CHAPTER VI DESIGN OF ROAD

1. ROAD DESIGN

1.1 General

The design section of the Project road is divided into two sections, generally rolling section between Bereina and Palipalla Hill, and the flat section between Palipalla Hill and Malalaua. The route in the rolling section passes through the foot of Inapi Ridge and Palipalla Hill. Savanna is prevalent in hill sections, while valley sections are mostly covered by dense jungle. The Alika swamp is located on the right bank of the Miaru river and is always swampy even in dry season. The route in flat section crosses the estuaries of the Kapuri, Lakekamu and Tauri rivers. The vicinity of the Kapuri river is perennially under water.

The location of the project road is shown in Figure 6-1. As shown in the figure, nearly one third of the design section is through swamp and there is no access road to the project site except near Bereina and Malalaua. A helicopter was used to reach the site and field reconnaissance was done by treking after landing on an appropriate open area.

Since field survey for the route alignment and cross sections could not be done readily due to inaccessibility, the horizontal and vertical alignments were determined from topographical maps at a scale of 1:1000 which were made by photogrammetry as discussed in Chapter III. Supplemental field surveys were conducted for bridge sites where more accurate topographical maps were required and for areas covered by dense vegetation where the accuracy of photogrammetry was not assured.

The design works were carried out based on the following concepts.

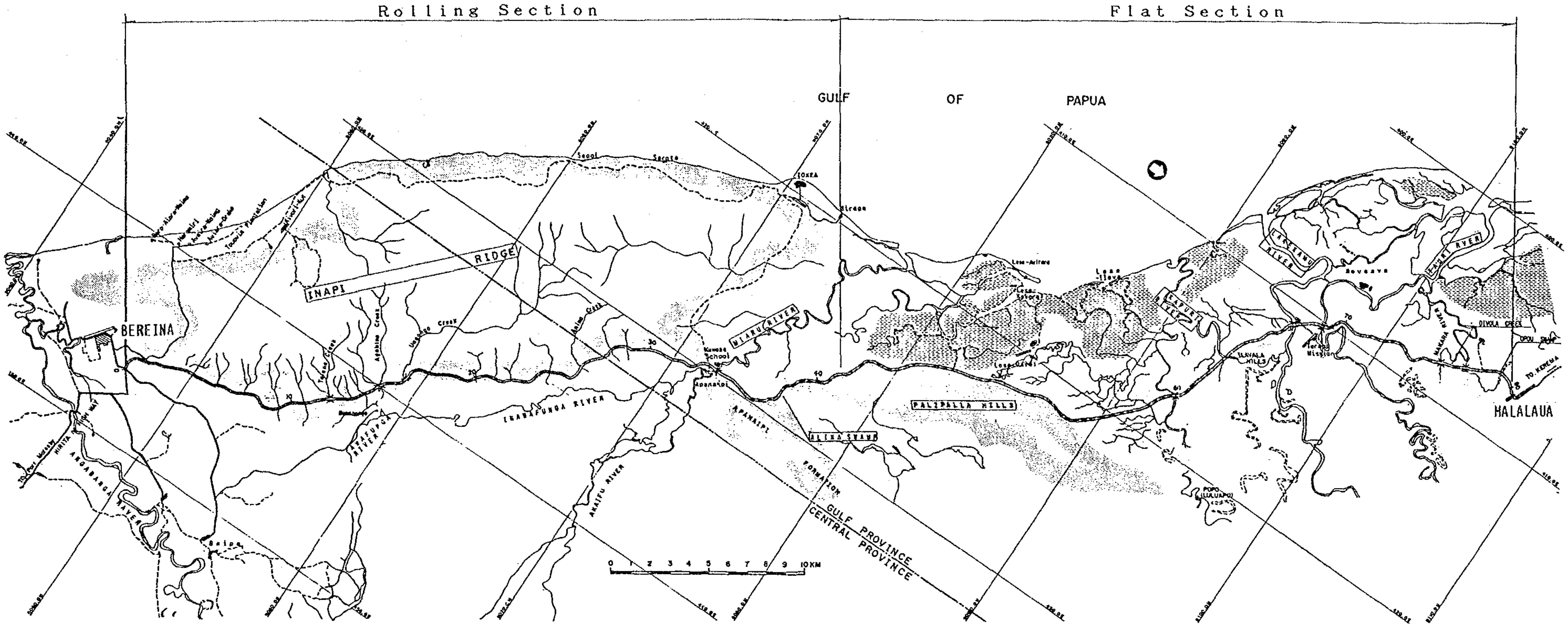


Fig. 6-1 LOCATION MAP

(1) As the project road is a part of trunk system to connect the capital city Port Moresby with the second largest city Lae its structure shall be all-weather type passable throughout the year.

(2) The route shall be based on the preliminary design conducted by the Cardno & Davies Study in 1982.

(3) In the rolling section, cut and fill volumes for the earth works, and the haul distance to balance cut and fill volumes shall be as small as possible to minimize construction cost.

In the level section, the embankment height shall be determined from prospected flood level estimated from hydrological and hydraulic analyses as described in Chapter V. The location of borrow pits for embankment material shall be located as near as possible to the project road. Side borrow shall not be considered anywhere.

(4) The procedures to determine the filling structure on soft ground in swamp area are discussed in this chapter taking into account ease of construction and maintenance.

1.2 Geometric Design Standard

The design of the road was based on the Rural Road Design Manual of DOW issued in April 1985. Items not covered in the manual were taken from standards widely adopted in Japan and other countries.

The geometric design standards employed in this project are listed in Table 6-1. Detailed discussion to determine design standards is presented in Appendex.
Standard lane configurations for embankment and bridge section are shown in Figure 6-2.

Table 6-1 GEOMETRIC STANDARDS FOR THE PROJECT ROAD (1)

ITEM	UNIT	DESIGN STANDARD	* REMARKS
1. Design Speed (Sealed Pavement)	kph	70	Terrain Type : Flat and Rolling Classification : Medium (Traffic Volume 100 - 400 vpd)
2. Formation Width	m	8.5	
Carriage Way	m	6.5	
Shoulder	m	1.0	
3. Sight Distance			
Stopping Distance	m	90	Table 4.7.1
Intermediate	m	180	-do-
Overtaking	m	350*	-do- * 2 to 2.5 km intervals are desirable between overtaking zones
4. Horizontal Curve			
Desirable Radius	m	400	Table 7.5.2 (60 km/hr)
Minimum Radius	m	155	Table 2.1 Based on supererevation "e" of 0.10
Minimum Length	m	120	Table 5.12
Minimum Deflection Angle			
	Degrees	4.0	S'elevation is needed Table 5.12
	Degrees	1.5	Curve is needed Table 5.12
5. Cross Fall			
Bitumen Sealed	%	3.0	
Gravel Shoulder	%	4.0	
6. Maximum Superelevation	%	10.0	

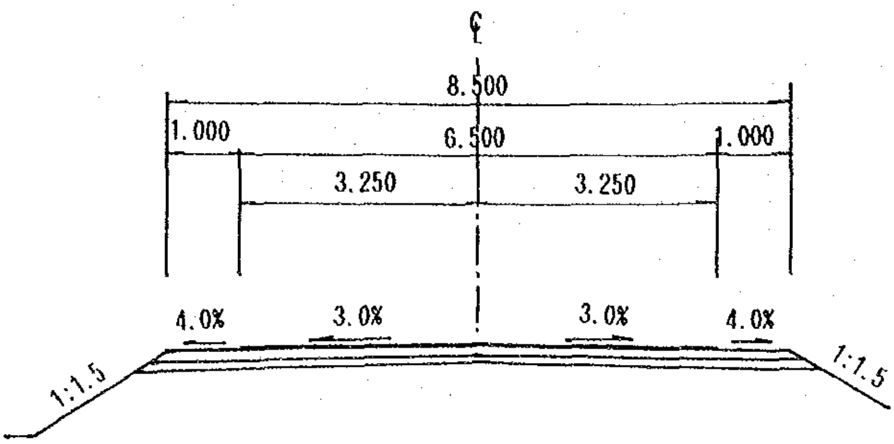
*Note : The tables and figures noted in the REMARKS refer to Rural Road Design Manual of DOW

Table 6-1 GEOMETRIC STANDARDS FOR THE PROJECT ROAD (2)

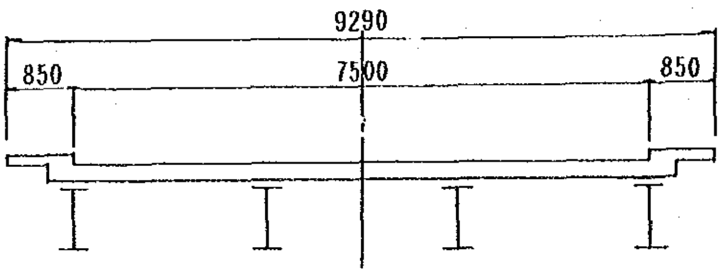
ITEM	UNIT	DESIGN STANDARD	* REMARKS
7. Minimum Radius of Horizontal Curve having Adverse Crossfall not exceeding 0.03 sealed	m	1000	Table 5.9
8. Superelevation Run-off Maximum Relative Grade	%	0.55 (1/182)	Table 5.7
9. Gradient			
General Maximum	%	6	Table 2.1
Absolute maximum (Length)	% (m)	8 (700)	-do-
10. Vertical Curves			
Minimum Radius of Crest Curve	m	1755	Fig. 6.4.1 Stopping Sight Distance
Minimum Radius of Sag Curve	m	1740	Fig. 6.5 Headlight Stopping Distnce
Minimum Length of Crest Curve	m	90	Fig. 6.4.1
Minimum Length of Sag Curve	m	90	Fig. 6.5
11. Maximum Combined Gradient	%	10.5	Japanese standard
12. Minimum Vertical Clearance	m	5.0	Bridge Design Standard of DOW

* Note : The tables and figures noted in the REMARKS refer to Rural Road Design Manual of DOW

Embankment



Two Way Bridge (L>40m)



Single Lane Bridge (L<40m)

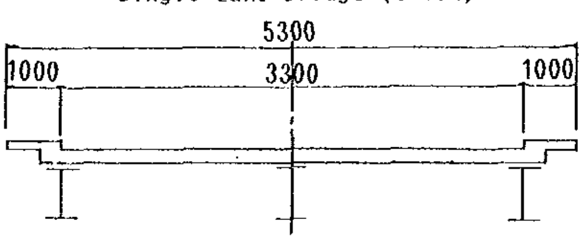


Fig. 6-2 STANDARD LANE CONFIGURATION

1.3 Horizontal and Vertical Alignment

1.3.1 Horizontal alignment

The horizontal alignment of project road was determined through field reconnaissance and geological investigations based on the route selected in the Cardno & Davies Study. The initial route proposed by the Cardno & Davies study and other alternative routes selected in this study were first drawn on the manuscript map made by photogrammetry at a scale of 1:1000. A comparative study was then done in consideration of soft ground, bridge location and flood area. More extensive field reconnaissance was conducted when necessary for selection of suitable alignment. The final alignment for the whole project area was determined using modified manuscript maps after the supplementary field surveys of jungle areas and bridge site.

As the result of the above design works, the total project road length is computed at 80.596 km consisting of earthworks of 80.052 km (99.3%) and bridging of 0.544 km (0.7%). The first 41 km from Bereina is in rolling terrain and the remaining 39 km to Malalalua in flat.

The minimum curve radius applied in the rolling terrain is 200m, the minimum straight length on S-curving portion bends being 86m, mostly between 300m and 700m. The minimum curve radius of 240m is applied on the flat, the minimum straight length being 92m, and the general curve radius applied being between 500m and 1,000m.

The co-ordinates of the road centerline were computed based on the finalized route, and are submitted as a part of Tender Documents.

1.3.2 Vertical alignment

Vertical alignment was designed in consistency with conditions of horizontal alignment discussed in 1.3.1.

Trial designs were made using the Highway Computer Aided Design (HI-CAD) system to obtain the best fit alignment to satisfy conditions that of balanced cut and fill in rolling sections, and lowest possible embankment height above flood level in flat sections. The HI-CAD system worked quite efficiently to find the shortest longitudinal haul distance in rolling sections.

Since design conditions vary between the rolling and flat or level section, design methods for each section will be described separately.

(1) Repeat design by CAD system for rolling sections

Vertical alignment of the rolling section is designed to fit the main control points mentioned below:

- Top height of drainage pipes + coverage of 0.6m
- Q50 flood levels in creeks + coverage of 0.3m
- Bridge slab height planned
- Not to exceed the embankment height of 7.0m
- Not to exceed the cutting depth of 14.0m
- Minimization of earthworks in volume and haul distance.

As mentioned above, to minimize the earthwork volume, repeat design by HI-CAD system was adopted in consideration of following engineering points.

- To achieve a balance of cut and fill volume within work sections shorter than 500m in the mass curve.
- The cutting earth volume to be compared with compacted fill volume by multiplying by the following coversion factors:

Type D (common)	0.85
Type B (ripping soils)	0.95
Type A (solid rock)	1.05

As a result of the alignment design, the maximum vertical gradient was found to be 6.0% while minimum vertical radius was 1,800m in both crest and sag sections.

(2) Lowest road level in swamp

The vertical alignment of the flat section is designed to conform to the main control points mentioned below:

- Top height of drainage pipe + coverage of 0.6m
- Q50 flood levels in swamps + coverage of 0.3m
- Bridge slab height

The flood levels of the rivers are discussed in Chapter V.

As a result of the alignment design, the maximum longitudinal slope will be 3.1% while minimum vertical radii were 3,200m and 2,700m at crest and sag section respectively. A longitudinal section (profile) at a scale of 1:1000 in the horizontal direction and 1:100 in the vertical direction was drawn under the plan figure to provide an overview of the project road.

1.4 Cross Section

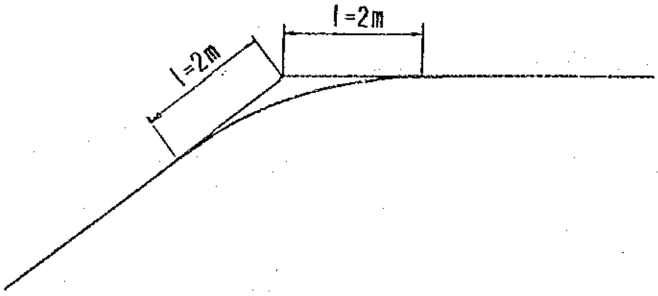
Basic design concepts regarding cross sectional segments such as slope and side ditches are discussed below:

(1) Embankment slope

Sideslopes are proposed as 1:1.5 from stability analysis up to a maximum height of 7m. The same slope is proposed for the soft ground section after confirming the stability as discussed below.

