 0 | 0 | 0 | 1 | 0 | 0 | 1 | 0 | 0 | 1 | 0 | 3 |
| < 4.0 | 0 | 0 | 0 | 1 | 3 | 10 | 11 | 8 | 2 | 5 | 3 | 43 |
| < 4.5 | 0 | 0 | 3 | 5 | 15 | 9 | 20 | 16 | 7 | 12 | 6 | 94 |
| < 5.0 | 0 | 0 | 0 | 3 | 21 | 4 | 12 | 29 | 9 | 16 | 18 | 114 |
| < 5.5 | 0 | 0 | 0 | 0 | 7 | 6 | 4 | 14 | 4 | 9 | 10 | 54 |
| < 6.0 | 0 | 0 | 0 | 0 | 0 | 4 | 4 | 0 | 2 | 1 | 5 | 16 |
| < 6.5 | 0 | 0 | 0 | 1 | 0 | 1 | 3 | 5 | 3 | 2 | 3 | 18 |
| < 7.0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 0 | 0 | 1 | 1 | 7 |
| 7.0 ≦ | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Total | 0 | 0 | 4 | 12 | 48 | 35 | 58 | 72 | 27 | 49 | 46 | 349 |

Δ : Epicentral Distance (km)
M : Magnitude

**Table 7-9 Annual Maximum Accelerations during 1963 - 1987**

| Year | Oliveira,C. Eq.(1) | McGuire,R.K. Eq.(2) | Esteva,L.& Rosenblueth,E. Eq.(3) | Katayama,T. Eq.(4) | Okamoto,S. Eq.(5) |
|---|---|---|---|---|---|
| 1963 | 0.19 | 3.02 | 0.25 | 0.75 | 0.00 |
| 1964 | 0.71 | 7.92 | 0.84 | 2.88 | 0.02 |
| 1965 | 0.30 | 3.61 | 0.32 | 0.84 | 0.00 |
| 1966 | 0.26 | 3.29 | 0.28 | 0.73 | 0.00 |
| 1967 | 0.71 | 6.90 | 0.73 | 2.02 | 0.01 |
| 1968 | 0.40 | 4.27 | 0.40 | 1.05 | 0.00 |
| 1969 | 0.31 | 3.57 | 0.32 | 0.77 | 0.00 |
| 1970 | 0.52 | 5.34 | 0.53 | 1.38 | 0.00 |
| 1971 | 0.30 | 3.60 | 0.32 | 0.80 | 0.00 |
| 1972 | 0.60 | 5.81 | 0.60 | 1.51 | 0.00 |
| 1973 | 1.24 | 8.91 | 1.12 | 2.40 | 0.07 |
| 1974 | 0.38 | 4.42 | 0.41 | 1.11 | 0.00 |
| 1975 | 0.40 | 4.95 | 0.47 | 1.40 | 0.00 |
| 1976 | 0.57 | 6.31 | 0.64 | 1.95 | 0.00 |
| 1977 | 0.66 | 6.57 | 0.68 | 1.90 | 0.00 |
| 1978 | 0.35 | 4.32 | 0.40 | 1.11 | 0.00 |
| 1979 | 0.49 | 5.28 | 0.52 | 1.41 | 0.00 |
| 1980 | 0.69 | 6.22 | 0.66 | 1.60 | 0.00 |
| 1981 | 0.78 | 6.32 | 0.71 | 1.50 | 0.01 |
| 1982 | 0.29 | 3.41 | 0.30 | 0.71 | 0.00 |
| 1983 | 0.30 | 3.88 | 0.35 | 0.99 | 0.00 |
| 1984 | 0.58 | 6.19 | 0.63 | 1.82 | 0.00 |
| 1985 | 1.12 | 8.31 | 1.01 | 2.21 | 0.04 |
| 1986 | 0.50 | 5.36 | 0.53 | 1.43 | 0.00 |
| 1987 | 0.61 | 5.93 | 0.61 | 1.58 | 0.00 |

**Table 7-10 Maximum Accelerations for Six Return Periods (gal)**

| Model (Eq. No.) | Return Period (Year) | | | | | |
|---|---|---|---|---|---|---|
| | 50 | 100 | 200 | 500 | 1000 | 10000 |
| (1) by C. Oliveira | 1.1 | 1.2 | 1.3 | 1.4 | 1.5 | 1.6 |
| (2) by R.K. McGuire | 9.3 | 9.9 | 10.4 | 11.0 | 11.4 | 12.2 |
| (3) by Est. & Rosen. | 1.1 | 1.2 | 1.3 | 1.4 | 1.4 | 1.5 |
| (4) by T. Katayama | 2.9 | 3.2 | 3.4 | 3.6 | 3.8 | 4.2 |
| (5) by S. Okamoto | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 |

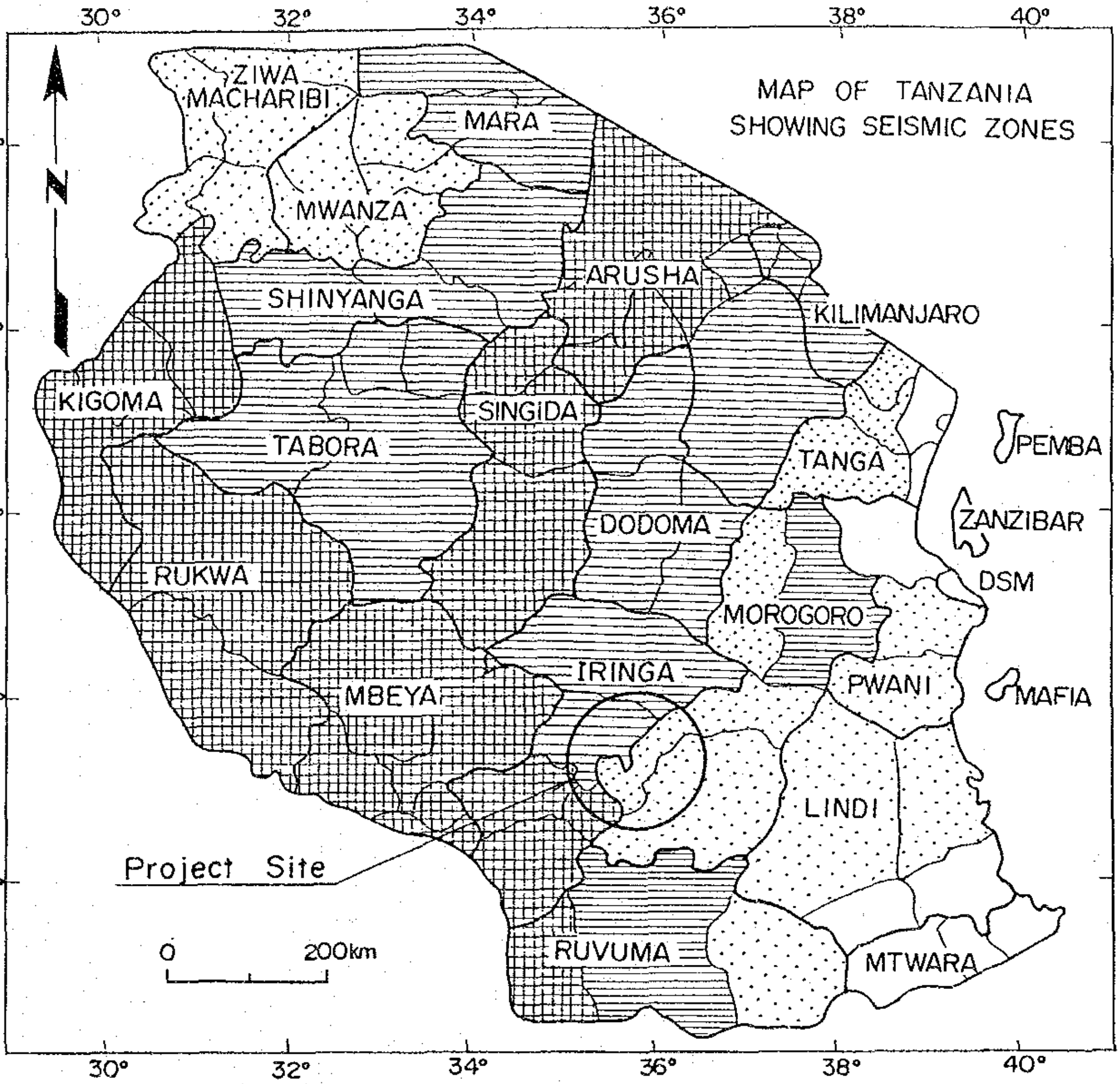
(2) Seismic Coefficient Adopted

The design seismic intensity of the project site is approximately 13 gal for a 10,000-year return period according to the result of statistical analysis carried out under estimation of maximum acceleration. It is adequate for a maximum of about 0.05 to be adopted from this as the design horizontal seismic coefficient.

On the other hand, according to the data collected in Tanzania shown in Fig. 7-21, the project site is located at the boundary between Zone 1 and Zone 2, and moreover, it is only a very short distance from Zone 3.

The design horizontal seismic coefficient of the project site located at such a place was therefore set as 0.10 taking into consideration both stochastic analyses and the abovementioned data.

Fig. 7-21 Seismic Map of Tanzania




HAZARD LEVEL OF EARTHQUAKES Richter scale

- ZONE 0, rare or unknown, magnitude M = 0 - 4
- ZONE 1, fairly week, less frequent, magnitude M = 3 - 5
- ZONE 2, moderately frequent, not violent, magnitude M = 5 - 6
- ZONE 3, frequent, strong, magnitude M = 6 - 7

( BY MINISTRY OF WORKS TANZANIA, BUILDING DIVISION, 1978 )

# Chapter 8 DEVELOPMENT PLAN

Chapter 8

# DEVELOPMENT PLAN

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# Chapter 8 DEVELOPMENT PLAN

## 8.1 Comparison Study on Development Schemes

### 8.1.1 Review of Existing Schemes

(1) Outline of Hydroelectric Power Development of Rufiji River Basin

The Rufiji River located in the central-south part of Tanzania is the largest river in the country with a catchment area of 177,000 $km^2$ corresponding to one fifth of the entire area of the country. The river basin is constituted of numerous tributaries, which broadly divided, comprise the three systems of the Great Ruaha River System, the Kilombero River System, and the Luwegu-Luhombero River System.

Regarding the Rufiji River Basin, the United Nations Food and Agriculture Organization (FAO) carried out a comprehensive survey for irrigated agriculture development in 1956 - 1961, and among the dam sites suitable for flood control and storage of irrigation water confirmed at that time, there were the Stiegler's Gorge, Kingena, and Njangasi sites of the Luwegu River System which were planned as sites where hydroelectric development of low head would be possible. However, survey of the possibilities for other high-head hydroelectric power projects were not made at all.

Later, in 1982, the Rufiji Basin Development Authority (RUBADA) carried out a desk study of the hydroelectric potential of the Rufiji River Basin based on the beforementioned results of study by the

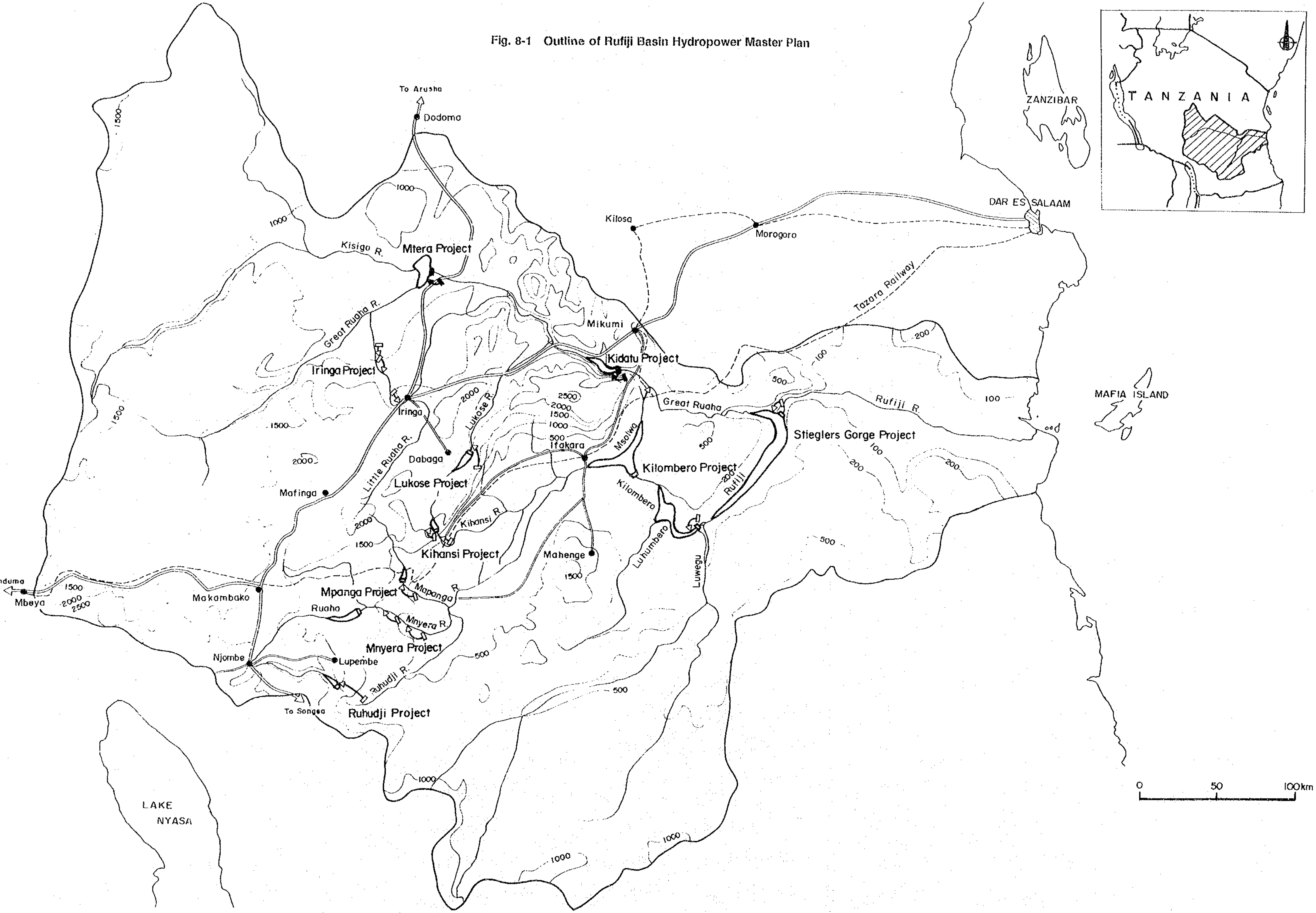
FAO and 1:50,000 topographic maps published subsequent to that study, and it was confirmed that numerous high-head hydroelectric power development sites exist at the upstream part of the Rufiji River.

In order to examine the results of this desk study in further detail, RUBADA commissioned Norconsult to make a study, and in 1984, the Rufiji River Basin Hydroelectric Power Development Master Plan was formulated. The outline of this Master Plan is given in Fig. 8-1 and Table 8-1.

**Table 8-1 Potential Hydropower Project in Rufiji Basin (Optimum Sequence - Standard Demand Forecast)**

| Project | Date Required | Installed Capacity (MW) | Firm Energy (GWh/year) | Capital Cost ($10^6$ $) |
|---|---|---|---|---|
| Kihansi River | 1992 | 285 | 1,344 | 310.5 |
| Mpanga River | 2001 | 160 | 617 | 173.5 |
| Ruhudji River | 2005 | 685 | 2,173 | 492.0 |
| Mnyera River | 2015 | 485 | 2,506 | 639.0 |
| Iringa | 2023 | 87 | 412 | 133.0 |
| Lukose River | 2024 | 130 | 566 | 177.0 |
| Stiegler's Gorge | 2025 | 1,400 | 5,880 | 1,181.0 |
| Kilokmbero River | After 2034 | 464 | 2,171 | 954.0 |

Fig. 8-1 Outline of Rufiji Basin Hydropower Master Plan



According to this Master Plan, it is estimated to be possible for hydroelectric power development to be done at 14 sites in the Rufiji River Basin for a total output of 3,696 MW, the order of priority in development being two sites on the Kihansi River, total output 285 MW, one site on the Mpanga River, 160 MW, three sites on the Ruhudji River, total output 685 MW, two sites on the Mnyera River, total output 485 MW, three sites on the Little Ruaha River, total output 87 MW, one site on the Lukose River, 130 MW, one site on the Rufiji River mainstream, 1,400 MW, and one site on the Kilombero River, 464 MW.

The upstream part of the Kihansi River down to an elevation of approximately 1,400 m is of a gentle river-bed gradient of 1/50 to 1/130, but from this vicinity it abruptly changes and becomes a swift stream of river-bed gradient 1/30. Downstream part of EL. 1,100 m, the Uzungwa Scarp of a height difference of approximately 800 m is descended in a distance of approximately 5 km by a series of waterfalls including Kihansi Falls of a drop of 200 m, comprising a violent stream down to the Kilombero Plain of elevation approximately 300 m. The runoff of the Kihansi River is large with an average runoff of 0.027 $m^3/s/km^2$, and moreover, the runoff duration is stable throughout the year. According to the Master Plan, the swift stream portion of the Kihansi River is to be developed divided into two stages, the Upper Kihansi Project and the Lower Kihansi Project.

Further, in 1985, TANESCO commissioned ACRES to carry out a survey for electric power development in Tanzania from 1985 to 2010, and in this survey also, the two sites of the Kihansi River System, with total output modified to 207 MW, were named as the first that should be developed in the Rufiji River Basin.

(2) Review of Existing Development Schemes

The Kihansi Project proposed in the Master Plan is that of an Upper Kihansi Project of constructing a rockfill dam of height of 65 m, high water level of EL. 1,560 m, and effective storage capacity of 140 $\times 10^6$ $m^3$ at a point of river-bed elevation 1,500 m on the Kihansi River mainstream 500 m downstream from the confluence with the tributary Ingomo River. Drawing water of a maximum available discharge of 24.0 $m^3/s$, and conducting the water to an underground powerhouse by a headrace tunnel 2.95 km in length for generation of 45 MW. And discharging into the Kihansi River at EL. 1,320 m, 5 km downstream of Uhafiwa Village, by a tailrace tunnel 4.67 km in length. And a Lower Kihansi Project of constructing a concrete gravity dam of high water level elevation of 1,320 m at a point of river-bed elevation 1,300 m on the Kihansi River mainstream approximately 1.5 km downstream of the Upper Kihansi Project outlet. Drawing water from this for a maximum power discharge of 28.0 $m^3/s$, conducting this water to an underground powerhouse by a headrace of 2.6 km and a penstock of 1.4 km for generation of a maximum 240 MW. And discharging the water into the Kihansi River at EL. 295 m approximately 0.5 km upstream of Kihansi Bridge by a tailrace tunnel 4.85 km in length.

In the Master Plan, the Kihansi Project was to be developed divided into 6 phases as shown in Table 8-2.

Table 8-2 Development Phasing of Kihansi Project

| Phase | Installed Capacity (MW) | | | | |
|---|---|---|---|---|---|
| | Upper | | Lower | | Total |
| | Expansion | Cumulative | Expansion | Cumulative | Cumulative |
| 1 | Dam only | | 60 | 60 | 60 |
| 2 | | | 60 | 120 | 120 |
| 3 | | | - | 120 | 120 |
| 4 | | | 60 | 180 | 180 |
| 5 | | | 60 | 240 | 240 |
| 6 | 45 | 45 | - | 240 | 285 |