(2) Cutting slope

1:1.0 slope is proposed based on geological investigation and stability analysis. A 2m wide berm will be constructed every 5m in height.
Rounding at the top corner of cutting slope is proposed at 2m of each tangent line.

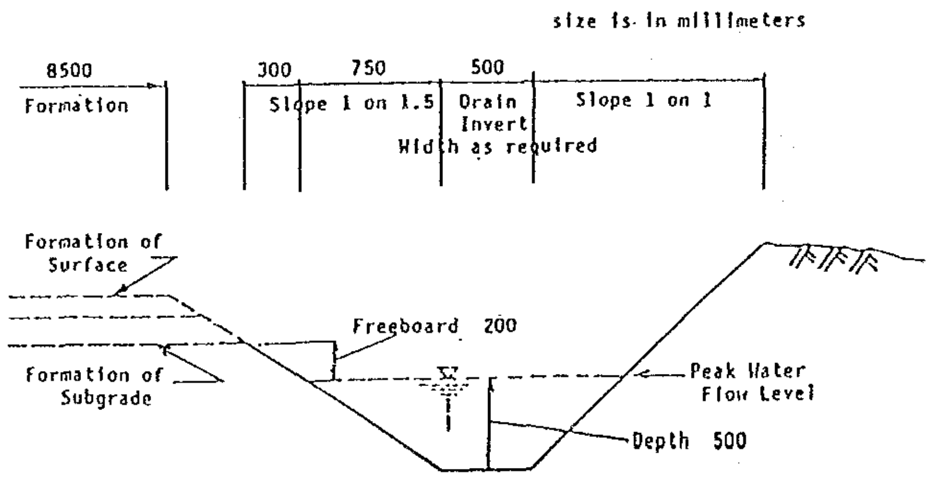


(3) Slope protection

From the experience in past projects in PNG, plants are expected to grow fast on slopes after construction, hence seeding is omitted, However, in parts of the Alika swamp, installation of Reno mat is proposed at the toe of slope to prevent washout.

(4) Side ditches (Cut-off drains)

Side ditches are planned to be constructed outside the shoulders in cutting sections as illustrated below:



(5) Pavement thickness

The design thickness for pavement is 39cm. The detailed procedure to arrive at the design thickness is discussed in Chapter Ⅷ.

(6) Typical cross section

Typical cross sections for cut and fill, and for soft ground are shown in Figure 6-3 (1) & (2).
The road width varies only in the transition section from two lanes to single lane at bridge approaches and over a length of more than 30m.

(7) Drawings of cross section

Drawings of cross section were made using HI-CAD System at a scale of 1:100 at an average interval of 50m, an interval of 25m was used for the rolling section while 100m was adopted for the flat or level section. The earthwork volume was then computed based on these drawings.

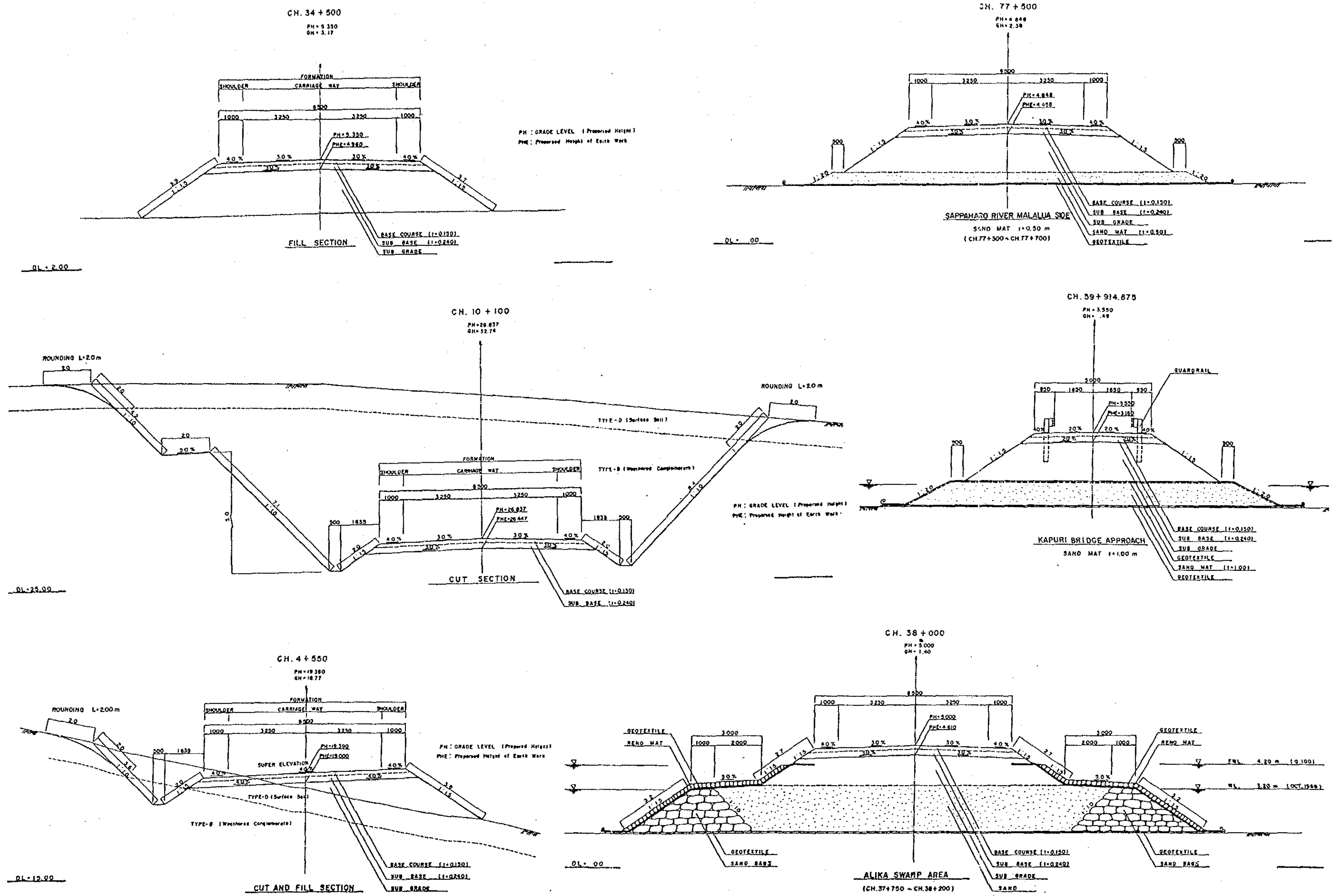


Fig. 6-3 TYPICAL CROSS SECTIONS OF CUT & FILL AND SOFT GROUND

1.5 Earthwork Volumes Computed

After the finalization of the alignment and cross sections, the earthwork volumes were computed by soil categories and construction lots.

Construction will be divided into two lots, lot I is from Bereina to Miara river (33.5km), lot II is from Miara river to Malalaua (47km). The division of work is discussed in chapter XI.

With reference to the computed earthwork volumes, the following are pointed out:

- In lot I, the earthworks are almost balanced except at the final cutting site of about 16,000m^3 where 9,000m^3 will be transferred to lot II
- In lot II in the flat section, embankment material will be taken from five borrow pits near the project road to make up 350,000m^3 deficiency which includes loss due to settlement.
  The settlement volume will amount to 42,000m^3 in addition to the sand mat volume of 180,000 m^3.

1.6 Intersections.

1.6.1 Locations

The project road intersects existing roads at the following three locations.

a. The CH 0+200 near the beginning point of the project road at Bereina to connect with Hiritano highway and the road for Beipa.

b. The CH 1+450 near Bereina to connect with the road connecting Bereina and Iokea.

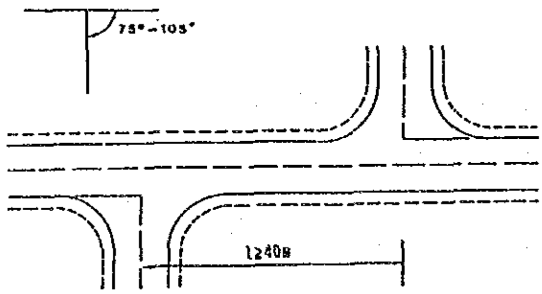
c. The CH 33+400 near Apanaipi to connect with the road from Iokea.

Preliminary design for general overview of the above intersections including approaches was done using the 1:1000 map. Detailed design for entrances and exits of intersections used 1:400 topographical maps.

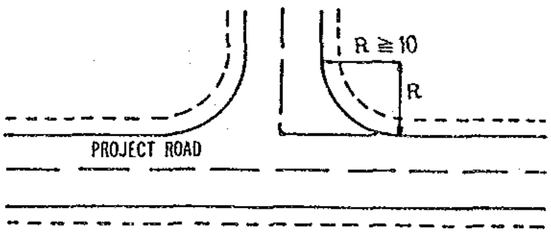
1.6.2 Details

General standards are taken from the Road Design Manual of DOW. The following are primary standards applied to the Project.

1) Intersection angle will be between 75-105 degrees. Staggered intersections were used for the intersections mentioned above to avoid accidents caused by ignorance of stop and yeild regulations which are introduced to right angle intersections. The distance between staggered intersections shall be not less than 40m.



2) Radius of corners shall be more than 10m.



3) Lane width including shoulder at legs of intersection shall be 8.0m.

4) The vertical gradient inside an intersection shall be less than 3%.

5) The approach section shall also be paved and an edge line shall be drawn on outer-edge of roadway.

Plans of the three intersections with existing roads are shown in Figure 6-4*.

In addition to the main intersections mentioned above, intersections with small tracks near villages were also planned. The pavement width of project road is kept constant over whole part of intersection while no pavement is laid for the approaches. Locations of these intersections are as follows :

CH	14+200	near Babanongo
CH	34+000	near Apanaipi after passing Miaru river
CH	49+400	near Lese Oarai
CH	54+100	near Popo
CH	67+600	near Terapo Mission
CH	68+500	near Moveave
CH	78+200	near Tapaka

1.7 Road Furniture and Marking

Road furniture and markings were designed for traffic control and management, in axxordance with the Design Manual of DOW and Australian Standards.

1.7.1 Road signs

The following road signs are adopted based on the Australian standard 1742-1986, "Manual of Uniform Traffic Control Devices".

(1) Guide sign

- Intersection Direction Board

  Intersection direction board on the project road for three major intersections with existing road will be installed at 80m ahead of the intersection.

- Finger board

Two types of finger board are installed at the shoulders of the project road near intersections with small tracks as well as of the above mentioned three main intersections.

(2) Regulation sign

- Give way sign

Give way signs will be placed on shoulders on small tracks in front of all intersections including the three major intersections. The Give way sign is also placed beside the town bound lane in front of a single lane bridge.

- No overtaking or passing sign

No overtaking or passing signs will be installed on the opposite lane to the town bound lane in front of single lane bridge.

- School sign

School signs will be erected on roads connected to schools as well as on the project road near intersections with the roads leading to schools.

(3) Warning sign

- Give way ahead

A give way ahead sign will be installed before the Give way sign on town bound lane at a single lane bridge.

- One lane bridge sign

At a single lane bridge, the Narrow bridge sign (W4-1) and One lane sign (W8-16) will be installed on both town bound lane and its opposite lane before the Give way ahead sign and No overtaking or passing sign respectively.

1.7.2 Road edge guide post

Road edge guide posts will be erected at the following locations for visual guidance and to reduce maintenance works in accordance with rural Roads Design Manual of DOW.

(1) Curves

i Inside of curves	To be placed at tangent points (TP)
ii Outside of curves	To be placed at tangent points, at even intervals around the curves, and at approaches to curves.
(2) Pipe culverts	At inlets and outlets of pipe outlets.
(3) Single lane bridges	In pairs at: commencement of taper in width at formation and 10m before that.

1.7.3 Guard rail

Guard rails will be placed at the approaches to bridges to avoid collision with bridge-railing. The length of the guard rail will be 12m for a two lane bridge and 28m for a single lane bridge in compliance with Rural Roads Design Manual of DOW.

1.7.4 Markings

The following types of marking taken from Australian Standard 1742-1986, "Manual of Uniform Traffic Control Devices", are planned :

- <u>Separation lines</u> of 3.0m length and 100mm width will be marked on the road centerline at intervals of 9.0m

- Barrier lines will be marked at the approach section to the one lane bridge and at no over-taking sections.

- Edge lines on the both shoulders will be marked throughout with 100mm width.

- Holding lines, continuing line will be marked continuously to show intersections.

2. SETTLEMENT AND STABILITY OF EMBANKMENTS

2.1 Definition of Soft Ground

Based on the general experience of JICA study team, the following simple criteria for highway embankments on soft ground are given to identify the questionable soft grounds:

General Criteria on Soft Ground

Soils	Peat	Clay	Sand
N values by SPT	less than 4	less than 4	less than 10
Unconfined Compression Strength*qu kg/cm2	less than 0.5	less than 0.5	less than 0.5
Cone Coefficient by Dutch Cone Test: qu kg/cm2	Not exceeding 10	Not exceeding 5	Not exceeding 40

2.2 Outline of Soft Ground

Referring to all geological data surveyed by the JICA study team in 1988 and the Cardno &Davies study in 1982, the soil profile for the Project road was examined as described in Chapter III. A total of about 14.0km of soft ground was identified in thirteen (13) sections as shown in Fig. 6-5 and Table below:

Section	Location	Chainage	Length (m)	Embankment Height (m)	Boring No.
MI (a)	Miaru	33+550 - 34+150	506	3.70	B-6
AL (a)	Alika	37+750 - 38+200	450	3.60	B-8
KA (a)		57+100 - 59+800	2700	2.50	S-2
KA (b)	Kapuri	59+800 - 60+100	231	3.00	B-9
KA (c)		60+100 - 62+400	2300	2.50	S-3
KA (d)		62+400 - 64+000	1600	2.50	S-4
LA (a)	Lakekamu	67+100 - 67+500	278	4.00	B-11
LA (b)		68+200 - 68+550	350	1.70	B-14
TA (a)		67+500 - 68+200	700	2.00	B-11
TA (b)	Tauri	68+850 - 68+850	178	4.40	B-14
TA (c)		68+000 - 69+000	150	1.70	B-14
MA (a)	Makara	73+100 - 76+300	3256	2.20	B-16
SA (a)	Sappaharo	76+300 - 77+700	1357	2.20	B-17
			14,056		

The soft ground sections in Bereina to Miaru river section were judged to be no great problem due to over consolidated clay, the intended boring S_1 for swamp area was omitted.