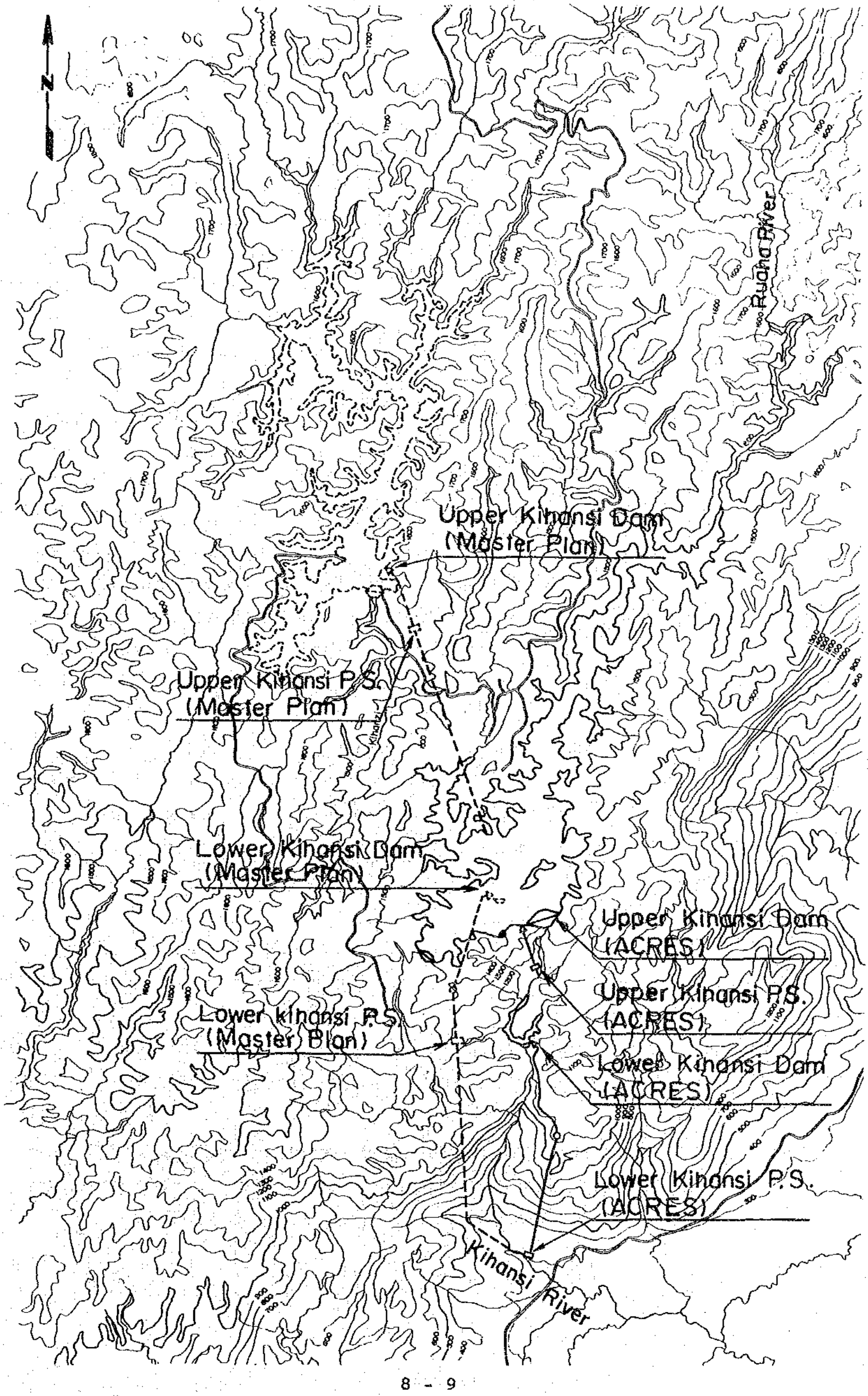
On the other hand, in the review of the Master Plan by ACRES, a large-scale modification was made in the Lower Kihansi Project to improve the economic nature of the Project. The Lower Kihansi Project proposed as a result construction of an earthfill dam 15 m in height with high water level of EL. 1,130 m at a point of river-bed elevation 1,120 m on the Kihansi River mainstream approximately 4 km southeast of Ukami Village, with which water of a maximum discharge of 22.8 $m^3/s$ is to be drawn, and with this water conducted by a headrace of 2.05 km and a penstock of 3.0 km to a powerhouse to be constructed at the left-bank ground surface of the Kihansi River approximately 1 km upstream of Kihansi Bridge for generation of 162 MW. With regard to the Upper Kihansi Project, ACRES did not make a detailed study, but the dam site is selected at a point of river-bed elevation 1,290 m on the Kihansi River mainstream approximately 3 km downstream of its confluence with the tributary Great Ruaha River.

The development schemes proposed in the Master Plan and the outline of the development schemes proposed as a result of review by ACRES are shown in Table 8-3 and Fig. 8-2.

Table 8-3 Development Scheme of Kihansi River

| Description | Unit | Master Plan Scheme | | Acres Report Scheme | |
|---|---|---|---|---|---|
| | | Upper Kihansi | Lower Kihansi | Upper kihansi | Lower Kihansi |
| Catchmetn area | km2 | 377 | 581 | 583 | 590 |
| River runoff | $m^3/s$ | 6.8 | 16.6 | 16.6 | 16.9 |
| Reservoir | | | | | |
| High Water Level | m | 1,560.00 | 1,320.00 | 1,400.00 | 1,132.00 |
| Low Water Level | m | 1,530.00 | 1,315.00 | 1,330.00 | 1,130.00 |
| Surface Area | $km^2$ | 8.2 | ---- | 11.2 | ---- |
| Effective Storage Capacity | $10^6m^3$ | 140.00 | ---- | 409.00 | ---- |
| Dam | | | | | |
| Type | | Rockfill | Concrete Gravity | Rockfill | Rockfill |
| Crest Elevation | m | 1,565.00 | 1,324.50 | 1,403.00 | 1,135.00 |
| Height | m | 65 | 33 | 115 | 15 |
| Crest Length | m | 370 | 200 | 850 | 100 |
| Volume | $10^6m^3$ | 2,150 | 50 | 11,000 | 300 |
| Development Scheme | | | | | |
| Standard Intake Water Level | m | 1,550.00 | 1,320.00 | 1,377.00 | 1,130.00 |
| Tail Water Level | m | 1,320.00 | 295.00 | 1,140.00 | 290.00 |
| Effective Head | m | 220.00 | 1,010.00 | 230.00 | 798.00 |
| Maximum Discharge | $10^3/s$ | 24.0 | 28.0 | 23.0 | 22.8 |
| Installed Capacity | MW | 45 | 240 | 45 | 162 |
| Annual Firm Energy | $10^6kWh$ | 106.0 | 1,245 | 222 | 945 |
| Investment Cost | $10^6$ S | 136 | 174.5 | 169.5 | 151.7 |

Fig. 8-2 Development Scheme of Kihansi River



The aim of the review by ACRES is that since the development scheme proposed in the Master Plan is for an underground powerhouse to be provided deep underground for the Lower Kihansi Project, the headrace, penstock, tailrace, and the respective access tunnels are all long with an enormous construction cost, so that the dam site is altered to a point downstream with the greater part of the headrace made an open canal and the penstock and powerhouse arranged on the ground surface to reduce the construction cost. As a result of the reexamination of the two development schemes shown in the Table below, the Lower Kihansi Project proposed by ACRES is to have a reduction in effective head of only 21 per cent whereas the reduction in construction cost is estimated to be as much as 30 per cent, and the economics is greatly improved. This construction cost does not include interest during construction, and since the development scheme proposed in the Master Plan includes an inclined penstock shaft 1,200 m in length, the longer construction period is to be required, and when interest during construction for that period is considered the reduction in construction cost with the modified scheme of ACRES is to reach 32 per cent.

| | Master Plan (A) | ACRES Plan (B) | (B)/(A) |
|---|---|---|---|
| Effective Head | 1,010 m | 798 m | 79% |
| Construction Cost without IDC | $216.1\times10^6$$ | $151.7\times10^6$$ | 70% |
| Construction Cost with IDC | $267.8\times10^6$$ | $182.0\times10^6$$ | 68% |

Further, the reexamination made by ACRES was hoped that the Lower Kihansi Project could start operation in 1997, so that shortening of the construction period was an important condition.

The aim of reexamination of the Upper Kihansi Project was that since in the development scheme proposed in the Master Plan the location of the dam site was selected at a point upstream of the confluence with the Ruaha River, the discharge into the reservoir was to be only 40 per cent of that of the Lower Kihansi Project. This is to lead to a small regulating effect of the reservoir. While the head obtained by the total waterway length of 7.8 km is to be only 240 m, so that the dam site was shifted to downstream of the confluence with the Ruaha River to increase the regulating effect of the reservoir, and to obtain a head of 230 m with a total waterway length of 1.2 km, thereby greatly improving the economics of the development scheme.

The development scale proposed in the Master Plan is to be for annual plant factors of under conditions of firm discharge of only 25 per cent with the Upper Kihansi Project and 38 per cent with the Lower Kihansi Project. Considered from the present annual load factor in Tanzania of about 64 per cent with the power supply structure dominated by hydro electric power, and the fact that this trend is to continue in the future, it is judged that the development scale proposed in the Master Plan is excessively large. On the other hand, with the development scale proposed by ACRES, the annual plant factors under firm discharge is estimated to be about 56 per cent for the Upper Kihansi Project and 67 per cent for the Lower project, and from the viewpoint of plant factor the scale may be judged to be rather small. However, from the standpoint of the reservoir scale, the effective storage capacity is to correspond to as much as 78 per cent of the annual inflow, and when the fact that the runoff duration of the Kihansi River is stable is considered, it may be judged to be excessive.

Field investigations were made regarding the development scheme proposed in the Master Plan and the development scheme proposed by ACRES. As a result, the conclusion reached was that the development scheme proposed by ACRES is fundamentally reasonable, but it is necessary for the following modifications to be added:

Upper Kihansi Project

- From the standpoint of the topography, in view of the elevation of the saddle at the right-bank side of the dam site, the reservoir high water level is necessary to be made EL. 1,390 m or lower.

- In view of the runoff duration of the river being stable, the optimum reservoir storage capacity is thought to be smaller than the capacity in the ACRES proposal.

- The powerhouse is planned as an underground type, but it is necessary to consider a surface type also.

Lower Kihansi Project

- The scheme is for a run-of-river type with no regulating reservoir, but when operation during the period until completion of the Upper Kihansi dam, alleviation of restrictions on operation of the Upper and Lower projects after start of operation of the Upper Kihansi Project, and effective runoff utilization of the remaining catchment area are considered, it is thought reasonable for a regulating reservoir to be provided. Further, since the dam site is at a narrow point, it is difficult to construct a settling basin, and from this aspect also it is reasonable to have a regulating reservoir.

- Since the dam site has been selected at a point immediately below the confluence with a tributary, the valley width is large and the dam crest length long to make the dam volume large, but at a site 500 m downstream of the confluence, the valley width is narrow and that site is thought to be suitable as the dam site.

- The headrace has been made an open canal except in part, but the topography of the waterway route is rugged and complex so that an open canal is not suitable. The penstock has been made a surface type, but the topography of the site proposed for the penstock is complex, while moreover, there are places where the slope has collapsed so that a surface type is not suitable.

Sequence of Development

- The Master Plan contemplates the Kihansi Project being developed divided into 6 phases, while also in the reexamination by ACRES, this system has been adopted in part, but the Kihansi River has a large firm discharge and it is possible for operation to be started from the beginning at about 80 per cent of the entire capacity after completion of the upper dam even at the time of commissioning of Lower Kihansi Power Station alone. The power demand is of an urgent nature and it is necessary for Upper Kihansi Power Station to start operation simultaneously with completion of the upper dam. Therefore, it is thought suitable for the Kihansi Project to be developed in the two phases of the Lower Kihansi Project as Phase I, and Upper Kihansi Project as Phase II.

### 8.1.2 Selection of Optimum Development Scheme

(1) Basic Principles

In making a study of the entire development scheme, since dam sites for reservoirs do not exist other than where proposed in the Master Plan and by ACRES, it is considered that the fundamental development scheme alternatives are limited to just two. However, the development scheme proposed by ACRES is clarified to be modified as a result of review upon field investigations, and therefore, the optimum development scheme is to be selected upon making modifications and then comparing with the development scheme proposed in the Master Plan.