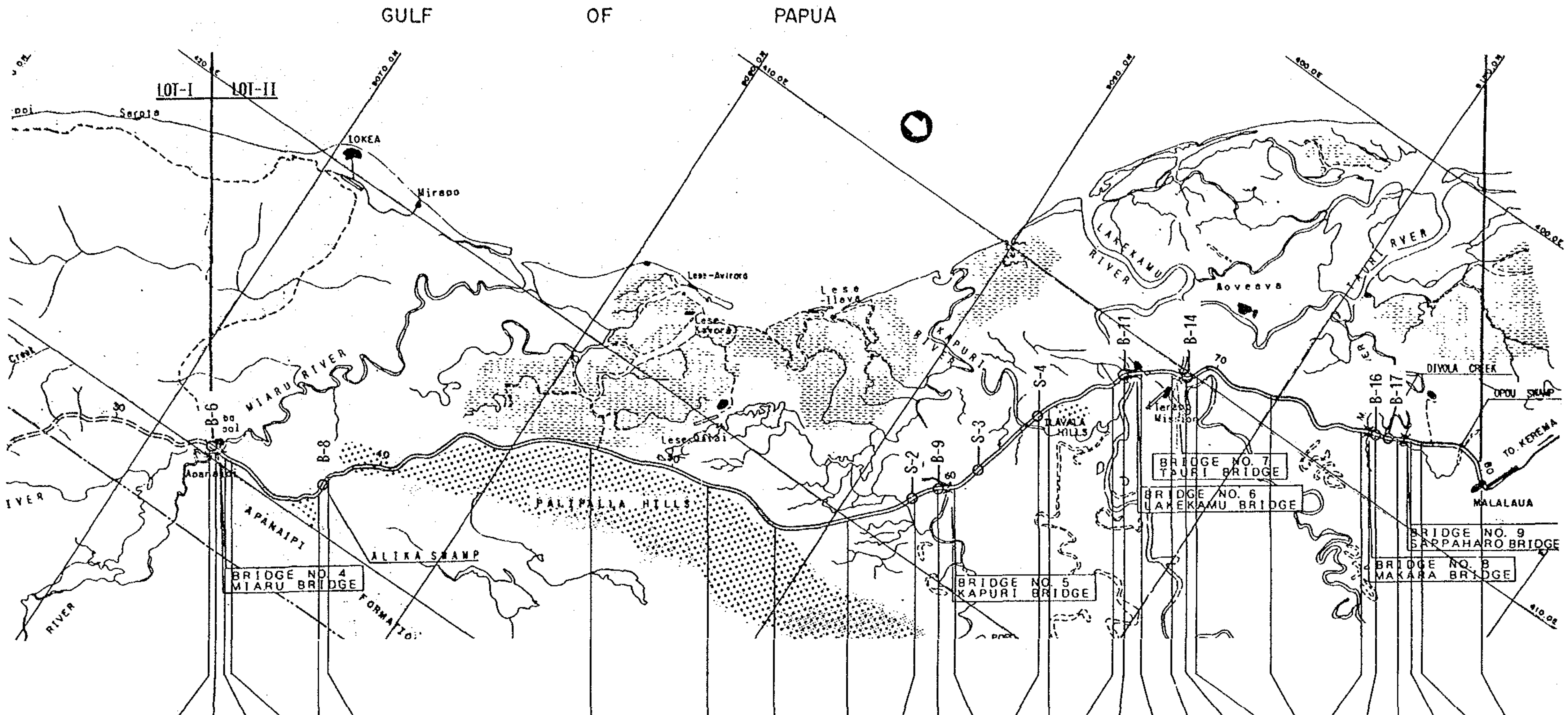
Outlines of the thirteen (13) soft ground sections located in Miaru to Malalaua section may be summarized as follows (See Figure 6-5):

a) Section MI(a) near Miaru river

The endless deep soft ground with loose silty sand was recognized by drilling to 40 meters. The N values did not exceed 30 in sand and 5 in sandy silt. However, no clayey soil was found.

b) Section AL (a) at Alika swamp

The top 1.8 meters is brown peat, the next 17 meters is silty clay including a 2.5 meter-thick sand layer in the upper portion. There is a hard silty sand layer at the bottom of the swamp with a N value of more than 40.



No. of Section for Calculation	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27
Soft Ground	MIARU				ALIKA					KAPURI						LAKEKAMU		TAURI				MAKARA			SAPPAHARO		
No. of Cross Section for Settlement Analysis	MI-(a)				AL-(a)					KA-(a)	KA-(b)		KA-(c)	KA-(d)		LA-(a)	LA-(b)	TA-(a)	TA-(b)	TA-(c)		MA-(a)			SA-(a)		
CHAINAGE (33+500; 33+550)			34+150	37+750	38+200				57+100	59+800	60+100		62+400	64+000	67+100	67+500	68+200	68+550	68 850	69+000	73+000		76+300			77+700	80+596
Length	506m				450m					2700m	231m		2300m	1600m		278m	700m	350m	178m	150m		3256m			1357m		

Fig. 6-6 LOCATION OF QUESTIONABLE SOFT GROUNDS

c) Section KA(a) - KA (c) from Palipala hills side to Kapuri river

There is no great problem in general, since the ground consists of thin clayey sand and silty or sandy clay.

d) Section KA (d) from Kapuri river to Ilavala hill

There is no great problem here either, since the ground consists of silty sand below and of silty clay at less than 3.5 meters.
Old river courses are sporadically seen in this section with some organic material but the soils below water level are not different from others in this section.

e) Section LA (a) - TA (c) Terapo area including Lakekamu river to Tauri river

There is no great problem in general, since there is a rocky layer at a depth of more than 4 meters below upper soils of sandy clay.

f) Section MA(a), SA(a) at Makara and Sappaharo river

The section near Makara river must be carefully treated, since the ground consists of a clay layer deeper than 20 meters. The N values of the clay are generally not less than 4 which will make a stable embankment possible in this section. An adjacent spot (200m) to the abutment of Tauri river bridge has some soft ground, but shallow.

2.3 Basis of Design for Soft Ground

The road filling heights on the soft ground were determined with reference to proposed road level.
The basic data for the analysis are given as follows:

- Sub soil conditions are respectively given by drilling logs with test results.

- The height to be embanked at low speed on the soft ground will be controlled to less than the 10cm per day.

- The embankment materials have following properties according to their sources:

Source of material	Unit Weight t/m^3	Cohesion tf/m^2	Internal Friction of Angle (degrees)
Bereina-Miaru	1.8	2.8	28
Palipala hills	1.8	2.8	28
Ilavala hill	1.8	5.2	31.5
Malalaua hills*	(1.8)	(5.2)	(31.5)

- The triaxial compression test for the fill material of Malalaua hills was not done yet, however, it is assumed for the analysis that a material like Ilavala hill would be available beneath the top soils of the hills.

- The safety factor against sliding shall be more than 1.2, and the allowable residual settlement upon the completion of embankment shall be generally less than 30cm.

- The soil properties applied to calculation for soft ground analysis are shown in the Attachment - 2 of Appendix.

2.4 Settlement Analysis

2.4.1 Total settlement and time-settlement curve

The total settlement of the soft ground is conceptionally presented as following equation;

$$S = Si + Sp + Ss \quad \cdots\cdots\cdots \quad (1)$$

where,

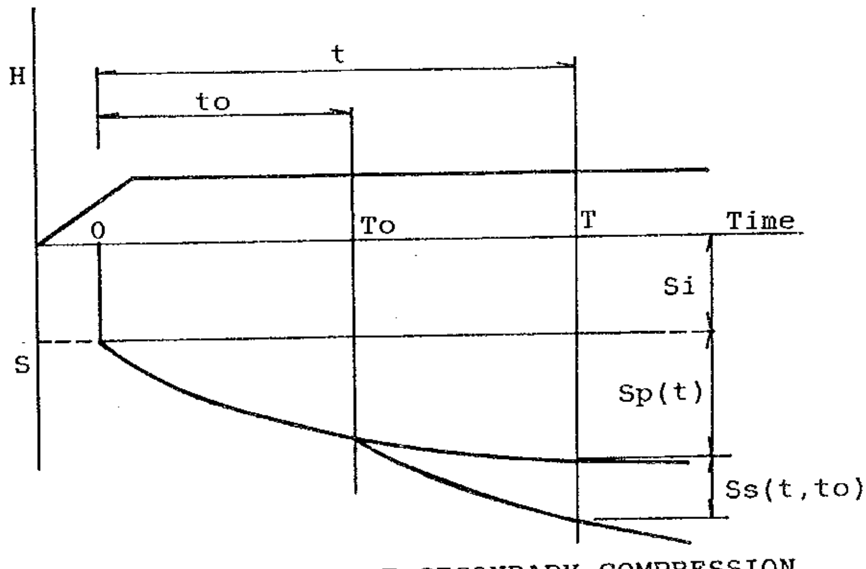
S ; total settlement
Si ; instant settlement
Sp ; settlement caused by primary consolidation
Ss ; settlement caused by secondary compression

Adoption of a time function to the equation (1) gives the equation (2) shown as follows:

$$S(t) = Si + Sp(t) + Ss\,(t_0, t) \quad \cdots\cdots\cdots \quad (2)$$

t_0 ; the time when secondary compression will start

Equation (2) is illustrated below:



STARTING TIME OF SECONDARY COMPRESSION

Each element in equation (2) is calculated as follows:

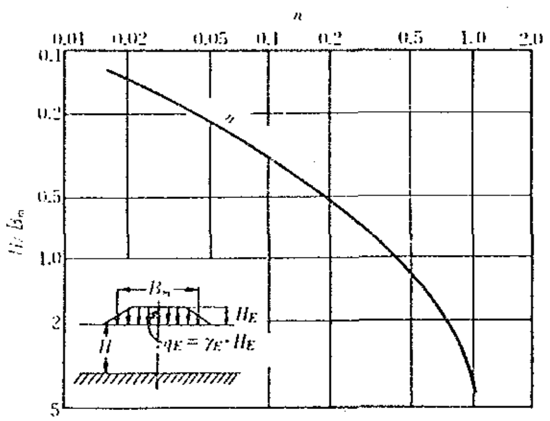
1) Si ; instant settlement

For silty or clayey soil

$$Si = \frac{qE \cdot Bm}{E} \cdot n \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (3)$$

where,

qE	:	Embankment load (fg/m2)
Bm	:	Width loaded (m)
n	:	Coefficient as to height and width of loading road as shown below.
E	:	Average modules of elasticity (tf/m2)



COEFFICIENT (n) OF EMBANKMENT SECTION

For sandy soil

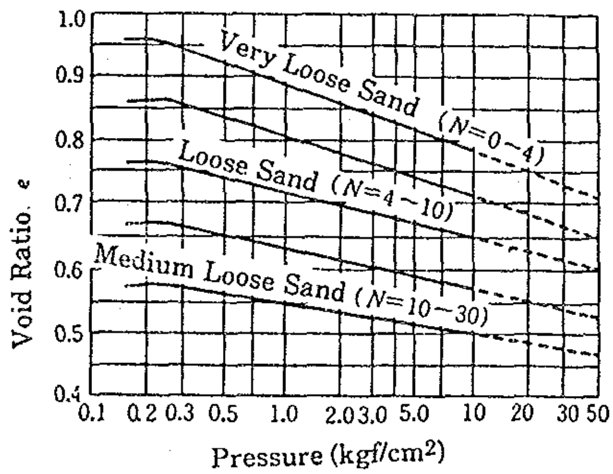
$$Sc = \frac{e_0 - e_1}{1 + e_0} \cdot H \quad \cdots\cdots \quad (4)$$

where.

e_0 : Initial void ratio given at Po on Figure 2.4

e_1 : Void ratio after consolidated, given at $P_0 + \triangle P$ as shown below

H : Thickness of stratum (cm)



Pressure versus void ratio curve of sand (B.K.Hough)

2) Sp (t); settlement caused by primary consolidation

$$Sp(t) = U.Sc \quad \cdots\cdots \quad (5)$$

where,

Sc : The consolidation settlement (Sc) is analyzed by following formula referring the respective e-log P curve

$$Sc = \frac{e_0 - e_1}{1 + e_0} .H$$

where

e_0 : Initial void ratio given at Po on e-log P curve

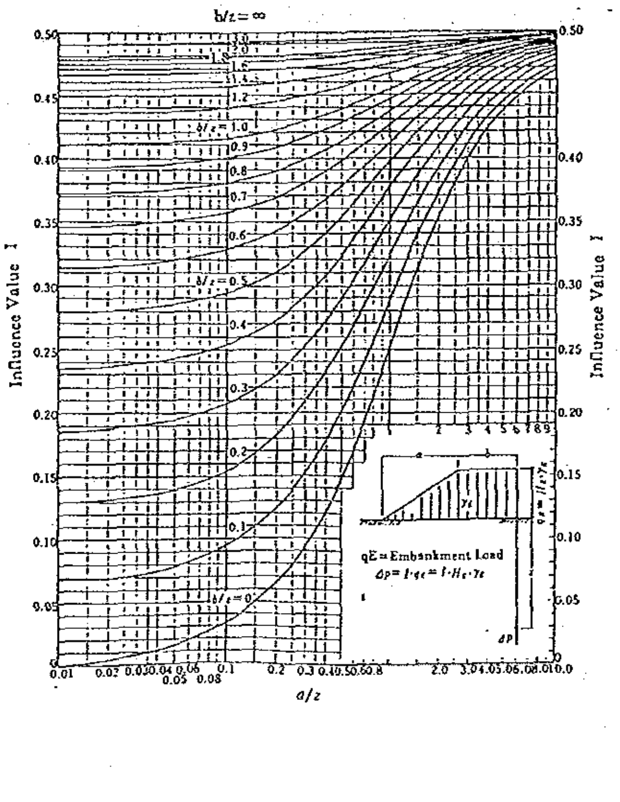
e_1 : Void ratio after consolidation, given at $P_0 + \triangle P$ on e-log P curve

H : Thickness of stratum (cm)

e-log P curves are given in the Attachment - 2 of Appendix to the Report

P_0 : Over burden pressure before embankment (Kgf/cm2)

$\triangle P$: Increased stress in a ground by embankment load (kgf/cm2), given below

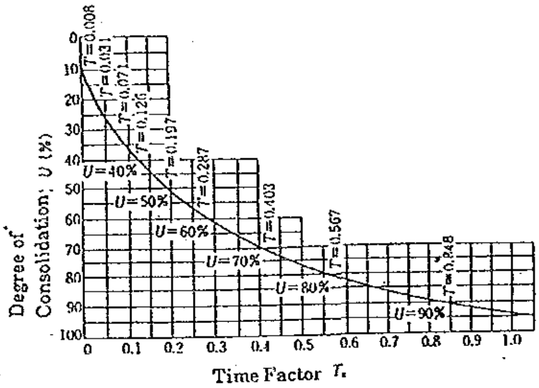


U : Degree of compression given by % as shown below which are related with time factor (t-day)

$$t = \frac{D^2}{Cv} Tv \text{ (day)}$$

where

D	:	Thickness of soft soil layer
Cv	:	Primary consolidation coefficient
Tv	:	Time coefficient or factor