For the optimum development scheme selected by the above procedure, detailed investigation works are to be carried out, and based on the results, a final decision is to be made concerning the optimum development plan on examination of such element as the reservoir scales, dam heights, maximum discharges.

(2) Conditions of Comparison Study

Comparison studies for the optimum development scheme are to be made based on the following conditions:

- A development scheme for large firm energy and low generating cost is to be selected.

- Firm energy is to be calculated based on 100 per cent probability discharge for a reservoir-type development scheme, and 97 per cent probability discharge for a regulating reservoir-type or a run-of-river-type development scheme.

- Energy calculation is to be done utilizing runoff data in the Master Plan Report.

- Estimation of construction cost is to be done by same manner described in the Master Plan Report.

- A development scheme which is to be possible to be constructed by the year scheduled for start of operation (1997) is to be selected.

(3) Modifications of the Alternative Scheme Proposed by ACRES

i) Reservoir Dam

In order for the regulating effect by the reservoir to be made a maximum for the Lower Kihansi Project also, it is necessary for the dam site to be selected downstream of the confluence of the Kihansi River and the Great Ruaha River. In such case, the only locations with the possibility as dam sites are the site proposed by ACRES as the upper dam site, and the site proposed as the lower dam site, also by ACRES. Judging by topographical conditions, the reservoir scale possible for the former is up to high water level elevation of 1,390 m and total storage volume of $330 \times 10^6$ $m^3$ so that adequate capacity is secured as a reservoir, whereas for the latter, a high water level of EL. 1,180 m and total storage capacity of $34 \times 10^6$ $m^3$ is to be the limit, and when dead storage capacity and intake static draft head are considered, an effective storage volume of $15 \times 10^6$ $m^3$ is the maximum for an insufficient capacity as a reservoir, and therefore, as a dam site for a reservoir there is no location available other than the Upper Kihansi Dam site proposed by ACRES.

ii) Single-Stage Development Scheme

To use the head from the Upper Kihansi dam site to the Lower Kihansi powerhouse in a single-stage development scheme means it is unavoidable for the powerhouse to be an underground type deep inside the mountain, in which case, in order for the construction cost to be held to a degree comparable with other development schemes, a construction period of about 8 years is considered to be necessary and this is judged to be unrealistic.

iii) Regulating Reservoir

In the Master Plan and the development scheme proposed by ACRES, a regulating reservoir is not provided for the Lower Kihansi Project.

Development of the Lower Kihansi Project is to be started initially about 3 years in advance of the Upper Kihansi Project, and even in such case the annual firm energy is estimated to be about 490 GWh, corresponding to as much as 20 per cent of the energy production of the entire electric power system. Because of this, it is judged unsuitable for the power system that, as a run-of-river power station, only base load can be supplied and peaking operation in accordance with load variations is not possible. Also, after start of operation of the Upper Kihansi Project, it becomes necessary for completely identical operation to be done at the Upper and Lower power stations, while at low water level of Upper Kihansi Reservoir, maximum capacity operation is impossible for both the Upper and Lower power stations, which likewise are unsuitable from the standpoint of the power system. Consequently, a regulating reservoir is to be provided in the Lower Kihansi Project.

iv) Waterway Route

In the Upper Kihansi Project, the waterway length is shorter for a right-bank route compared with a left-bank route, while it is possible for the penstock and the powerhouse to be provided at the ground surface. Hence, it is judged that a right-bank route is judged to be optimum. In the Lower Kihansi Project, the waterway length is shorter with a left-bank route compared with a right-bank route, and a left-bank route is considered to optimum, while from the standpoint of the topography, it is judged suitable for the headrace and penstock to be tunnel-type.

v) Installed Capacity

It is necessary for the installed capacity of a development scheme to be decided in accordance with the conditions of demand in the power system. The power generating capability of the system at the time of start-up of the Kihansi Project is planned as given in Table 8-4.

**Table 8-4 Generating Facilities in 1996 (from ACRES Report, 1989)**

| Name of P/S | Installed Capacity (MW) | Annual Energy (GWh) | Firm Energy (GWh) |
|---|---|---|---|
| Pangani System | | | |
| Great Ruaha River | 330 | | 1,480 |
| Other Rivers | | | |
| Panagani Redevelop | 43 | | 195 |
| Thermal | 110 | | 548 |
| Ground Total | 483 | | 2,223 |

According to the study by ACRES in 1989, the power demand at that time was anticipated as 2,306 GWh/yr, with maximum load as 411 MW (annual load factor 64 per cent). Up to 77 per cent of the power supply of the system is made up by hydro electric, with all of those plants having reservoirs with which annual regulation of runoff is to be possible, and the annual plants factors are around 51 to 57 per cent. Considering reserve capacity of the system to be about 15 per cent of maximum load, demand and supply are almost balanced in terms of both kWh and kW. Accordingly, demand corresponding to subsequent new demand will correspond to power sources to be newly developed. Consequently, the Kihansi Project is to meet the demand corresponding to new demand from 1997 to 2002.

Therefore, the installed capacity of the Kihansi Project is to be determined with equivalent peak duration time as 13 hours considering the present and the future variation in the form of demand.

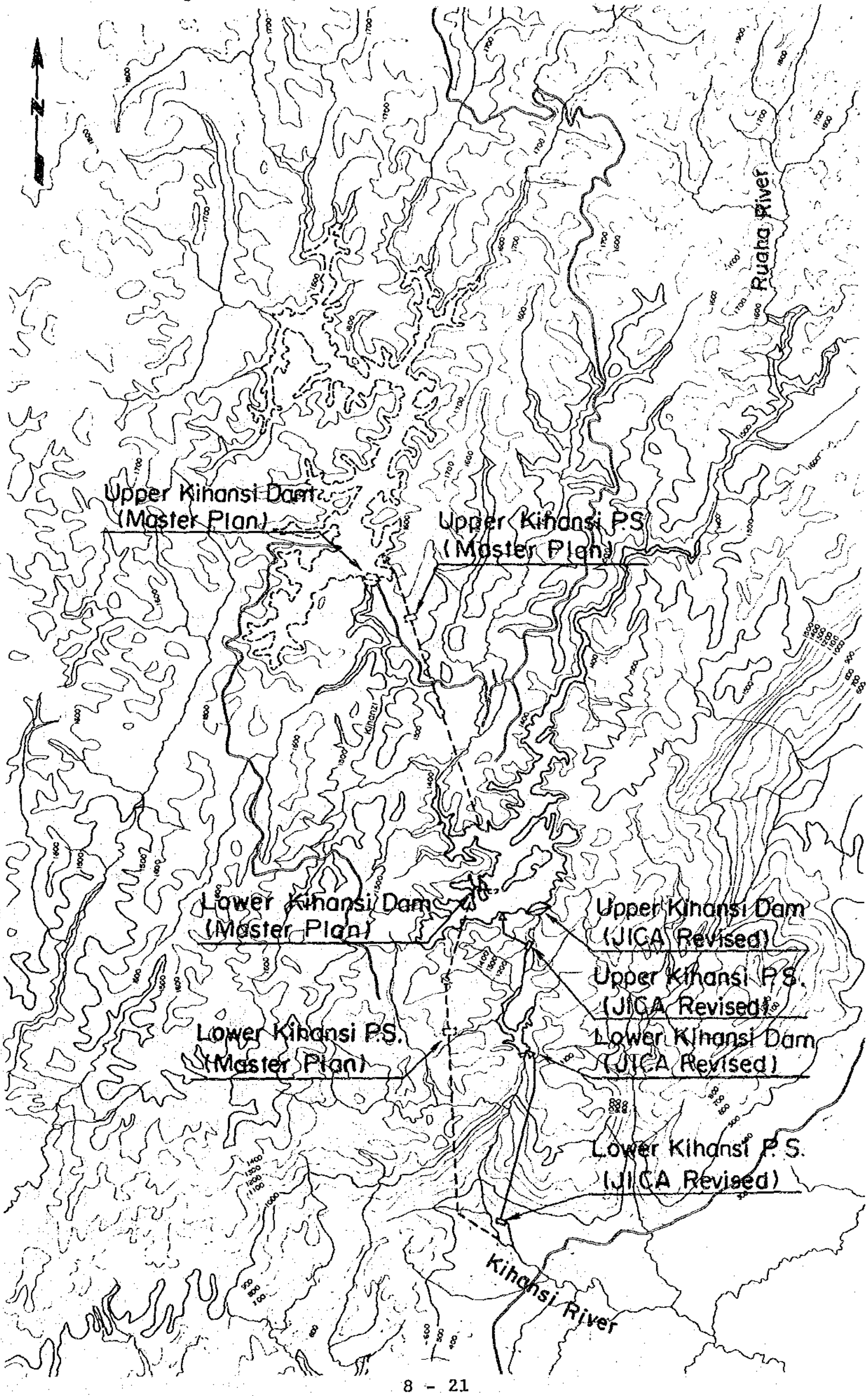
(4) Comparison with Master Plan

A comparison of the development scheme modified as described in (3) and the development scheme proposed in the Master Plan is shown in Table 8-5 and Fig. 8-3.

## Table 8-5 Comparison Study on Development Scheme

| Item | Unit | Master Plan Scheme | | | Revised Acres Scheme | | |
|---|---|---|---|---|---|---|---|
| | | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total |
| Catchment Area | $km^2$ | 374 | 577 | | 583 | 590 | |
| Reservoir | | | | | | | |
| High Water Level | m | 1,560.00 | 1,320.00 | | 1,370.00 | 1,135.00 | |
| Low Water Level | m | 1,530.00 | 1,315.00 | | 1,330.00 | 1,127.00 | |
| Gross Storage Capacity | $10^6$ $m^3$ | 170.00 | 7.00 | | 175.00 | 0.73 | |
| Effective Storage Capacity | $10^6$ $m^3$ | 140.00 | ---- | | 144.00 | 0.55 | |
| Available Drawdown | m | 30.0 | ---- | | 40.0 | 7.0 | |
| Dam | | | | | | | |
| Type | | Rockfill | Concrete Gravity | | Rockfill | Concrete Gravity | |
| Height x Length | m | 65 x 370 | 33 x 200 | | 85 x 630 | 25 x 110 | |
| Volume | $10^3$ $m^3$ | 2,150 | 50 | | 4,820 | 25 | |
| Development Scheme | | | | | | | |
| Maximum Discharge | $m^3/s$ | 24.0 | 28.0 | | 26.4 | 26.5 | |
| Standard Intake Water Level | m | 1,550.00 | 1,320.00 | | 1,356.70 | 1,131.00 | |
| Tail Water Level | m | 1,320.00 | 295.00 | | 1,140.00 | 290.00 | |
| Gross Head | m | 240.00 | 1,025.00 | | 216.70 | 841.00 | |
| Effective Head | m | 220.00 | 1,010.00 | | 208.70 | 809.50 | |
| Installed Capacity | MW | 45 | 240 | | 47.3 | 180.8 | |
| Annual Firm Energy | $10^6$KWh | 96 | 791 | 887 | 185 | 869 | 1,054 |
| Investment Cost | $10^6$ $ | 136.0 | 174.5 | 310.5 | 176.6 | 172.2 | 348.8 |
| Unit Cost for Firm Energy | $/kWh | 1.41 | 0.22 | 0.35 | 0.95 | 0.20 | 0.33 |

Fig. 8-3 Comparison Study on Development Scheme



The firm discharges in the development scheme proposed in the Master Plan are 6.56 $m^3/s$ for the Upper Kihansi Project and 10.62 $m^3/s$ for the Lower Kihansi Project, and the firm energy amounts with these discharges are 96 GWh for the Upper Kihansi Project and 791 GWh (including downstream increase due to the Upper Kihansi Project) for the Lower Kihansi Project. The development scheme modified as described in "(3) Modifications of the Alternative Scheme Proposed by ACRES" is made to have a high water level elevation of 1,370 m in order to provide a reservoir scale of about the same degree as in the Master Plan.