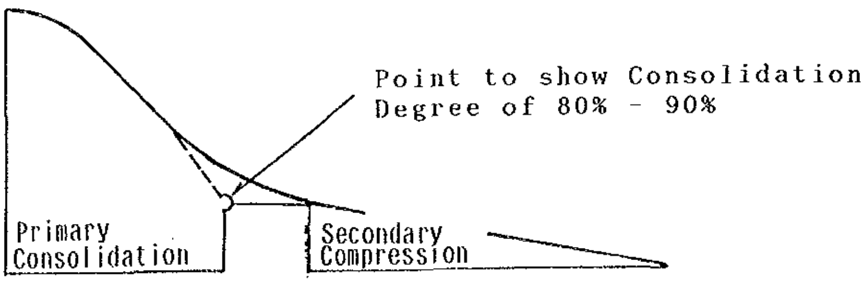


3) Ss (t_0, t) ; settlement caused by secondary compression

Ss (t_0, t) = α.H.log (t/t_0) .... (5)

where,

α	:	coefficient of secondary compression α = 0.00018 Wn
H	;	thickness of the soft soil layer
t_0	;	the time when secondary compresison will start at U = 80% ~ 90% as shown below



RELATION BETWEEN CONSOLIDATION DEGREE
AND STARTING TIME OF SECONDARY COMPRESSION

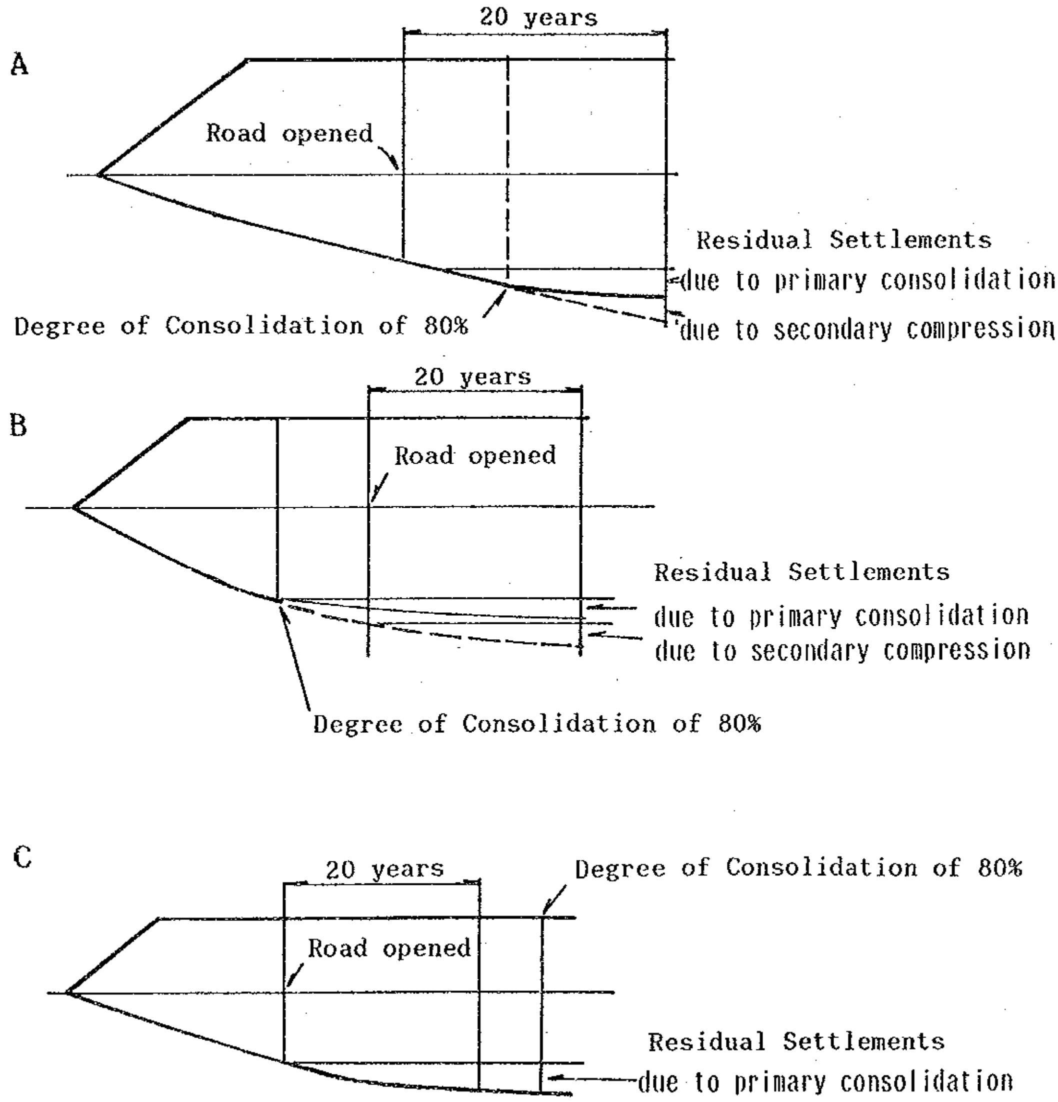
The total settlement and time-settlement curve of each cross section are shown in Figures 6-6 (1)* to (12)

2.4.2 The Residual Settlement

The residual settlement Sr is defined as follow and estimated.

$$Sr(t) = S(t + 20 \text{ years}) - S(t) \quad \ldots\ldots\ldots\ldots \quad (6)$$

Adoption of the above concept on the residual settlement gives following 3 cases for getting the residual settlement of each cross section.



DESIGN RESIDUAL SETTLEMENTS AND
SECONDARY COMPRESSION

The results calculated on settlement for each cross section are summarized in Table 6-2.

Embankment settlements by starting time of pavement work were taken account of the embankment volume due to settlements.

Table 6-2 RESULTS OF SETTLEMENT ANALYSIS

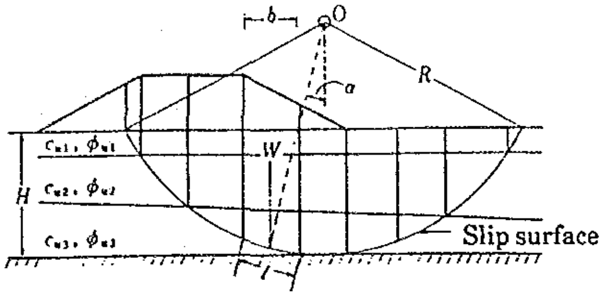
		Before Pavement	After Opening Road		
Point Examined	Boring No.	Embankment Settlement (cm)	Residual Settlement (cm)	Allowance as Final	
				Free Board Q50 (cm)	Allowance >30 (cm)
High Bank near Abutment					
MIARU	B6	39	3	60	57
ALIKA	B8	85	23	80	57
KAPURI	B9	43	17	210	203
LAKEKAMU	B11	32	4	240	236
TAURI	B14	25	21	240	219
COMMON BANK ON SOFT GROUND					
KAPURI	S2	26	4	80	76
	S3	37	3	80	77
	S4	17	4	80	36
LAKEKAMU	B11	17	4	40	29 *
TAURI	B14	10	11	100	93
MAKARA	B16	34	7	130	125
SAPPAHARO	B17	32	5		

* Additional Embanlement may be required before pavement works.

2.5 Stability Analysis

2.5.1 Formula applied

The slice method was applied to the stability analysis of embankment on soft ground as illustrated below:



The slip surface is balanced in the following formula;

$$Fs = \frac{\Sigma\{C\,l + (W\cos\alpha - U)\tan\phi\}}{\Sigma W.\sin\alpha}$$

where.

Fs	:	Safety factor
C	:	Cohesion (tf/m2)
l	:	Slipping length in each sliced block (m)
W	:	Weight of each sliced block (tf/m)
φ	:	Angle of internal friction (degree)
α	:	Average inclined angle of slipping slice (degree)
U	:	Pore water pressure (tf/m)

2.5.2 Computed safety factors

The safety factors of embankments on soft ground are shown in Table 6-3.

Table 6-3 RESULTS OF STABILITY ANALYSIS

Soft Ground	Data Source of Drilling laf	Safety Factor
Ungongo area	B3	2.2
Miaru area	B6	2.1
Alika swamp	B8	1.4
Palipala to Ilavala	B9	2.8
Terapo Mission area	B12 B13	2.1 2.5
Makara to Sappaharo	B15 B18	1.7 1.7

No impossible or unstable embankments were found out in the Project.

These stability analyses are illustrated in Figures 6-7 (1)* ~ (7)*

2.6 Embankments on soft Ground

The principal countermeasures for filling on soft grounds are proposed depending on the above analysis and site conditions as follows:

(1) Clearing of grass

Long grass exceeding 2m is growing in and above the water of Alika swamp. This grass shall be cleared with its roots.

(2) Grubbing of tree roots

Shrubbery and trees are growing in the swamps on both sides of the Kapuri river. All roots of trees and bushes shall be removed from the embankment area.

(3) Sheeting with geotextile

Bamboo or timber spreading on soft ground was discussed on site, however, geotextiles were preferred according to the experience of the DOW.

(4) Spreading the sand mat

Sand shall be spread in the water on soft ground until trafficability for dump trucks is secured. The thickness of the mat should exceed the depth of water standing in swamps. The permeability of sand material shall be ensured: subsoil drains were accordingly planned only for the sand mat using material from the Ilavala hills, but not from the coast.

(5) Placing of sand bags

Sand bags shall be placed on the slope of the sand mat in the water, which may stop the sand disapearing into the water.

(6) Low Speed embankment

It should be noted that the low speed embankment method will be applied for embankment on soft ground for wise economy; generally the speed shall not exceed an embanked height of 10 cm in a day.

(7) Bearing unit

Timber piling is proposed to prevent settlement behind abutments on soft ground. Embankment within 10m from the back of an abutment will be reinforced as a bearing unit with driven timber piles at one meter centers.

3. DRAINAGE DESIGN

3.1 Design of Drainage Culverting

Estimation of design discharge was carried out applying the Rational Method. Catchment areas were measured from topographic survey maps, scale of 1:50,000 for Bereina to the north end of Palipala Hills and 1:1,000,000 for the residual area up to Malalua. Data on catchment areas are shown in Figure 6-8*.
For a catchment area which has a certain size on a map and being able to read contour and main channel length, design rainfall intensity was estimated based on Tc, time of concentration. For a catchment area for which it is hard to estimate the Tc, the design discharge was estimated from the regression curve shown in Figure 6-9 and 6-10 calculated from obtained relations between catchments and the design discharge of larger catchments.

Diameters and required lane numbers were selected based on the rating curve shown in Figure 6-11 which was calculated as Culvert Flowing with Outlet Control. The longitudinal slope of the culvert was considered to be flat.

Most of all proposed sites for installation of culverts were located in very gently sloping areas, less than 1.0%. The rating curve estimated with the condition of flat longitudinal gradient was judged appropriate to determine culvert design with a reasonable safety factor.

3.2 Drainage in Kapuri Swamp

The drainage system from Kapuri river flood plain was assessed carefully conducting flood routing in several cases. Flood hydrography was estimated from hydrography parameters introduced in the "Flood Estimation Manual" adopting the design peak flood discharge as 1,160m^3/sec.

Fig. 6-12* to Fig. 6-14* shows the simulation results varying the assumed storage area at upstream.

Finally it was judged that there is no possibility of overflow topping the road formation even in the worst case that has no drainage pipe at all

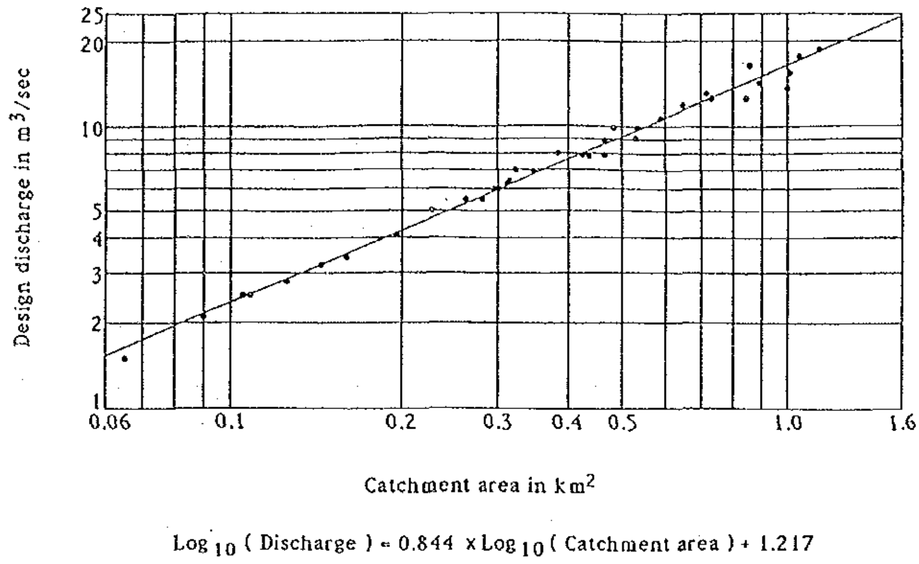


Fig. 6-9 SMALL CATCHMENT DISCHARGE (RIX = 110mm)