The modified development scheme is superior to the development scheme proposed in the Master Plan with regard to firm energy, construction cost per unit energy, and construction period. Therefore, the detailed investigations are to be made on the modified development scheme as the optimum development scheme.

## 8.2 Study on Development Scale

### 8.2.1 Basic Condition for the Study

(1) Fundamental View

Study on development scale is to be carried out on the optimum development scheme selected in "8.1 Comparison Study on Development Scheme" based on the result of the detailed investigation works.

The development scale of this project is to be judged by utilizing cost of the most practical alternative thermal power plant as a benefit which will be built in Tanzania in case that there is no available hydroelectric power project in Tanzania.

In this study, the indigenous coal thermal power plant at mine mouth is adopted as the most practical alternative thermal power plant in Tanzania.

The possibility of the following alternative of this project has been considered. Four thermal power plants of different type: indigenous coal thermal power plant at mine mouth, indigenous coal thermal power plant at Dar Es Salaam, imported coal thermal power plant at Dar Es Salaam and oil thermal power plant at Dar Es Salaam.

The oil thermal power plant has to be excluded from alternative because of the present energy situation in the world. The imported coal thermal power plant at Dar Es Salaam will have a benefit of cheaper fuel cost but will require a large sum of investment cost for construction of port facilities and coal handling facilities. The indigenous coal thermal power plant at Dar Es Salaam will require a heavy expense for inland transportation of fuel (US$42/ton) and also will have a difficulty in securing the transportation means.

Therefore, the indigenous coal thermal power plant at mine mouth has been selected as the most practical alternative thermal power plant in Tanzania.

As the indices of the study, the annual surplus benefit (B - C) and benefit cost ratio (B/C) obtained from equalized annual costs (C) for the project life of the hydro-electric power facility and the equalized annual cost (B) of the alternative thermal facilities are used. Market prices with import taxes in June 1989 are used for the study, and the cost of transmission lines to the trunk transmission line from the power plants are taken into account for the evaluation. Parameters of the alternative thermal power plant are shown in Table 8-6.

(2) Equalized Annual Cost

The equalized annual cost of a hydroelectric power facility consists of depreciation and operation-maintenance cost. This is estimated by multiplying the annual cost factor by the investment cost.

Equalized Annual Cost = Annual Cost Factor × Investment Cost
= Depreciation + Interest + Operation and Maintenance

Depreciation + Interest = Investment Cost × Capital Recovery Factor

° Capital Recovery Factor = $\frac{i(1+i)^n}{(1+i)^n - 1}$

- n: Service Life
  - Civil Facility 50 years
  - Hydro-mechanical Facility 35 years
  - Electro-mechanical Facility 35 years
- i: Discount Rate 10.00%

| | |
|---|---|
| Civil Facility | 10.09% |
| Hydro-mechanical Facility | 10.37% |
| Electro-mechanical Facility | 10.37% |

° Operation and Maintenance Cost (Rate to Investment Cost)

| | |
|---|---|
| Civil Facility | 0.50% |
| Hydro-mechanical Facility | 1.50% |
| Electro-mechanical Facility | 1.50% |

The annual cost factors of each facility are given below:

| | |
|---|---|
| Civil Facility | 10.59% |
| Hydro-mechanical Facility | 11.87% |
| Electro-mechanical Facility | 11.87% |

(3) Benefit

The benefit of the project is summarized according to the project cost, maintenance and operation costs, and the fuel cost of an alternative thermal power plant. The effective power output and effective energy that are used to calculate the advantages of the project are given according to the below conditions.

a) The effective power output at the receiving end is expressed by the below equation. This equation reduces the station service rate by 0.3 per cent, the forced outage rate by 0.3 per cent, the scheduled outage rate by 2.0 per cent, and the transmission loss rate by 1.8 per cent from the firm peak output. The firm peak output is defined as the 95 per cent probable output.

$$\text{Effective power output} = (1-0.003) \times (1-0.003) \times (1-0.02) \times (1-0.018) \times \text{Firm peak output}$$

b) The effective energy at the receiving end is expressed by the below equation that reduces the station service rate by 0.3 per cent and transmission loss rate by 2.0 per cent from the annual firm energy.

The reason why only the firm energy is adopted to evaluate the benefit is that there is little possibility to save fuel cost of thermal power plant by utilizing incremental energy from hydropower projects during the high water season since hydroelectric power dominates the system of Tanzania.

The annual firm energy is defined as annual average energy production during equivalent peak duration time.

$$\text{Effective Energy} = (1 - 0.003) \times (1 - 0.018) \times \text{Annual Firm Energy}$$

Table 8-6 Alternative thermal Power Plant for Optimization Study

Interest Rate = 10%

| Item | Unit | Description | |
|---|---|---|---|
| Type | | Coal Thermal Power Plant | |
| Installed Capacity | MW | 100 | |
| Annual Plant Factor | % | 75 | |
| Thermal Efficiency | % | 25.6 | |
| Annual Energy Production | $10^6$kWh | 657 | |
| Investment Cost | | | |
| Thermal Power Plant | $10^6$\$ | 148.0 | |
| Transmission Line | $10^6$\$ | 6.7 | |
| Service Life | | | |
| Thermal Power Plant | Years | 25 | |
| Transmission Line | Years | 35 | |
| Capital Recovery Factor | | | |
| Thermal Power Plant | % | 11.02 | |
| Transmission Line | % | 10.37 | |
| Heat Rate | Btu/kWh | 13,270 | |
| Operation & Maintenance Cost | | | |
| Thermal Power Plant | % | 3 | |
| Transmission Line | % | 1.5 | |
| Fuel Price | \$/MBtu | 2.28 | |
| Annual Cost | | | |
| Capital Receiving | | Fixed Cost | Variable Cost |
| Thermal Power Plant | $10^3$\$ | 16,309.6 | |
| Transmission Line | $10^3$\$ | 694.8 | |
| O & M Cost, Administration Cost | | | |
| Thermal Power Plant | $10^3$\$ | 3,996.0 | 444.0 |
| Transmission Line | $10^3$\$ | 90.5 | 10.1 |
| Fuel Cost | $10^3$\$ | | 19,907.9 |
| Total | | 20,396.1 | 20,362.0 |
| Annual Cost at Receiving End | | | |
| kW Cost | \$/kW | 274 [1)] | |
| kWh Cost | \$/kW | | 0.0351 [2)] |

1) $\frac{20,396.1 \times 10^3 \$}{100,000 \text{ kW}} \times 1.343^{3)} = 274 \text{ \$/kW}$

2) $\frac{20,362.0 \times 10^3 \$}{657 \times 10^6 \text{ kWh}} \times 1.134^{3)} = 0.0351 \text{ \$/kW}$

3) Adjustment Factor for kW & kWh

| Item | kW | kWh |
|---|---|---|
| Transmission Loss (%) | 6.2 | 6.2 |
| Station Service Rate (%) | 6.0 | 6.0 |
| Forced Outage Rate (%) | 4.0 | - |
| Scheduled Outage Rate (%) | 12.0 | - |

$$\text{kW Adjustment Factor} = \frac{1}{(1-0.062)\times(1-0.06)\times(1-0.04)\times(1-0.12)} = 1.343$$

$$\text{kWh Adjustment Factor} = \frac{1}{(1 - 0.062) \times (1 - 0.06)} = 1.134$$

### 8.2.2 Study on Reservoir Scale

(1) Reservoir Operation Plan

The seasonal variations and variations over the years in monthly average inflows of Upper Kihansi Reservoir during the 61-year period are shown in the mass curve of Fig. 8-4. Seasonally, there is a trend for inflow to be greatest during the half-year from January to June and smaller from July to December. The annual average inflow during the 61-year period is $495 \times 10^6$ m$^3$ (15.7 m$^3$/s), the inflow from January to June being $314 \times 10^6$ m$^3$ (20.1 m$^3$/s) and approximately 1.8 times the inflow during July to December of $181 \times 10^6$ m$^3$ (11.4 m$^3$/s). The inflows by year as seen over the years are $888 \times 10^6$ m$^3$ for the year of highest water (1968) during the 61-year period, approximately 3.2 times the inflow of the year of lowest water (1954) of $280 \times 10^6$ m$^3$. As the above figures show, the breadth of variation in inflow to the reservoir is small seasonally, but fairly large when looked at over the years. The periods that wet years and dry years occur consecutively are very long at about 20 years each. Accordingly, the increase in firm discharge by averaging out of inflow through the regulating effect of a reservoir is possible with less effective storage capacity, but in order to increase firm discharge through the replenishing effect of the reservoir requires a large-scale reservoir.

Calculations of energy production in making the study are performed by electronic computer using the monthly inflows for the 61-year period from January 1927 through december 1987.

The firm discharge is defined as the discharge that can always be used for power generation 95 per cent of the time during the 61-year period, and is determined so that power discharge is to be a maximum using the mass curve of inflow. The relationship between effective reservoir capacity and firm discharge is shown in Fig. 8-5.

In calculation of energy production, the ratio of available drawdown to the effective head is 25 per cent at maximum for the Upper Kihansi Project, but is less than 1 per cent for the Lower Kihansi Project, so that for the Upper Kihansi Project, an efficiency curve is considered, but for the Lower Kihansi Project constant efficiency is considered. The standard intake level to be the reference for design of turbines and generators, is given at high water level. Evaporation from the reservoir is ignored. For reservoir operation, and DP method is used and ideal rules are set up so that energy generation is to be a maximum.