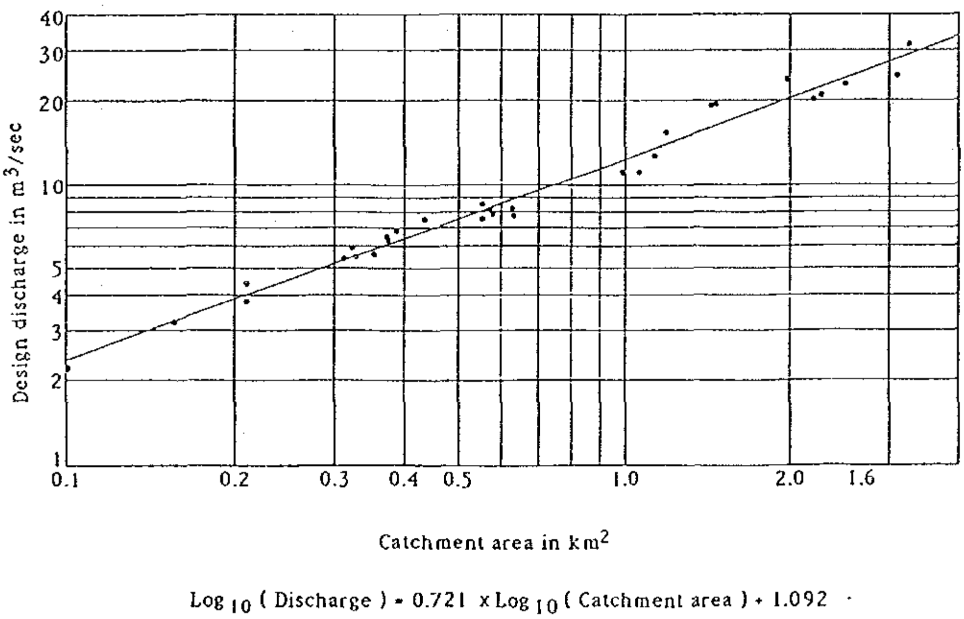
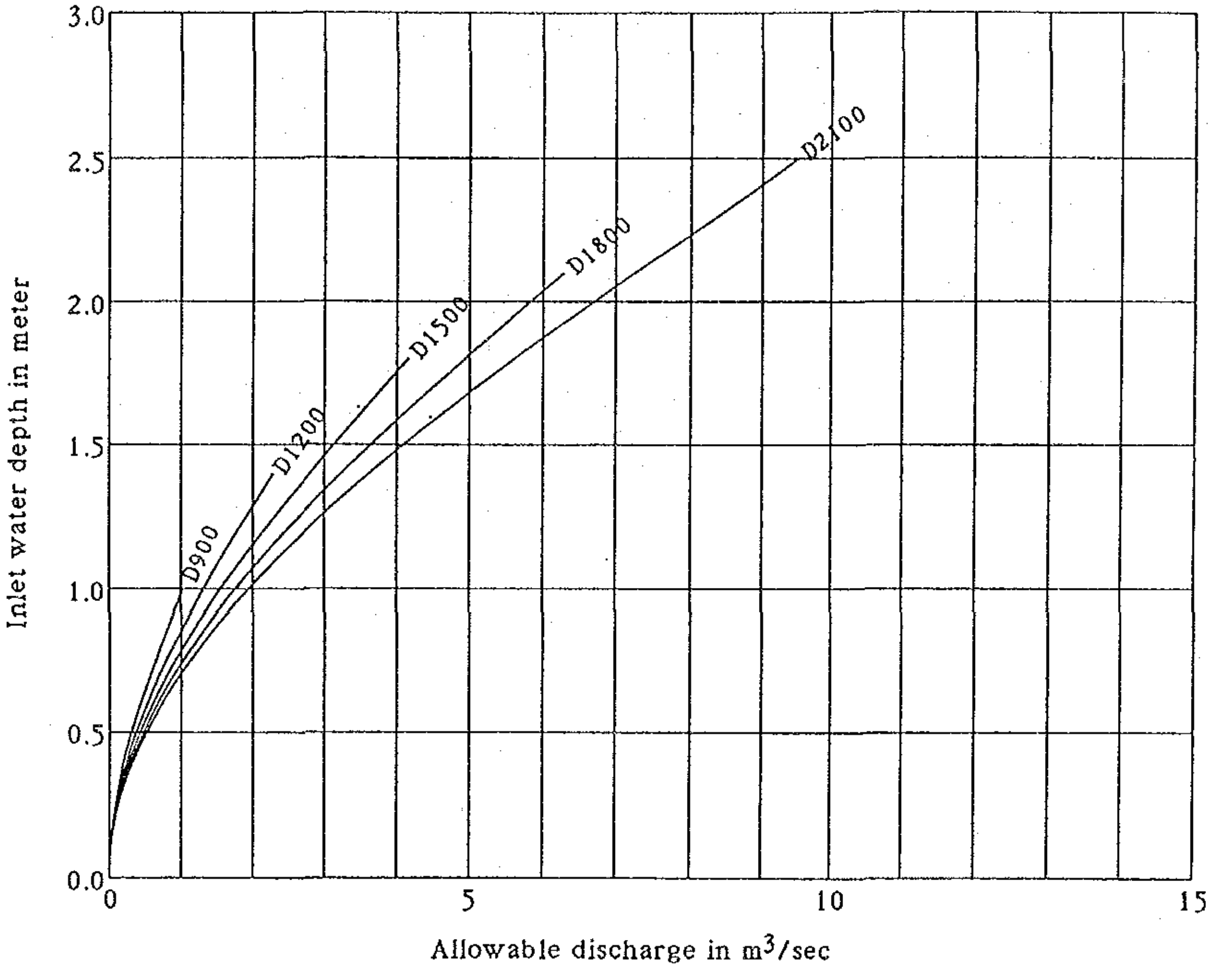


Fig. 6-10 SMALL CATCHMENT DISCHARGE (RIX = 100mm)



Assumption : Longitudinal gradient is flat.
Manning's roughness coefficient is 0.024
Headloss coefficient due to entrance is 0.500

Fig. 6-11 CSP RATING CURVE

except for the Kapuri river channel itself at the bridge site. However, drainage pipes of 900mm dia. were planned at 50m intervals in the lowest embankment stretch of 3,200 metres (CH 58+700~ CH 61+900) to ensure that the flood level shall not to rise above the level analyzed in the Study.

3.3 Selection of Steel Pipe Culvert

Before the design of the drainage culverting , the type of culvert was examined by comparing steel pipe and concrete box culverts under the following conditions :

(1) Concrete box culverts (700mm×900mm) constructed from precast concrete blocks made in field factory are feasible, and these can allow a 300mm lower road level than when using a steel pipe culvert.

(2) A steel pipe culvert (900mm diameter) is easily constructed in a swamp road but requires a minimum earth cover of 600mm for structural safety.

Based on the above conditions, the costs can be compared as below:

Comparison of Culvert Type at CH60 + 600 ~ CH61 + 600

Item	Steel Pipe Culvert ϕ 900mm per 226	Concrete Box Culvert 700mm × 900mm per 208m
A. Cost of Culvert		
(1) Price of goods	K. 51,500 (at POM)	K. 75,400 (at Terapo Field Factory)
(2) Transportation to the site	K. 2,900 (POM to Site)	K. 20,000 (Terapo to Site)
(3) Setting Up	K.9,900	K.47,000
Sub total	K. 64,300	K. 142,400
B. Cost of Earthwork	K. 253,400 (12,485m^3)	K. 177,000 (8,720m^3)
Total	K. 317,700	K. 319,600

Note : The costs in the above table were estimated in June 1988.

Although the results show no meaningful cost differences between the two culvert types, JICA study team selected the steel pipe culvert for the Project considering the following points :

(1) The higher embankment of 30 cm, in case of steel pipe culverting will provide an additional safety factor to cover the design flood level.

(2) Settlement of soft ground may cause slipping out of joints between precast blocks, so that the cracks would easily appear in the pavement due to the small cover of less than 20cm by which the top slab of box culvert supports upper part of the pavement structure.

3.4 Drainage of Road Surface

3. 4. 1 Embankment section

Drainage of the road surface of embankment sections is allowed naturally to flow down from shoulder to slopes. No special device like to set up an asphalt curb along shoulder edge was considered necessary due to the small catchment area of road surface.

However,side ditches at the toes of embankment slopes were designed, where required.

3. 4. 2 Cut section

Drainage of the road surface in cut sections was designed to flow into the side ditches constructed outside of the shoulders. The catchment area for the side ditch includes the cut slope below the first berm, and other rainfall on the upper cut slope is to be drained to the outside of the cut slope by the berm drains.

3. 4. 3 Formula applied

The discharge from road surfaces and cut slopes is calculated from road surfaces and cut slopes is calculated by following by following Rational Formula :

$$Q = \frac{1}{3.6 \times 10^6} \ C.I.A \qquad m^3/s.$$

where

Q : Discharge (m^3/s)

C : Run off coefficient of 0.75 is applied

I : Two years return-rainfall intensity of 103.7 mm/hr is applied

A : Catchment area (m^2)

As for the flowing capacity of the ditches is examined as follows :

$$q = a.\ v \ (m^3/s)$$

Where

q : Flowing capacity of ditch (m^3/s)

a : Section area of ditch (m^2)

v : Velocity given by Manning Formula (m/s).

The velocity of water flow examined does not require any protection of ditch surfaces from erosion.

CHAPTER VII DESIGN OF BRIDGES

1. PROPOSED BRIDGE SITES

In the Cardno & Davies Study, fourteen(14) bridge sites were investigated, however five(5) of them (Opou, Divola, Alika, Ungongo viaduct and Anina) were finally proposed to be replaced by culverts due to low flood quantities.

JICA study team in 1988 reviewed the proposed fourteen(14) bridge sites from the 100 year flood volumes as follows:

Name of Proposed Bridge Site	Peak Flood (m^3/s) Q100,	(Q50)	Choice of Bridge/Culvert
Taiena creek	48		Bridge
Agobino creek	59		Bridge
Ungongo creek Ungongo viaduct	155	(141)	Bridge Culverts
Anina swamp	37	(34)	Culverts
Miaru river	2,180		bridge
Alika swamp	150	(142)	Culverts
Kapuri river	1,160		Bridge
Lakekamu river	4,210		Bridge
Tauri river	3,590		Bridge
Makara river	20		Bridge
Devola creek	7	(6.5)	Culverts
Sappoharo river	22.5		Bridge
Opou swamp	23.5	(21)	Culverts

The selection of bridge or culverts was made on the site as described below:

(1) Ungongo viaduct

Total flood of Ungongo creek was estimated at 155 m^3/s. The main waterway width is 10 m and minor field traces of floods are spread over 300 metres.

The viaduct for flood was replaced by culverts of 2,100 mm dia × 12 No., which may cause the peak water level in the creek to rise up to 1.8 metres from the bank but without risk of damage.

(2) Anina swamp

The flood of Anina swamp was estimated at 37 m^3/s. The shallow field trace determined will be drained by culverts of 2,100 mm dia × 4 No..

(3) Alika swamp

Hydrological analysis on the Alika swamp indicates that it acts as a waterway to connect between the saucer and the hinterland bordered by Palipala hills and Apanaipi formations.

During flooding of the Miaru river, the water flows into the hinterland which acts as a reservoir, and then water flows down to the saucer facing towards the sea during flood in Alika, or dry seasons.

The general flowing volume was estimated at 20m^3/s at the maximum (300 m width × 3 m depth with 0.025 m/s velocity).

Therefore, it was judged to be able to drain by culverts of 2,100 mm dia × 9 No..

(4) Divola creek

The flow in Divola creek is caused by tidal movement and similarly for Makara and Sappahalo rivers. Estimated flood of 7 m^3/s at its peak can be drained through a culvert of 2,100 mm dia.

(5) Opou swamp

Estimated flood volume is 23.5 m^3/s which can be drained through a culvert of 2,100 mm dia and generally has no water in dry season.

Based on the above choice, the other nine(9) sites will be drained by bridge structures ; Taiena, Agobino, & Ungongo creeks, Miaru, Kapuri, Lakekamu, Tauri, Makara and Sappahalo rivers.

2. GENERAL SCHEME OF BRIDGES

2.1 Field Survey and Analysis

General schemes of nine(9) bridges were respectively planned based on the river cross-section survey and boring survey at each bridge site.

The location of each bridge site was determined by tape-measure with reference to the 1:500 maps or traverse survey points near the site.

The river cross-section surveys were conducted in July to August 1988 at the six(6) rivers: Miaru, Kapuri, Lakekamu, Tauri, Makara and Sappaharo rivers by survey boat (30HP Yamaha Motor) with echo sounder (computerized sporty sonar M410-M430) and casting lead rope (weight 4kg). Three(3) others creek's sections were surveyed when the supplementary field surveys were carried out in the creek-jungles. These river bed lines surveyed provide the necessary data to determine the bridge length as discussed below.

The mechanical boring surveys and the soil profile for each bridge site were discussed in Chapter IV.

This geological information was essential for the selection of foundation type of bridge sub-structures as discussed below.

Flood analysis of the rivers gives fundamental data for bridging levels together with river gauging survey by DOW at Miaru, Lakekamu and Tauri river which are discussed in Chapter V.

2.2 Bridge Plans

General schemes of nine(9) bridges are shown in the Drawings.

Flood data estimated and basic type of structure proposed are summarized below:

Flood Data and Bridge Plan

Name of Bridge	Flood Data					Type of Pile	Super Structure		
	Q100*	FWL	LWL	HWL	Current		Simple/ Continuous	Type of Girder	Bridge Length
Taiena	48	18.7	—	—	—	F	1-S	H-Beam	17
Agobino	59	12.7	—	—	—	F	1-S	H-Beam	20
Ungongo	155	13.1	—	—	—	F	1-S	H-Beam	20
Miaru	440*	5.0	1.0	3.5	1.8	F	3-S	Built up-G	90
Kapuri	180*	1.4	-0.2	0.8	1.-0	F	3-S	H-Beam	64.5
Lakekamu	1800*	2.5	0.2	2.0	3.2	B	3-C	Built up-G	120
Tauri	630*	2.5	0.4	1.3	1.6	B	3-C	Built up-G	120
Makara	20	2.1	1.2	2.0	1.0	F	2-S	H-Beam	40
Sappaharo	22.5	2.8	1.5	2.5	—	F	2-S	H-Beam	40

Note: Q100*: Design Peak discharge (m^3/sec)

Current : Design velocity on flood (m/sec)

B : Bearing type pile

F : Friction type pile

S : Simple type bridge

C : Continuous type bridge

During planning of the bridges, the following points were considered:

(1) No special consideration was given to navigable clearance of any rivers.

Soffit levels of bridges were considered for substantial amounts of floating debris as follows:

1.5m — Miaru, Lakekamu & Tauri river
1.0m — Kapuri, Makara, Sappaharo and other creek bridges

(2) Bridge length were determined based on the river cross sections surveyed at the places of abutments of which bases were properly shifted and set under the assumed slope of banks.

(3) The following type was considered : plate girder composite type not only for shorter simple spans but also for longer continuous type of bridge for reasons of economy.

Alternative bridge designs will be considered in tenders, however, the tenderers shall submit first the conforming tender to the specified bridge designs(girders), and if desired, the tenderers shall be allowed to submit alternative bridge designs with full calculations and drawings for consideration.

(4) The embankment heights behind abutments on soft ground were reviewed from the view point of stability as follows:

Name of Bridge	Embankment height behind Abutment	
	Bereina Side	Malalaua Side
Miaru	3.6m	3.3m
Kapuri	3.0m	3.1m
*Lakekamu	4.4m	4.2m
*Tauri	4.7m	4.4m
Makara	2.7m	2.1m
Sappaharo	2.0m	2.0m

As discussed in Chapter VI, conditions of soft ground near Lakekamu river and Tauri river are not better than at other places even though there appear to be rocky layers at 4~6 m depths. Therefore, 4.2~4.7 m heights may be the maximum height to be allowed.

The other heights from 2.0m to 3.6 m on soft ground are judged to be stable, however, to prevent their settlement as far as possible timber-piles will be driven into the ground behind abutments as the Bearing Unit, together with approach slabs which will be also placed behind abutments.

(5) Pile-cap positions of piers in the river were first proposed to be set above the LWL, if possible. However, the period of LWL was found to be so short that the pile-cap shall be set up above HWL. Occasional strong rainfall in the mountains raises the water levels even in the dry season.

(6) To prevent drift wood being clogged between piles, an additional pile will be driven and installed upstream of the pier's pile caps at the three(3) bridges: Miaru, Lakekamu and Tauri.

(7) For river bank protection from erosion the use of reno mattresses is proposed at following places:

- On both banks at the Taiena, Agobino and Ungongo creek bridges
- On Malalaua side bank of the Miaru river bridge.

(8) Liquefaction at Makara river bridge site was examined because a sand layer 5.7m thick was found at a depth of 4.0m to 9.7m of which N values were less than 10 as discussed later in this Chapter.

(9) Scour depths of river beds were examined by Lacey's formula using the bed-soil's grading factor of D_{50} gained from grading tests of samples from each river bed.

The scour depths from 2.4m to 3.8m were assessed for the design of foundations for the Makara, Kapuri and Miaru rivers.

(10) Tubular steel pipes were applied for the pile-foundations of abutments. Cast-in-situ concrete piles were applied for the rocky foundation of Lakekamu and Tauri piers. The other pier-foundations are tubular steel piles reinforced with RC concrete. Much drift wood was observed in the rivers during the investigation in 1988. The tubular steel piles in the rivers would be hit and deformed by drift wood, therefore, it is desirable to reinforce the steel piles with RC concrete infill.

3. DESIGN OF BRIDGE DETAILS

3.1 Design Criteria

3.1.1 Design standards applied

The Bridge structure is designed in conformity with Papua New Guinea practice, with the design criteria mainly conforming to the following domestic standards :

(1) PNG Standards 1001-1003 (1982)

(2) NAASRA Bridge Design Specification (1976)

(3) Earthquake Engineering for Bridges in PNG (1985 Revision).

Where appropriate, Japanese Standards, BS and AASHTO were adopted for these bridges after careful investigation.

3.1.2 Loadings

(1) Live Loads

In accordance with PNG Standards, standard vehicle loadings are as follows :

- A14 for local effects
- T44 one vehicle load for global effects (NAASRA)
- 60 ton overload vehicle substituting the overload vehicle.

(2) Seismic Loads

The bridges are in PNG Seismic Zone 4, Importance Factor 1.2. The bridges have been designed as non-ductile (Type C) in accordance with PNG Earthquake Code for Bridges because of the selected type of substructure.

(3) Other Loads

Other loads (wind, thermal, earth pressure, stream flow etc.) are in accordance with PNG Standards but generally are not critical.

Stream levels and velocity have been determined by stream flow records, or where records do not exist, by probability calculation.

3.2 Bridge Type

3.2.1 Superstructure

The composite girder type has been selected as the most suitable in each bridge from engineering and economic view points.

These bridges have been classified in the following three types according to the bridge length and supporting conditions at piers.

(1) Simple Girder Bridges

Taiena, Agobino and Ungongo Bridges are single spans of 17, 20 and 20 meters respectively.

(2) Simple girder bridges with Multiple Spans

Makara and Sappaharo have one central pier and Miaru and Kapuri will have three equal spans divided by two piers. These will be composite girder bridges to avoid secondary stresses caused by differential settlement between supports because the ground at these sites is comparatively soft to a considerable depth, and the piles will be predominantly supported by skin friction.

(3) Continuous Girder Bridges

Lakekamu and Tauri are single lane, three-span continuous composite girder bridges comprising two main girders with centre spans of 46 meters, and end spans of 37 meters.

At each of these bridge sites, bed-rock lies close to the river-bed surface, and there is no expectation of differential settlement at piers.

The girders will be fixed by pot bearings to each pier, thus resisting seismic force at the piers, and thermal displacements will occur at both abutments.

3.2.2 Substructure

(1) Abutments

The abutments comprise 6-steel piles connected through the pile cap which directly supports the superstructure girders via pot bearings. The piles will be steel tubular piles of 500mm minimum diameter, driven to sufficient depth to achieve the design pile loads. Pile lengths have been estimated based on NAASRA Section 4.

The design of all the abutments has been governed by seismic loading.

The abutments are designed to carry in both directions the seismic force specified in the PNG Code for non-ductile behavior and are designed for the seismically generated earth pressures, forces transmitted from the superstructure through the bearing, and inertia force of the headstock.

Linkage bars in elongated holes are provided as seismic restraints to prevent the superstructure sliding off the abutments.

(2) Piers

The piers comprise 4 piles connected through a pile cap which directly supports the superstructure by pot bearings.

The piles are steel tubular pipe of 800mm minimum diameter, with concrete filling of the upper part of the piles after driving. Because the steel pipe in the water will be difficult to maintain to prevent corrosion over the bridges' life-times, (expected to be more than 40 years) the piles will be filled with reinforced concrete above a specified level below existing river-bed level as determined from the design of the lateral load resistance.

The steel wall of the pile is neglected in determining the bending capacity of the pile over this region.

The pile depths required to support vertical loads have been estimated based on NAASRA Section 4.

The design of the pier is governed by seismic loading. The piers are to be designed to withstand in all directions the seismic force specified in the PNG Earthquake Code for Non-Ductile Behavior and is designed for the forces transmitted from the superstructure plus the inertial force of the headstock.

Linkage bars are provided between the girders.

3.3 Proposed Details

3.3.1 Slab thickness and reinforcement

The slab has been designed according to NAASRA, Section 5.
The slab thickness and reinforcing steel are designed accordingly. The concrete is strength Grade 25 and the slab thickness is 180mm where girder centres are 1,650mm, 200mm for 2300mm girder centers, and 230mm for 2700mm girder centres. The reinforcing yield strength is 410 MPa, with main reinforcing steel being 16mm bars at 200mm centres. Distribution steel is at 250mm centres for bridges with smaller girder centres, and 200mm for bridges with the larger girder centres.

3.3.2 Main girder section

The proposed sections of main girders at mid span are listed along with maximum deflection under live load in Table 7-1* in Appendix.

All simply supported girder bridges will have universal beam sections of 900mm deep of the series, (Steel Grade 350) as these are the most economical except for the Miaru river bridge.

The other bridges such as Miaru, Lakekamu and Tauri are proposed as built up plate girder sections as the span lengths are out of the available range of universal beam sections. The proposed beam for Miaru bridge is 1100mm deep, so that the level for the approach embankment is as low as possible. The Lakekamu and Tauri bridges, which are both founded on bed-rock lying close under the riverbed level, have economically suitable girder depths of 1800mm. These bridges will also be of grade 350 steel.

3.3.3 Cross beams and sway bracing

The arrangement of cross beams and sway bracing is designed appropriately. In relation to cross beams, all of the bridges except in group [1] (Lakekamu and Tauri) will have more than two main girders and have rigid cross beams at mid span to develop more efficient distribution of live load between the girders.

Other inner cross beams are provided at 4 to 5 metres pitch to maintain the shape of the main girders.

Cross beams at bearings have been designed as rigid frames to resist the transverse forces of earthquake and wind.

Sway bracing in the plane of the lower flanges has been used for the three bridges with spans of more than 30 meters.

The details discussed above are illustrated in Figure 7-1.

3.3.4 Substructure

The principal components of the substructures and a summary of pile loads and stresses are shown in Table 7-2* and 7-3* in Appendix.

All abutments and piers comprise piles connected through reinforced concrete pile caps which directly support the superstructure via bearings.

Long friction piles are required at all sites other than for the two Group I bridges (Lakekamu and Tauri) which will have a continuous superstructure. Friction piles at the abutment (6 piles per abutment) are 500mm or 600mm diameter with 14mm wall thickness and are driven open-ended to estimated depths of between 7 and 23 meters. The top metre of these piles will be concrete filled. Friction piles at the piers (4 piles per pier) are 800mm diameter with 12mm wall thickness and are estimated to be between 20 metres and 30 metres long. These are designed to be filled with reinforced concrete above a specified level below existing river-bed level as determined by the design of the lateral load resistance. The contribution of the steel wall to the bending capacity of the pile is neglected over this upper part of the pile. To assist the concrete placement, the piles will be driven with closed ends.

Bridge Section | Stractural Skeleton

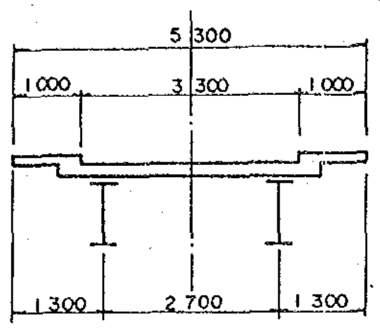
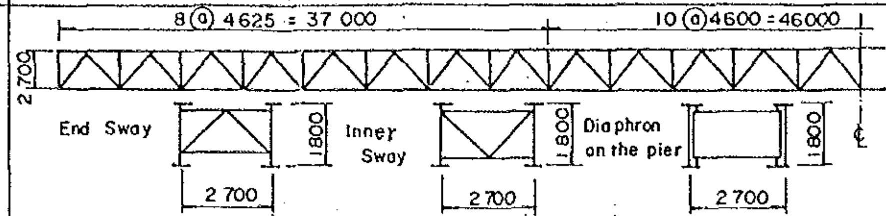
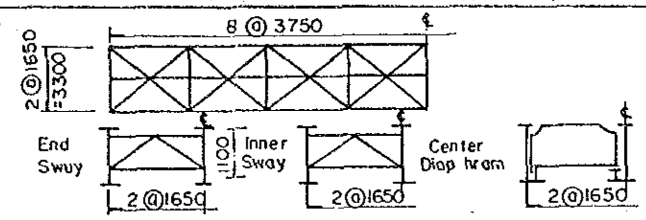
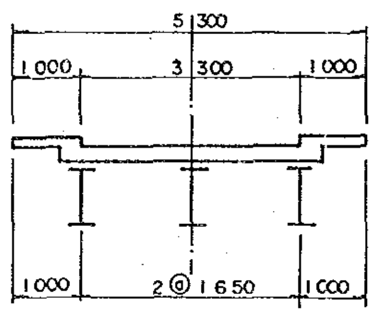
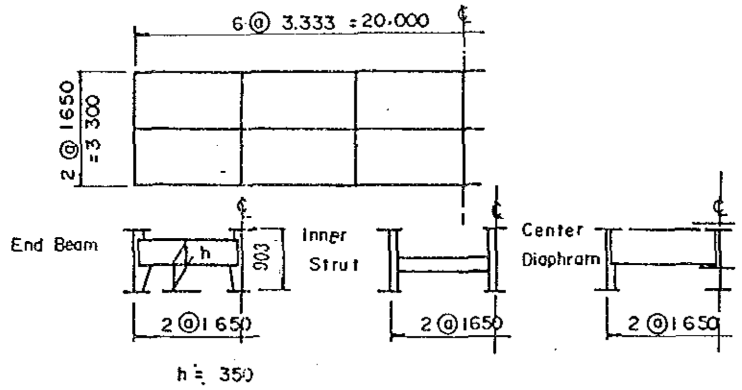
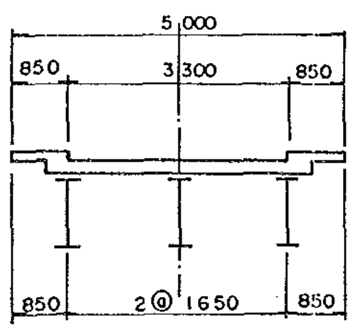
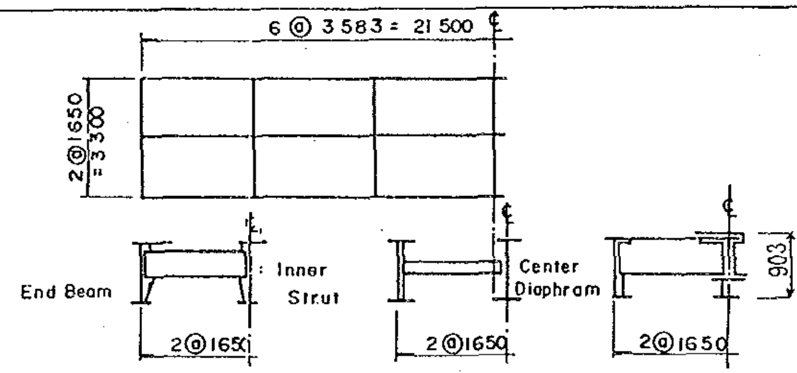
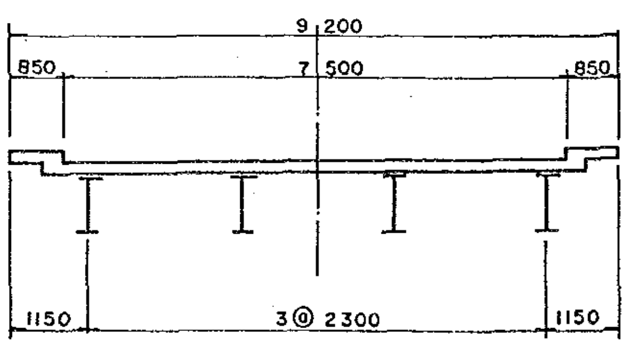
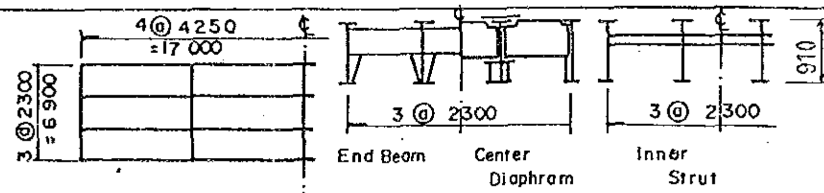
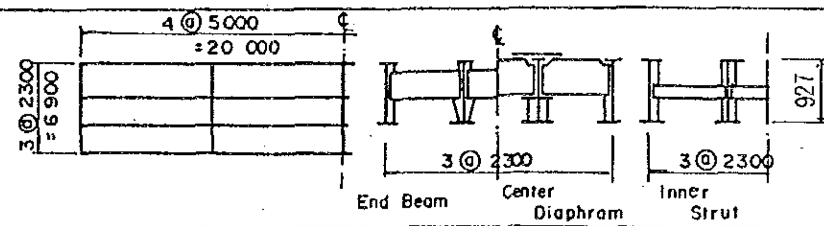
Group	Bridge	Section		Bridge	Bracing & Skeleton
Group [I]	· Lakekamu River · Tauri River		Deck 230 mm	· Lakekamu River · Tauri River 37+46+37	
				· Miaru River 3 @ 30.0	
Group [II]	· Miaru River · Makara Rivar · Sappaharo River		Deck 180 mm	· Makara River · Sappaharo River 2 @ 20.0	
Group [III]	· Kapuri River		Deck 180 mm	· Kapuri River 3 @ 21.5	
Group [IV]	· Taiena Creek · Agobino Creek · Ungongo Creek		Deck 200 mm	· Taiena Creek 17.0	
				· Agobino Creek · Ungongo Creek 20.0	

Fig. 7-1 DETAILS OF PLATE GIRDER BRIDGE

Foundation drilling at the bridge sites has provided one borehole at single span bridge sites, and a borehole at each abutment at the multi-span bridge sites. Where the design data from pile borelogs have not been sufficiently deep, the strata have been linearly interpolated between holes, and it has been assumed that the soil properties at the bottom of the hole remain unchanged at deeper levels. These assumptions should be conservative. All pile depths are governed by seismically generated loads and lengths are determined from NAASRA Section 4, adopting a factor of safety of 2.5 for the seismic force plus dead load. Compressive loads rather than tension loads always govern the pile depth assessment. Consideration should be given to the Contractor conducting further, deeper investigation at the friction pile sites prior to ordering of pile lengths, so that pile order lengths can be more accurately estimated.