The procedure for energy calculation is shown in Fig. 8-6. The area capacity curves for Upper Kihansi Reservoir and Lower Kihansi Regulating reservoir prepared from 1/5,000 topographic maps are shown in Figs. 8-7 and 8-8.

Fig. 8-4 Mass Curve at Upper Kihansi Dam Site

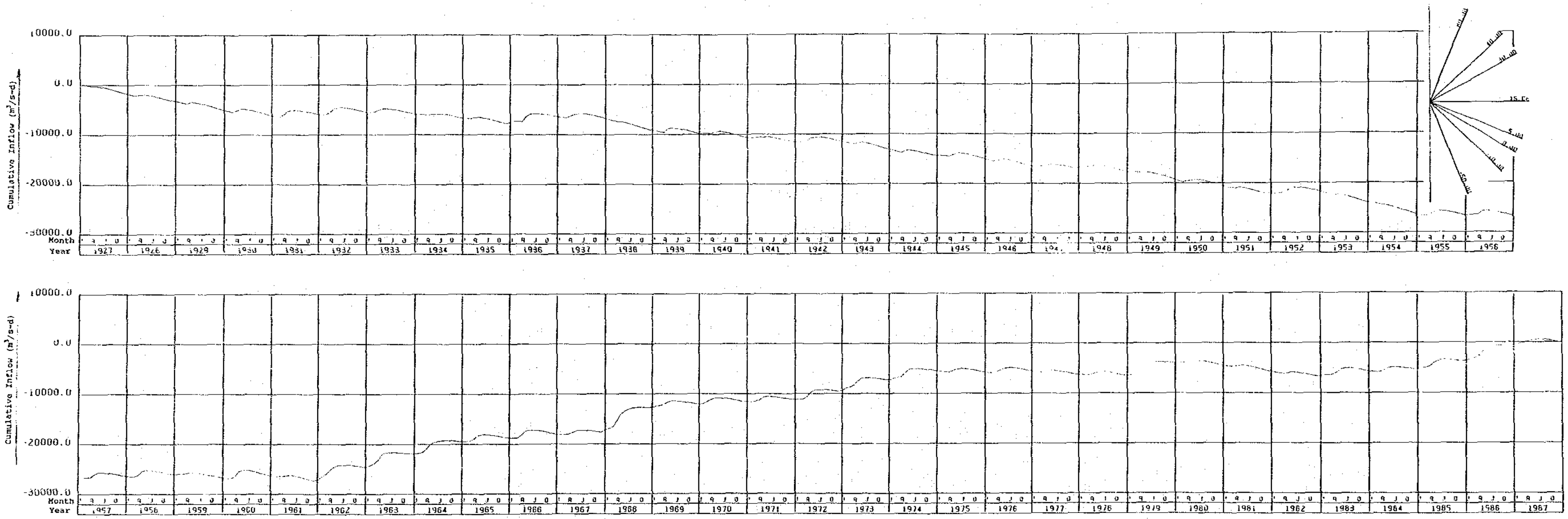
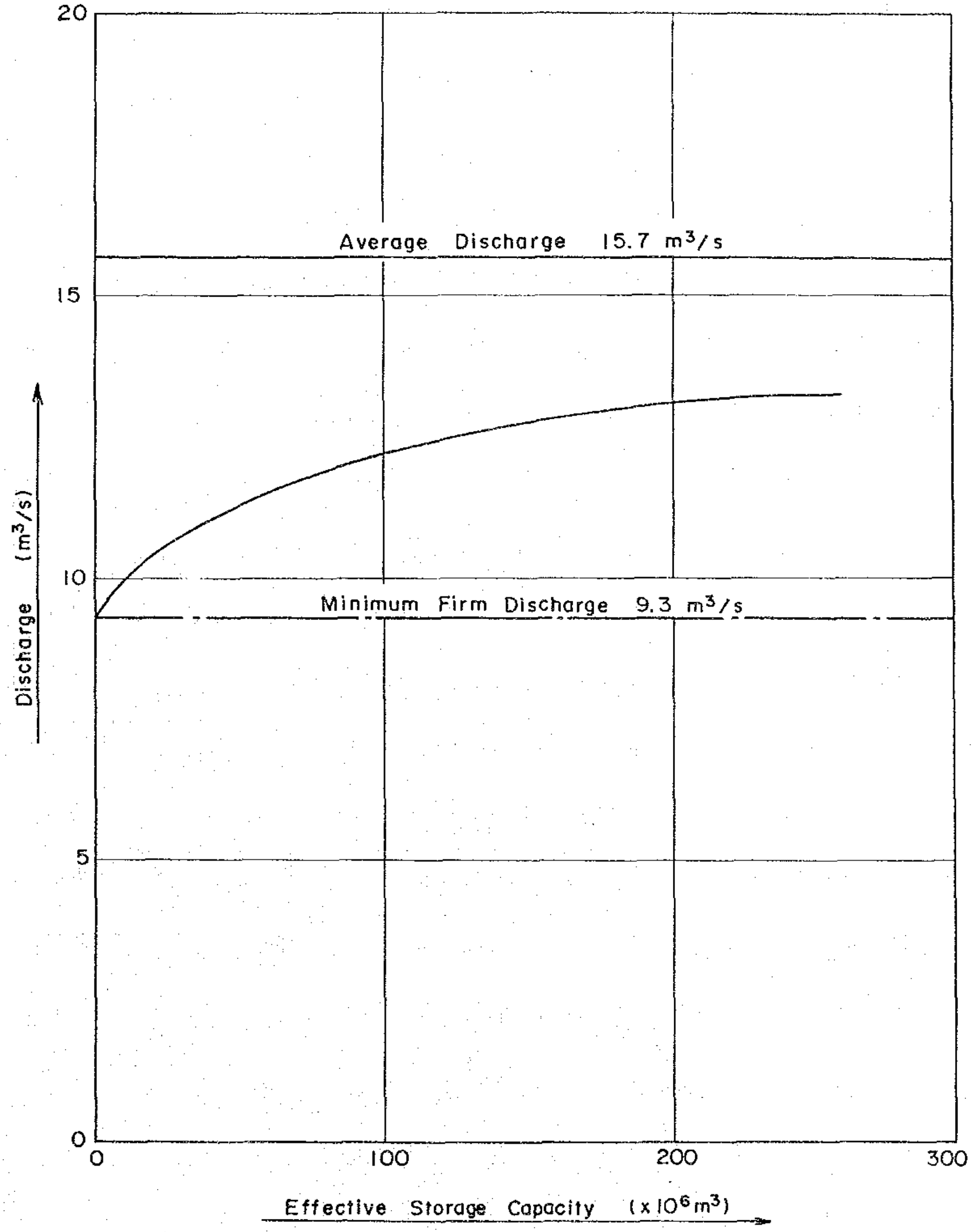