At the Lakekamu and Tauri bridges sites, a reasonable quality rock has been shown to exist about 4 to 8 metres below the bank levels of both rivers. Insufficient tests have been made on these strata to determine design parameters, and in this design, these silt stones, sandstones, limestones and agglomerates are assumed to have an ultimate bearing capacity of 10MPa. The piles have been designed for steel cylinders to be driven into the top of this rock strata, with a socket drilled or chopped out below this steel liner casing. The piles will be 800mm diameter at the abutments and 1000mm diameter at the piers. The length of the socket will be governed by the capacity of the piles to mobilise sufficient uplift resistance in the surrounding strata. The depth of socket at each abutment has been determined to be 4 metres deep below the steel liner casing, and 9 metres below the liner casing at the piers. The piles are then filled with reinforced concrete which provides the structural capacity for the loads, the liner casings being fully neglected.

3.3.5 Reaction and Bearing Type

There are many types of bearings available. In this project, standard Japanese bearings have been proposed because they have a long history of successful use under seismic conditions and can be readily repaired, if damaged after a large earthquake.

In PNG, bearings are usually adopted which can support the vertical reaction of the girders, and buffers or stoppers are located at either side of the girder to prevent the girder falling transversely off the bearing. Pot bearings are a suitable alternative because they can be easily fixed at the site, and because they support both vertical reaction and transverse forces by themselves.

Table 7-4* in Appendix shows reactions and bearing type selected according to the reactions.

3.4 Other Details

Other details of the bridge design are described below.

(1) DOW Standard handrails shall apply.

(2) Expansion joints are designed following the simple type of DOW.

(3) Painting of girders was agreed that either anti-corrosive paint or galvanizing will be used for the bridges. A detailed specification for the painting works shall be provided.

4. DETAILED ANALYSIS ON LIQUEFACTION

Detailed analysis for liquefaction at Miaru, Kapuri, Makara and Sappaharo bridge sites was done. This shows no danger of liquefaction due to rich fine grain size of the sands as described below.

4.1 Formula Applied for Analysis

Analysis was made based on the design manual which has been developed in Japan, and has been applied for the examination of liquefaction since the serious experience in Niigata earthquake (magnitude about 7.3) in 1964.

4.1.1 Sand layers to be examined

Saturated cohesionless alluvium layer of which:

- ground water level is less shallower than 10m
- Average grain size (D50) in the depth of 20m is found in the range of 0.2mm to 2.0mm.

4.1.2 Judgement on liquefaction

Judgement on the liquefaction can be made seeing the resistance ratio (F_L) calculated by following formulas:

$$F_L = R/L \quad (1)$$

$$R = R1 + R2 \quad (2)$$

$$L = \gamma d \cdot ks \cdot \frac{\sigma v}{\sigma' v} \quad (3)$$

$$\gamma d = 1.0 - 0.015x \quad (4)$$

$$\sigma v = (\gamma h + \gamma\ (x-h\ )\ /10 \quad (5)$$

$$\sigma' v = (\gamma h + \gamma'\ (x-h\ )\ /10 \quad (6)$$

Where,

F_L	:	Resisting ratio to liquefaction
R	:	Dynamic resistance forse ratio to shearing
L	:	Shearing stress ratio in earthquake
γd	:	Reducing factor for shearing force y depth
ks	:	lateral seismic coefficient on the ground surface (0.14)
σv	:	Total surcharge stress (kg/cm^2)
$\sigma' v$	:	Effective surcharge stress (kg/cm^2)

R1	:	the first factor to be calculated with relation between N values and effective surcharge σ'v as shown in Figure 7-2
R2	:	The second factor to be given by the average grain size (D50) as shown in Figure 7-3
x	:	Depth from ground surface
h	:	Ground water level from ground surface
γ	:	Unit weight of soil above ground water level (t/m^3)
γ'	:	Effective unit weight of soil below ground water level (t/m^3)

The judgement is made as follows:

If $FL < 1.0$
The sand layer is liquefiable

If $FL \geqq 1.0$
The sand layer is not liquefiable

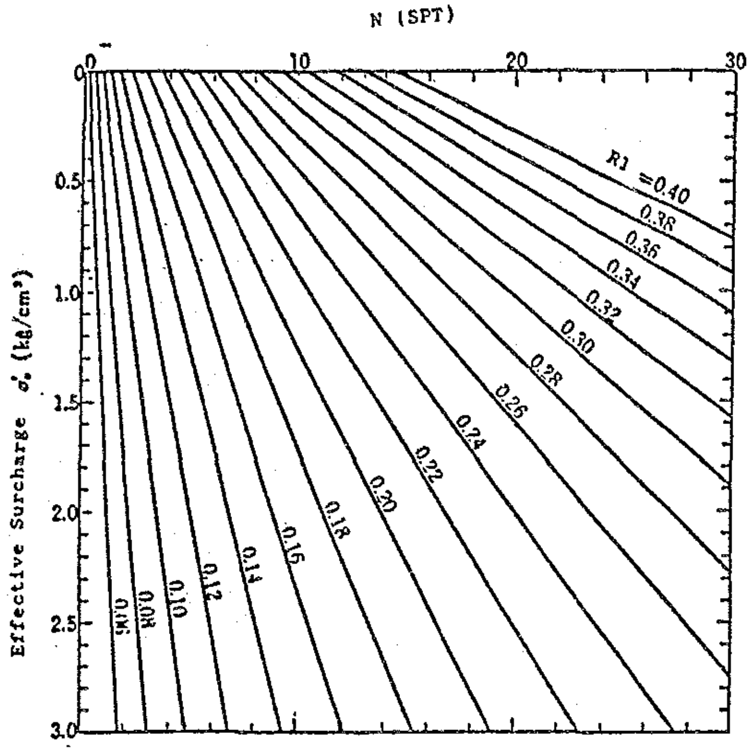


Fig 7-2 Relations between R1, and σ'v (Effective Surcharge)

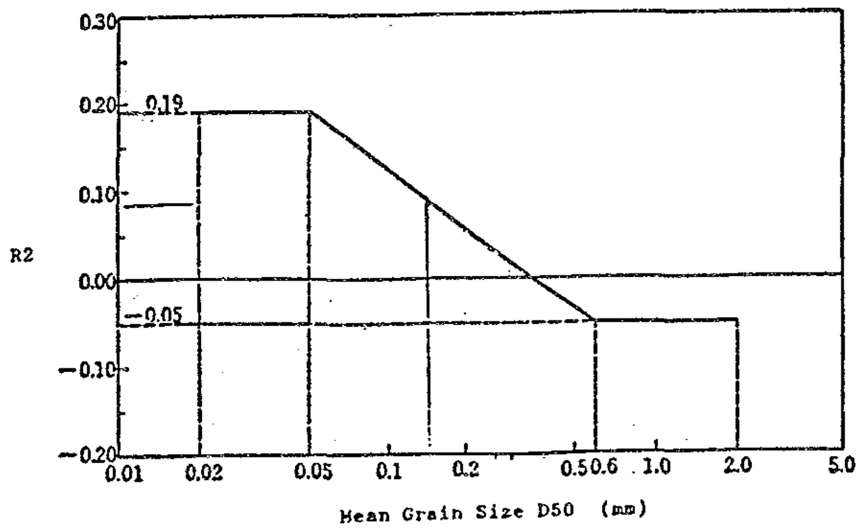


Fig 7-3 Relations between R2, and D50 (Mean Grain Size of Sand)

4.1.3 Countermeasures for liquefaction

If the sand layer is liquefiable in an earthquake, the soil factor to be applied for a seismic design shall be reduced by ratio (D_E) in the range from 0 to 1 as shown in table 7-5.

Table 7-5 Adjusting Factor (D_E) to soil factor

Value of (FL)	Depth (X) of layer (m)	Reducing ratio (D_E) for soil factor
$FL \leqq 0.6$	$0 \leqq x \leqq 10$	0
	$10 < x \leqq 20$	1/3
$0.6 < FL \leqq 0.8$	$0 \leqq x \leqq 10$	1/3
	$10 < x \leqq 20$	2/3
$0.8 < FL \leqq 1.0$	$0 \leqq x \leqq 10$	2/3
	$10 < x \leqq 20$	1

4.2 Results of Analysis

Results of the liquefaction analysis on the Miaru, Kapuri, Makara and Sappaharo bridge sites are shown below:

Name of Bridge	Values of FL Left Bank	Right Bank
Miaru	1.37 - 2.48	∠1 0.99 - 1.70
Kapuri	1.06 - 2.10	∠2 0.99 - 3.05
Makara	1.22 - 1.47	1.10 - 1.45
Sappaharo	1.01 - 1.80	1.27 - 4.49

Note:

∠1 0.99 at the depth of 12m is negligible for the seismic design (see Table 7-5)

∠2 0.99 at the depth of 3m is found at peat which formulates an isolate lens layer, which is also negligible.

The values of FL and DE are shown in Figures 7-4* to 7-7*, and calculation sheets are attached in Table 7-6* (1) - (8) in Appendix.

CHAPTER VIII PAVEMENT DESIGN

1. BASIC POLICY OF DESIGN

1.1 Design Standard

For the pavement structural design, Road Note 31, A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries, 1977 (Transport and Road Research Laboratory UK) was applied in accordance with DOW Road Design Manual. DOW Specification for Road and Bridge Works 1978 and its revision were also applied as the design standard in the material requirements for this pavement design.

1.2 Design Traffic Loading

The design traffic loading was determined by the following procedures.

a) Design Period = 10 Years

b) Initial Average Daily Traffic : 200 vehicles / day

The traffic data forecast in 1985 were given by the Department of Transport (DOT). JICA study team judged the initial ADT of 200 acceptable seeing the actual traffic on Hiritano highway.

The base year traffic in one direction, therefore, is 100 vehicles per day.

c) Annual Traffic Growth Rate = 3.0%

This was quoted in the paper from DOT mentioned above.

d) Cumulative Number of Standard Axles (8.2 ton)

Following assumptions were applied to calculate the cumulative number of standard axles:

- Percentage of commercial vehicles for daily traffic = 40%

- Conversion factor to obtain the equivalent number of standard axles = 0.45.

Cumulative number of standard axles was calculated as follows:

$$100 \times 0.40 \times 0.45 \times 365 \times \frac{(1 + 0.03)^{10} - 1}{0.03} = 75{,}922 = 0.08 \times 10^6$$

Conversion factor of 0.45 to be used to obtain the equivalent number of standard axles from number of commercial vehicles is given below:

Type of Road	Number of per commercial vehicle (a)	Number of standard axles per commercial axles (b)	Number of standard axles per commercial vehicle (a) × (b)
Motorway and trunk roads designed to carry over 1000 commercial vehicles per day in each direction at the time of construction	2.7	0.4	1.08
Roads designed to carry between 250 and 1000 commercial vehicles per day in each direction at the time of construction	2.4	0.3	0.72
All other public roads	2.25	0.2	$\boxed{0.45}$

Source : Road Note 29, TRRL UK

2. BEREINA TO MIARU RIVER SECTION PAVEMENT DESIGN (LOT 1)

2.1 Design Subgrade CBR

a) Design Subgrade CBR based on Soaked CBR

Soil test data for this section are shown in Table 8-1*. The soaked CBR value ranges from 3.4 to 7.0% and the mean value is 5.1% after discarding extreme low and high values.

The design subgrade CBR value is calculated by the following equation:

Design CBR value

= Mean CBR value - Estimated standard deviation

$$= \text{Mean CBR} - \frac{(\text{Maximum CBR} - \text{Minimum CBR})}{C}$$

$$= 5.1 - \frac{7.0 - 3.4}{2.48} = 3.6$$

Where C value are shown below.

Number of Sample	2	3	4	5	6	7	8	9	10 or more
C	1.41	1.91	2.24	2.48	2.26	2.83	2.96	3.08	3.18

b) Design Subgrade CBR based on Plasticity Index

Road Note 31 provides a table to estimate minimum design subgrade CBR values in connection with the depth of water table from formation level and the plasticity index (PI) of subgrade soils. In Table 8-1*, PI value ranges from 11 to 49 and the mean value is 19.6.

The PI value which 84% of all the test values are equal or smaller than the design PI as calculated below :

Design PI

= Mean PI + Estimated standard deviation
= 19.6 + 7.9 = 27.5

Therefore, PI value for the design use is about 30.
For the PI of 30, the estimated minimum design subgrade CBR values were obtained as below from the following PI-water table depth-CBR value relation table described in Road Note 31.

The depth of water table from formation level	Minimum CBR value
0.6 m	3 %
1.0 m	4 %

The estimated minimum design subgrade CBR value under paved roads for subgrades compacted to 95 per cent of British Standard maximum dry density is given from the following table :

Depth of watertable from formation level		Minimum CBR (per cent)					
		Non-plastic sand	Sandy clay P1=10	Sandy clay P1=20	Silty clay P1=30	Heavy clay P1=40	Silt
0.6m	(2 ft)	8	5	4	3	2	1
1.0m	(3.3 ft)	25	6	5	$\boxed{4}$	3	2
1.5 m	(4.9 ft)	25	8	6	5	3	
2.0 m	(6.5ft)	25	8	7	5	3	
2.5 m	(8.2 ft)	25	8	8	6	4	
3.0m	(9.8 ft)	25	25	8	7	4	
3.5m	(11.5 ft)	25	25	8	8	4	
5.0 m	(16.4 ft)	25	25	8	8	5	
7.0m	(23 ft) or more	25	25	8	8	7	

c) Determination of the Design Subgrade CBR

The soil type of the road bed soils along this route section are fine grained soils and have rather high PI values. From this consideration, the design subgrade CBR value was determined at 4 %.