Fig. 8-5 Effective Storage Capacity and Firm Discharge at Kihansi Dam Site



## Fig. 8-6 Flow Chart of Calculation of Power and Energy

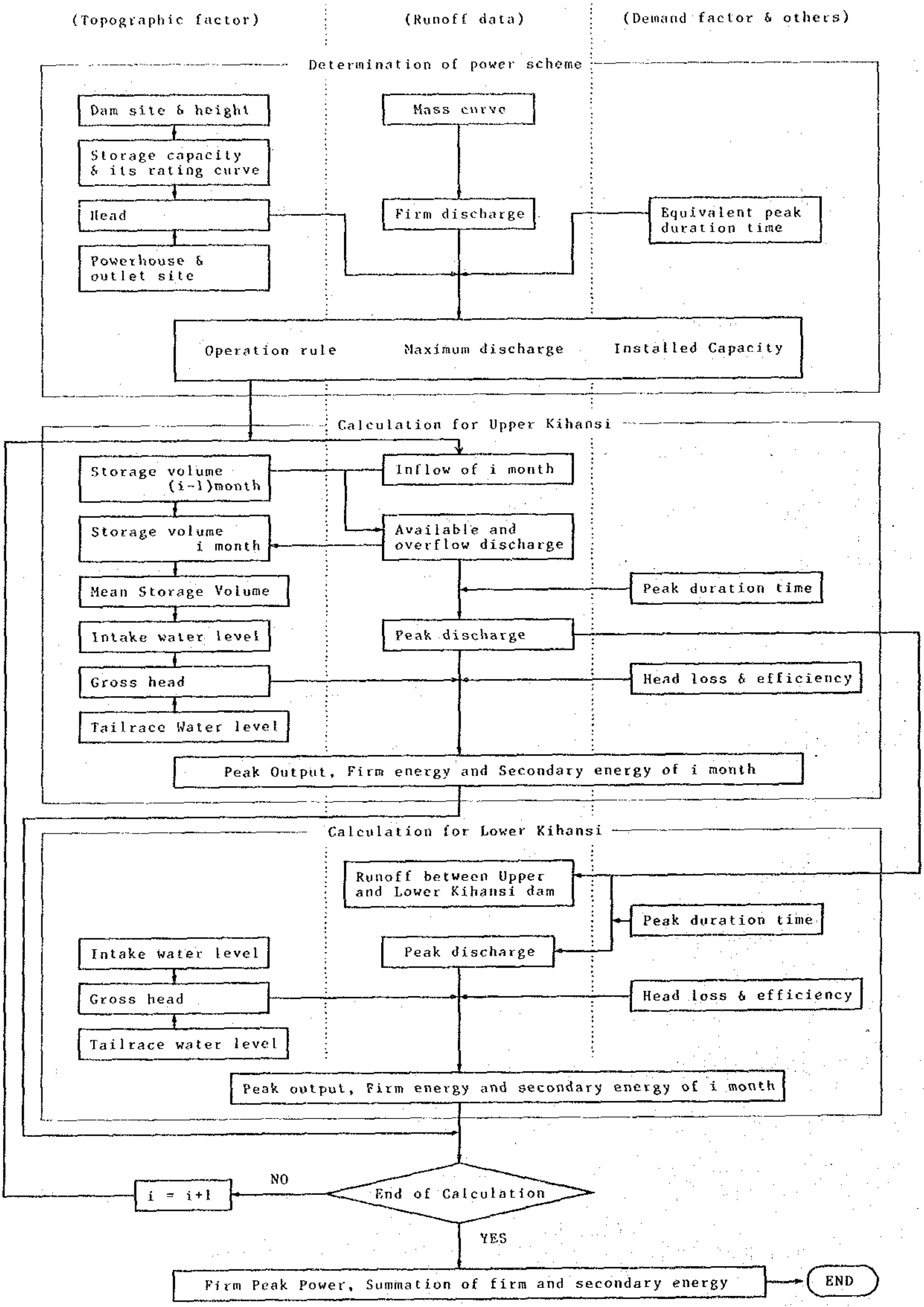
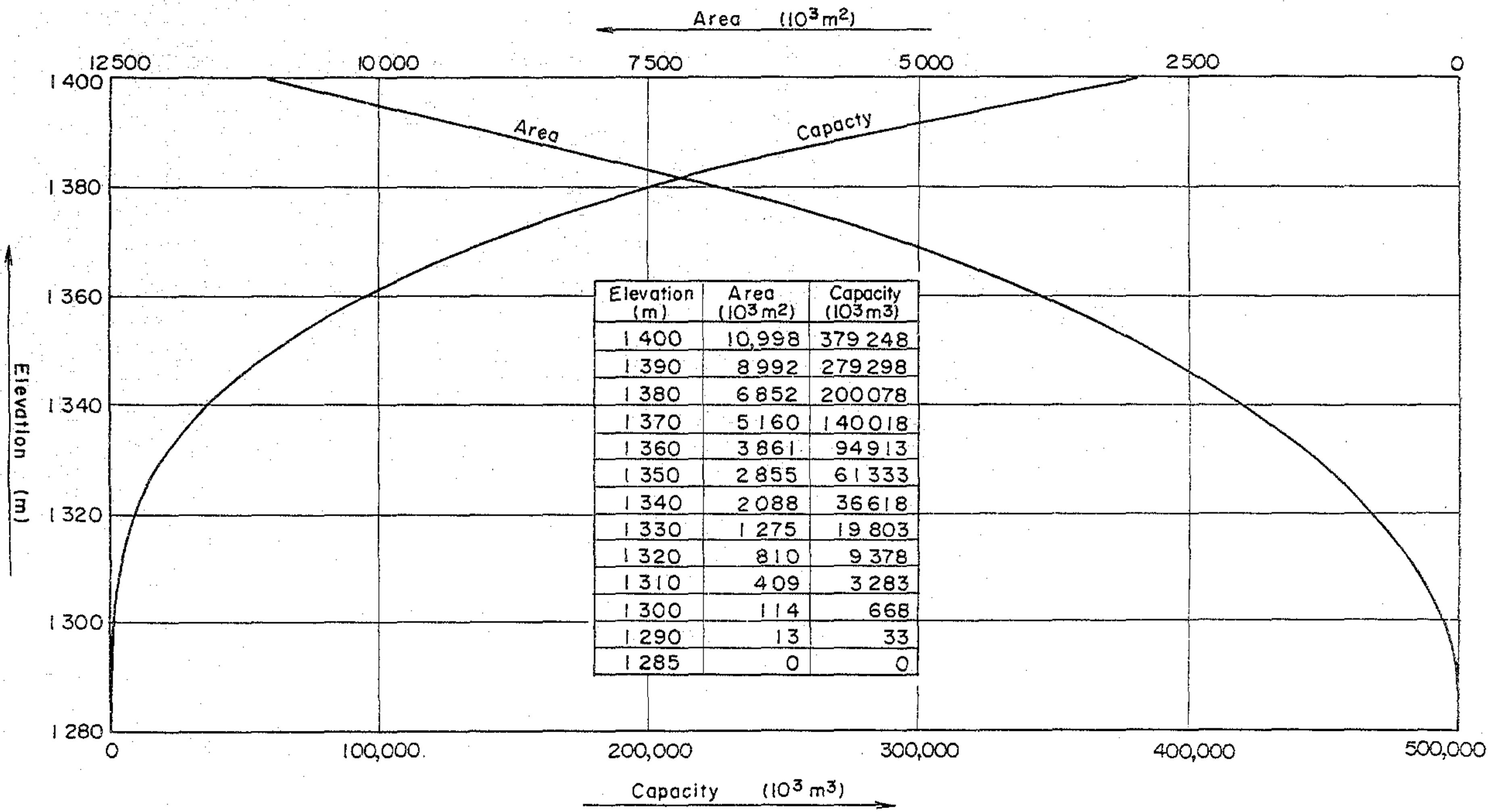
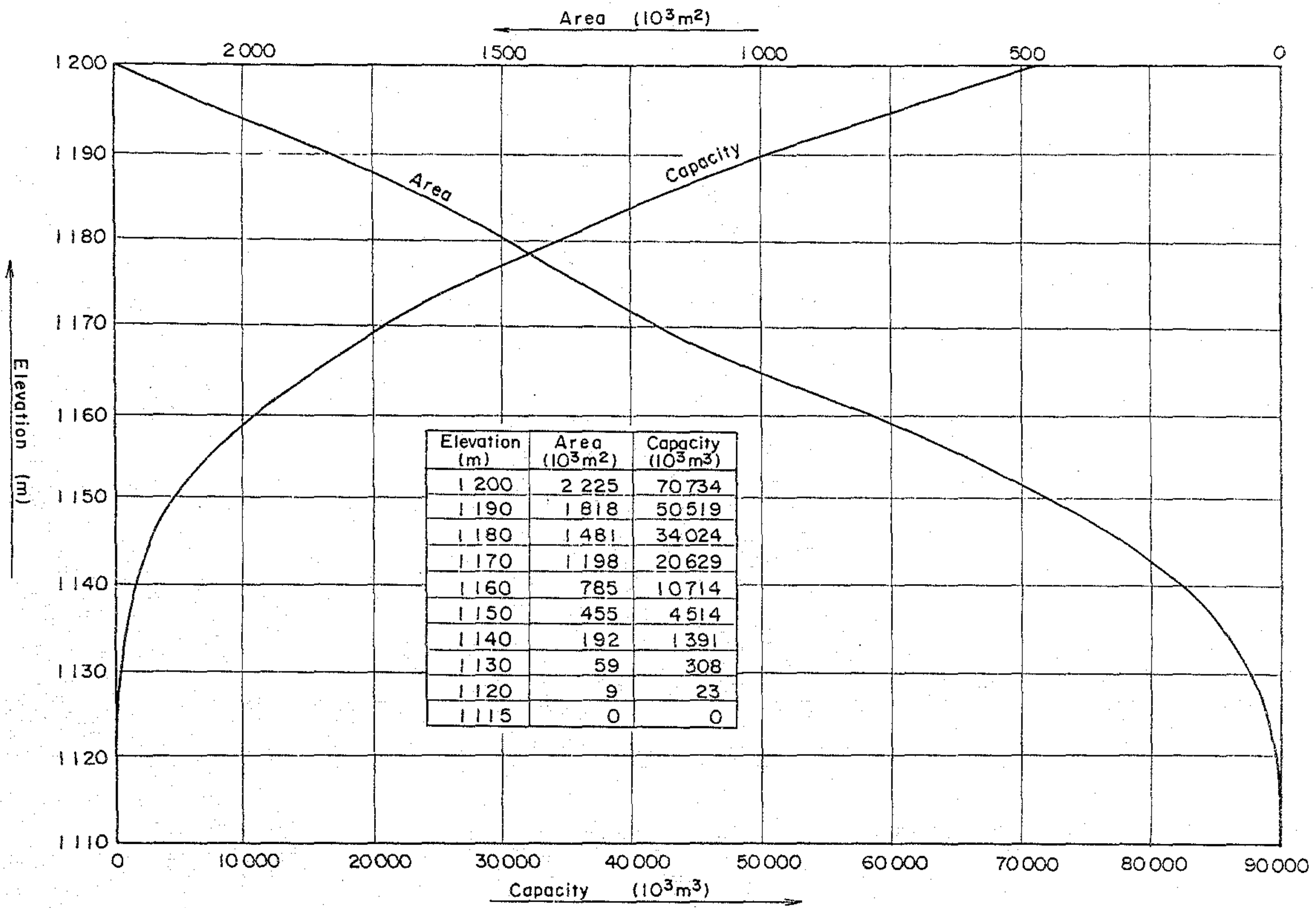


Fig. 8-7 Area Capacity Curve at Upper Kihansi Dam Site




| Elevation (m) | Area ($10^3 m^2$) | Capacity ($10^3 m^3$) |
|---|---|---|
| 1 400 | 10,998 | 379 248 |
| 1 390 | 8 992 | 279 298 |
| 1 380 | 6 852 | 200 078 |
| 1 370 | 5 160 | 140 018 |
| 1 360 | 3 861 | 94 913 |
| 1 350 | 2 855 | 61 333 |
| 1 340 | 2 088 | 36 618 |
| 1 330 | 1 275 | 19 803 |
| 1 320 | 810 | 9 378 |
| 1 310 | 409 | 3 283 |
| 1 300 | 114 | 668 |
| 1 290 | 13 | 33 |
| 1 285 | 0 | 0 |

Fig. 8-8 Area Capacity Curve at Lower Kihansi Dam Site




| Elevation (m) | Area ($10^3 m^2$) | Capacity ($10^3 m^3$) |
|---|---|---|
| 1 200 | 2 225 | 70 734 |
| 1 190 | 1 818 | 50 519 |
| 1 180 | 1 481 | 34 024 |
| 1 170 | 1 198 | 20 629 |
| 1 160 | 785 | 10 714 |
| 1 150 | 455 | 4 514 |
| 1 140 | 192 | 1 391 |
| 1 130 | 59 | 308 |
| 1 120 | 9 | 23 |
| 1 115 | 0 | 0 |

(2) Study on Reservoir Scale

A study on reservoir scale is made for the basic development scheme selected in "8.1 Comparison Study on Development Scheme".

As described in "(1) Reservoir Operation Plan" the breadth of variation in inflow to the reservoir is small seasonally, but rather large when looked at over the years.

When aiming to increase the regulating capability by increasing the effective capacity of the reservoir, it is possible to raise the stream utility factor and increase the discharge, but on the other hand, it is necessary to increase the scale of the reservoir and the construction cost becomes higher. On the other hand, if the effective capacity is to be made small, it suffices for the scale of the dam to be small and reduction in construction cost can be aimed for, but regulating capability is decreased, and along with decrease in discharge, spill-over discharge is to increase. In view of the above, it is necessary for the optimum storage capacity to be decided taking into account the runoff duration and characteristics of the reservoir. For Upper Kihansi Dam, which is a reservoir dam, the limit to the maximum high water level elevation is considered to be 1,390 m in view of the topography at the dam site and the elevation of the saddle on the right-bank side upstream of the dam site.

According to the results of analysis in "Chapter 6 Hydrology and Meteorology", sedimentation in 50 years is $750 \times 10^3$ $m^3$, but taking into consideration future increase in development in the area upstream of the reservoir, ample allowance is to be made and the dead water level of the reservoir is to be EL. 1,320 m and the dead storage capacity $9.4 \times 10^6$ $m^3$.

Since the Kihansi Project is a typical high-head, small discharge project, an increase in dam height to obtain an increase in head adversely affects the economics of the Project exceedingly, and so the low water level of the reservoir is fixed at EL. 1,330 m adding 10 m for static draft head of the intake to the dead water level. The effective storage capacity of the reservoir is studied with the five cases of available drawdown above this low water level of 60 m, 50 m, 40 m, 30 m and 20 m.

Furthermore, a study is also made on the case of providing a regulating reservoir for the Upper Kihansi Project with the purpose of daily regulation instead of providing a reservoir. In this case, the regulating reservoir capacity is made a volume at which inflow can be completely regulated.

The conditions of the comparison study are as follows:

- The optimum storage capacity is examined taking into account both the Upper Kihansi and Lower Kihansi Projects.

- Based on the study in "Chapter 5 Power Demand Forecast and Electric Power Development Plan", the plant factor in relation to firm discharge of the Lower Kihansi Project is taken to be about 55 per cent adding a reserve capacity ratio of 15 per cent to the annual load factor of 64 per cent, and the maximum discharge is selected with peak duration time as 13 hours, while for the Upper

Kihansi Project, the decrease in intake capability due to lowering reservoir water level is also taken into account in deciding the maximum discharge.

- Maximum unit capacity of turbine and generator is set at 51 MW which is the same as the existing maximum capacity in Tanzania.

- Evaluation of energy production is done considering only firm energy.

- Power Generation increase in the Lower Kihansi Project due to regulating effect at the Upper Kihansi Project is considered as all belonging to the Upper Kihansi Project.

The results of study are given in Table 8-7 and Fig. 8-9. Up to high water level elevation of 1,360 m the economics becomes better the higher the high water level, but above 1,360 m, the economics is impaired. This is because at above high water level of 1,360 m the increase in firm discharge by storage capacity increase becomes smaller, whereas because the increase in dam volume due to increase in dam height is large, the increase in cost becomes greater than the increase in benefit.

As a result of this study, the scale of Upper Kihansi Reservoir is to be decided as high water level of EL. 1,360 m, available drawdown of 30 m, and effective storage capacity of $75.1 \times 10^6$ $m^3$.

# Table 8-7 Study on Optimum Storage Capacity of Upper Kihansi Reservoir

| Item | Unit | Case 1 | | | Case 2 | | | Case 3 | | | Case 4 | | | Case 5 | | | Case 6 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total | Upper Kihansi | Lower Kihansi | Total |
| High Water Level | m | 1,390.00 | 1,140.00 | | 1,380.00 | 1,140.00 | | 1,370.00 | 1,140.00 | | 1,360.00 | 1,140.00 | | 1,350.00 | 1,140.00 | | 1,305.00 | 1,140.00 | |
| Lower Water Level | m | 1,330.00 | 1,136.50 | | 1,330.00 | 1,136.60 | | 1,330.00 | 1,136.80 | | 1,330.00 | 1,137.00 | | 1,330.00 | 1,137.10 | | 1,303.60 | 1,137.90 | |
| Available Drawdown | m | 60.00 | 3.50 | | 50.00 | 3.40 | | 40.00 | 3.20 | | 30.00 | 3.00 | | 20.00 | 2.90 | | 1.40 | 2.10 | |
| Gross Storage Capacity | $10^6 m^3$ | 279.30 | 1.39 | | 200.10 | 1.39 | | 140.00 | 1.39 | | 94.90 | 1.39 | | 61.30 | 1.39 | | 1.32 | 1.39 | |
| Effective Storage Capacity | $10^6 m^3$ | 259.50 | 0.53 | | 180.30 | 0.52 | | 120.20 | 0.51 | | 75.10 | 0.48 | | 41.50 | 0.45 | | 0.37 | 0.37 | |
| Dam Type | | Rockfill | Concrete gravity | | Rockfill | Concrete gravity | | Rockfill | Concrete gravity | | Rockfill | Concrete gravity | | Rockfill | Concrete gravity | | Rockfill | Concrete gravity | |
| Dam Height x Length | m | 125x680 | 35x177 | | 115x650 | 35x177 | | 105x620 | 35x177 | | 95x583 | 35x177 | | 85x550 | 35x177 | | 40x430 | 35x177 | |
| Dam Volume | $10^3 m^3$ | 11,500 | 54 | | 9,200 | 54 | | 7 600 | 54 | | 5,300 | 54 | | 3,200 | 54 | | 165 | 54 | |
| Maximum Discharge | $m^3/s$ | 29.00 | 24.90 | | 28.40 | 24.30 | | 27.20 | 23.40 | | 25.70 | 22.20 | | 24.20 | 20.90 | | 17.20 | 17.40 | |
| Standard Intake Water Level | m | 1,390.00 | 1,140.00 | | 1,380.00 | 1,140.00 | | 1,370.00 | 1,140.00 | | 1,360.00 | 1,140.00 | | 1,350.00 | 1,140.00 | | 1,305.00 | 1,140.00 | |
| Tail Water Level | m | 1,138.50 | 296.50 | | 1,138.50 | 296.50 | | 1,138.50 | 296.50 | | 1,138.50 | 296.50 | | 1,138.50 | 296.50 | | 1,138.50 | 296.50 | |
| Gross Head | m | 251.50 | 843.50 | | 241.50 | 843.50 | | 231.50 | 843.50 | | 221.50 | 843.50 | | 211.50 | 843.50 | | 166.50 | 843.50 | |
| Effective Head | m | 244.50 | 813.00 | | 234.50 | 813.00 | | 224.50 | 813.00 | | 214.50 | 813.00 | | 204.50 | 813.00 | | 159.50 | 813.00 | |
| Installed Capacity | MW | 61 | 171 | | 57 | 167 | | 52 | 161 | | 47 | 153 | | 42 | 144 | | 23 | 119 | |
| Firm Peak Power | MW | 110.8 | 101.8 | 212.6 | 106.0 | 101.8 | 207.8 | 97.3 | 101.8 | 199.1 | 86.1 | 101.8 | 187.9 | 71.8 | 101.8 | 173.6 | 19.7 | 101.8 | 121.5 |
| Annual Energy Production | $10^6$ kWh | 316.6 | 890.4 | 1,207.0 | 306.1 | 886.6 | 1,192.7 | 293.0 | 880.1 | 1,173.1 | 275.1 | 868.9 | 1,144.0 | 257.8 | 854.8 | 1,112.6 | 151.1 | 799.5 | 950.6 |
| Annual Firm Energy Production | 106 kWh | 475.5 | 551.0 | 1,026.5 | 441.5 | 551.0 | 992.5 | 393.3 | 551.0 | 944.3 | 335.7 | 551.0 | 886.7 | 272.8 | 551.0 | 823.8 | 106.8 | 551.0 | 657.8 |
| Annual Benefit | | | | | | | | | | | | | | | | | | | |
| Peak Power Benefit | $10^3$ US$ | 29,042 | 26,683 | 55,725 | 27,784 | 26,683 | 54,467 | 25,503 | 26,683 | 52,186 | 22,568 | 26,683 | 49,251 | 18,819 | 26,683 | 45,502 | 5,164 | 26,683 | 31,847 |
| Firm Energy Benefit | $10^3$ US$ | 16,341 | 18,936 | 35,277 | 15,173 | 18,936 | 34,109 | 13,516 | 18,936 | 32,452 | 11,537 | 18,936 | 30,473 | 9,375 | 18,936 | 28,311 | 3,586 | 18,501 | 22,087 |
| Total Benefit (B) | $10^3$ US$ | 45,383 | 45,619 | 91,002 | 42,957 | 45,619 | 88,576 | 39,019 | 45,619 | 84,638 | 34,105 | 45,619 | 79,724 | 28,194 | 45,519 | 73,813 | 8,750 | 45,184 | 53,934 |
| Investment Cost | | | | | | | | | | | | | | | | | | | |
| Civil Facilities Cost | $10^6$ US$ | 446.2 | 128.4 | 574.6 | 365.5 | 126.8 | 492.3 | 296.5 | 125.0 | 421.5 | 239.5 | 119.9 | 359.4 | 199.2 | 118.3 | 317.5 | 64.9 | 111.8 | 176.7 |
| Electro-Mechanical Facilities Cost | $10^6$ US$ | 30.2 | 98.8 | 129.0 | 28.9 | 97.9 | 126.8 | 27.0 | 96.3 | 123.3 | 21.5 | 86.4 | 107.9 | 20.1 | 85.3 | 105.4 | 13.3 | 78.9 | 92.2 |
| Total | $10^6$ US$ | 476.4 | 227.2 | 703.6 | 394.4 | 224.7 | 619.1 | 323.5 | 221.3 | 544.8 | 261.0 | 206.3 | 467.3 | 219.3 | 203.6 | 422.9 | 78.2 | 190.7 | 268.9 |
| Annual Cost | | | | | | | | | | | | | | | | | | | |
| Civil Facilities Cost | $10^3$ US$ | 47,253 | 13,598 | 60,851 | 38,706 | 13,428 | 52,134 | 31,399 | 13,238 | 44,637 | 25,363 | 12,697 | 38,060 | 21,095 | 12,528 | 33,623 | 6,873 | 11,840 | 18,718 |
| Electro-Mechanical Facilities Cost | $10^3$ US$ | 3,585 | 11,728 | 15,313 | 3,430 | 11,621 | 15,051 | 3,205 | 11,431 | 14,636 | 2,552 | 10,256 | 12,808 | 2,386 | 10,125 | 12,511 | 1,579 | 9,365 | 10,944 |
| Total Cost (C) | $10^3$ US$ | 50,838 | 25,326 | 76,164 | 42,136 | 25,049 | 67,185 | 34,604 | 24,669 | 59,273 | 27,915 | 22,953 | 50,868 | 23,481 | 22,653 | 46,134 | 8,452 | 21,205 | 29,657 |
| Annual Surplus Benefit (B - C) | $10^3$ US$ | -5,455 | 20,293 | 14,838 | 821 | 20,570 | 21,391 | 4,415 | 20,950 | 25,365 | 6,190 | 22,666 | 28,856 | 4,713 | 22,966 | 27,679 | 298 | 23,979 | 24,277 |
| Benefit Cost Ratio (B/C) | | 0.893 | 1.801 | 1.195 | 1.019 | 1.821 | 1.318 | 1.128 | 1.849 | 1.428 | 1.221 | 1.987 | 1.567 | 1.201 | 2.014 | 1.600 | 1.035 | 2.131 | 1.819 |
| Unit Annual Cost | US$/kWh | 0.107 | 0.046 | 0.074 | 0.102 | 0.045 | 0.068 | 0.093 | 0.045 | 0.063 | 0.083 | 0.042 | 0.057 | 0.086 | 0.041 | 0.056 | 0.083 | 0.038 | 0.045 |

Fig. 8-9 Study on Optimum Storage Capacity of Upper Kihansi Reservoir