2.2 Pavement Thickness Design

The layer thickness of the pavement was determined from the pavement design chart in Road Note 31 (Figure 8-1*) by applying the design traffic loading of 0.08×10^6 cumulative number of 8.2 - ton standard axle loads and the design subgrade CBR of 4 %. The obtained pavement layer thickness design is as follows:

Surface Double Bituminous Surface Treatment
Base course 150 mm (CBR not less than 80 %)
Subbase 240 mm (CBR not less than 25 %)

2.3 Consideration on Subbase

a) Subbase Material Properties

The test results on subbase materials (sandy gravel, or more likely gravelly sand or clay) from the expected borrow pits, such as Liquid Limit (LL), Plasticity Index (PI) and soaked CBR value are listed in Table 8-2*.
A summary is given below.

Summary of Subbase Material Test Results

Items	Ranges /1 of test values	Mean value	DOW specification limits
LL	35 - 43	39	not more than 30
PI	10 - 22	16	not more than 10
CBR	11 - 35	23	not less than 25

/1 Range were calculated from equation X ± s, where $\underline{X}$ is the mean value and s is the standard deviation of the individual value.

The above results shows that there are considerable amounts of materials with CBR values of not less than of 25 % which are out of DOW specification limits, and LL and PI are almost all out of the specification limits.

No alternative and suitable granular materials for subbase use could be found in this project area. The sandy gravels (gravelly sand or clay) in this area are well graded granular material, however, have rather high plastic nature.

Road Note 31 does not specify the LL and PI limits but only the CBR value for subbase materials. This must be for economy, on the assumption that a naturally occurring gravel or gravelly sand or clay should be utilized as subbase materials as much as possible.

b) Cement Addition (Cement Stabilization)

To improve the quality of naturally-occurring gravelly sand or clay material, such as to reduce PI and increase CBR, cement stabilization is widely applied in road construction. More technically this is recognized as Cement Addition with a small amount of Portland cement, i.e., 0.5 ~ 1.5 % by weight. Because of the low percentage of cement used, the materials are usually not completely stabilized but nevertheless are decidedly improved. For example, in the case of silty or clay gravel, as little as 1% cement may be used solely for PI reduction, and this provides good compaction and good workability for subsequent construction.
The results of cement addition tests are shown below and in in Fig. 8-2.

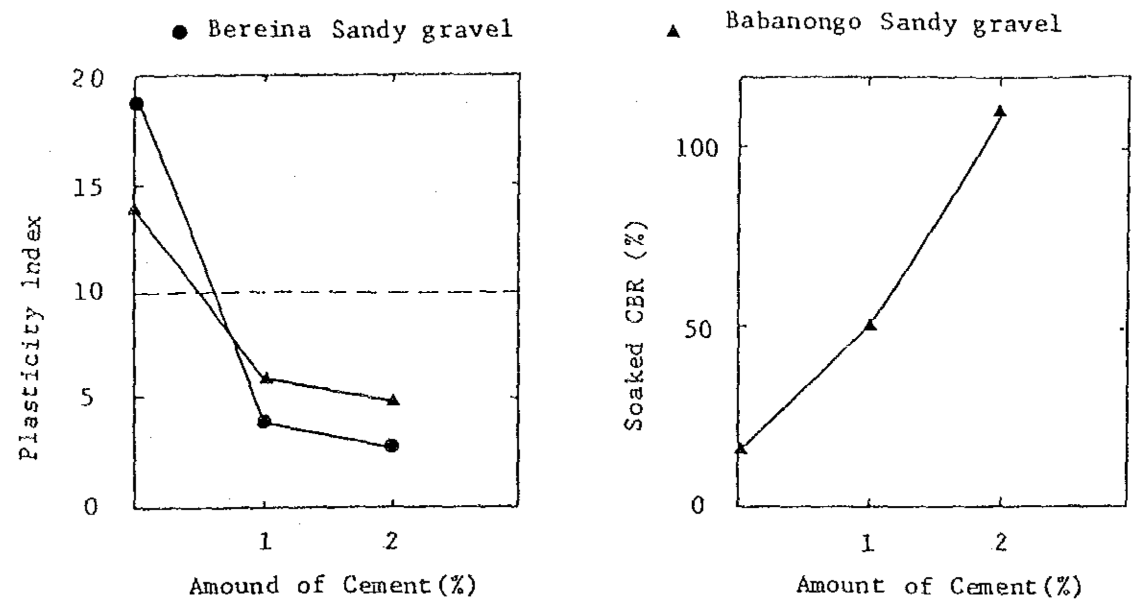


Fig. 8-2 Ip AND CBR IMPROVED BY CEMENT

The Cement Adding Effect on Subbase Sandy Gravel

Addition of Cement %	PI		Soaked CBR %
0	19[1)]	14[2)]	15
1	4	6	50
2	3	5	110

Note : Type A portland cement was added by weight of the dry aggregate

1) Bereina pit-run sandy gravel

2) Babanongo pit-run sandy gravel

The test showed that addition of 1 ~ 2 % cement by weight had a remarkable soil modification effect. The PI drops and CBR increases both sharply. It can be said that addition of 1.5 % would be sufficient for stabilization of this sandy gravel subbase.

c) Subbase Design

However, for economy , this cement stabilization should be applied to the top 100 mm layer of the subbase only.

Road Note 31 suggests that if good material is scarce, the quality of material beneath the top 100 mm layer of subbase can be relaxed, provided it has a CBR of at least 8 % when tested at the worst moisture content likely to occur.

The sandy gravel materials in this area mostly have soaked CBRs of more than 8 %. From the test results and such considerations, the following subbase design was determined.

Upper subbase 100 mm : Cement treated sandy gravel with 1.5 % cement content.

Lower subbase 140 mm : Non treated sandy gravel.

d) Subbase Material Specification for the Construction

Grading Requirements :

The grading of these sandy gravels is shown in Fig. 8-3*. The grading curves are mostly located on the upper limit side of DOW specification grading range, and the grading curves of some samples are outside the upper limit of the DOW specification. Therefore, a revised grading range for the construction specification is required for material quality control using these sandy gravels.

The following special specification for subbase material are proposed for this construction.

Grading Requirements for
Upper Subbase and Lower Subbase

Sieve Size	Percent by weight passing	
	DOW Specification Type B	Preferred Specification by JICA Study Team
75 mm	100	100
37.5 mm	60 - 100	70 - 100
19.0 mm	40 - 80	55 - 85
9.5 mm	30 - 60	40 -70
4.75 mm	20 - 45	30 - 55
2.36 mm	15 - 35	20 - 45
425 um	8 - 22	5 - 23
75 um	3 - 12	3 - 15

Upper Subbase material (Cement Treated Subbase) :

Plasticity Index : The fraction of cement treated material passing 425 μm sieve shall have a plasticity Index not greater than 10.

Soaked CBR : The cement treated material shall have a CBR value of not less than 25 % after 4 days soaking when compacted to at least 95 % maximum dry density as determined by AS 1289 Test No. E1. 1.

Lower Subbase Material (Untreated Subbase) :

Soaked CBR : Sandy gravel shall have a CBR value of not less than 8 % after 4 days soaking when compacted to at least 95 % maximum dry density as determined by AS 1289 Test No. E1. 1.

Prime Coat :

The compacted cement treated subbase shall be cured with a bituminous curing seal applied as soon as possible after the completion of final rolling. For this curing seal, DOW specification for Bituminous Prime Coat is to be applied.

Cutback bitumen : AMC 0, or a mixture of approximately 78 parts aviation kerosene or power kerosene with 100 parts of class 170 bitumen. Application rate, 0.4 - 0.8 l/m^2.

Blotter material : Clean, dry sand or stone screening. Application rate, 0.3 - 0.5 kg/m^2.

2.4 Base Course

Base course using crushed stone aggregate (38 ~ 0 mm) from Eboa rock quarry was planned. The material quality test value for Aggregate abrasion loss was 24 %, and this value meets DOW specification limits of abrasion loss of not more than 35 %. DOW Specification for Base Course Material and Bituminous Prime Coat were applied for this design.

2.5 Surface

Double Bituminous Surface Treatment (DBST) was designed based on DOW Specification for Bituminous Surface Treatment.

The treatment will be as follows :

First seal :
 Cover aggregate : 19 mm 65 - 85 m^2/m^3
 Bituminous material : Bitumen 170 class 1.25 - 1.65 l/m^2
Second seal :
 Cover aggregate : 9.5 mm 100 - 135 m^2/m^3
 Bituminous material : Bitumen 170 class 0.6 - 1.0 l/m^2

The actual application rate of bitumen binder and cover aggregate shall be determined by the method set out in Road Note 39 - Recommendations for Road Surface Dressing, published by the TRRL, UK.

Cover aggregates :

The material quality test results for the samples taken from Eboa quarry and Angabanga River are shown in the following table.

Test Items	Eboa quarry Rock		Angabanga River Coarse gravel	DOW Specification
	Quarry face sample	Boring sample		
Specific Gravity (SSD) t/m^3	2.24	2.23	2.7	—
Water absorption %	4.5	7.3*	1.0	5.0 Max.
Abrasion loss %	24	—	17	30 Max.
Sodium Sulphate soundness %	9.1	100*	1.5	12 Max.

* Out of DOW Specification limits, the test was done in Japan in December 1988.

The test results show :

Eboa rock sample taken from the quarry face meets DOW specification requirements for specific gravity, water absorption, abbrasion loss and sodium sulphate soundness.

Eboa rock sample taken from boring core does not meet DOW specification requirements for water absorption and sodium sulphate soundness.

Angabanga coarse gravel meets DOW specification requirements.

From these test results, the use of cover aggregates produced from Angabanga river gravel was planned for bitumenous surface treatment design.

2.6 Proposed Pavement Design

The pavement design for the lot from Bereina to Miaru river is proposed as shown in Figure 8-4.

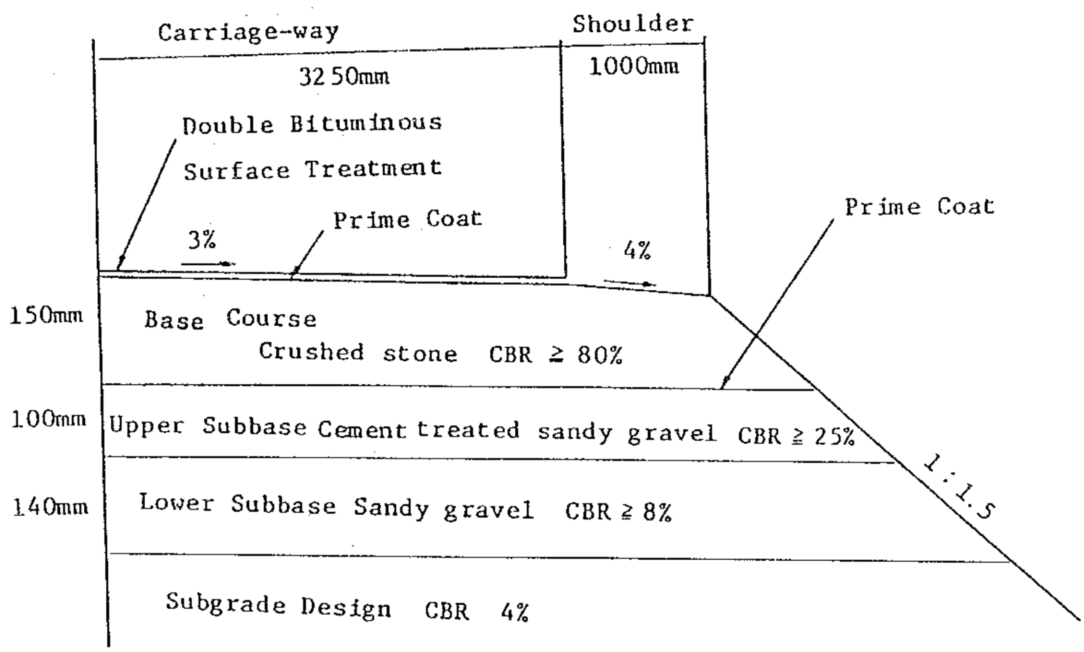


Fig. 8-4 PAVEMENT DESIGN PROPOSED FOR LOT I

3. MIARU RIVER TO MALALAUA SECTION PAVEMENT DESIGN (LOT II)

3.1 Design Subgrade CBR

Soil test data of the embankment materials for each segmented section between Miaru River to Malalaua are shown in Table 8-3*.

a) Miaru River to Kapuri River Sub-section

The embankment material for this section was planned mostly using Palipala hill soils. The test data of the representative soil are clayey soil with PI of 13, soaked CBR values of 3.3 %, 6.8 % and 9.4 %. The estimated minimum design subgrade CBR value for the soil having PI of 13 at the 0.6 m water-depth from formation level was obtained as CBR of 4 to 5 % from Road Note 31 CBR estimation table. For the soaked CBR value, X - S = 3.5 %. Design subgrade CBR value was determined at 4 % from the above consideration.

b) Kapuri River to Tauri River Sub-section

The embankment material was planned to use Illavala hill soils and the test results of the sampled soil are:
Silty sand with PI of 17, soaked CBR value of 5.3 % and 8.2 %. The estimated minimum design subgrade CBR value for the soil with PI of 17 at the 0.6 m water-table depth from the formation level was obtained as CBR of nearly 4 %. For soaked CBR value, X - S = 4.7 %. Design subgrade CBR value of 4 % was adopted.

c) Tauri River to Malalaua Sub-section

It is proposed that Malalaua hill soils be used as embankment material and the soil test results of the sample soil are :

Clayey silt with PI of 32, soaked CBR value of 3.7 %. The estimated minimum design subgrade CBR value for the soil with PI of 32 at the 0.6 m water-table depth from the formation level was obtained as CBR of nearly 3 %. Design CBR value of 4 % was adopted.

3.2 Pavement Thickness Design

The layer thickness of the pavement was determined from the pavement design chart in Road Note 31 by applying the design traffic loading of 0.08 $\times$ 10^6 cumulative 8.2-ton standard axle-loads and the design subgrade CBR of 4 %. The pavement thickness design was obtained as follows:

Surface Double Bituminous Surface Treatment

Base Course 150 mm (CBR not less than 80 %)

Subbase 240 mm (CBR no less than 25 %)

3.3 Subbase

a) Subbase Material Properties

The test results of subbase materials (sandy gravel or gravel - sand - clay) from the expected borrow pit in Malalaua hills are shown in Table 8-4*. The summary is shown below.

Summary of Subbase Material Test Results

Items	Ranges* of test values	Mean value	DOW specification limits
LL	28 - 36	32	not more than 30
PI	3 - 11	7	not more than 10
CBR	18 - 37	28	not less than 25

* Ranges were calculated from equation X $\pm$ s, where X is the mean value and s is the standard deviation.

The above results show that some parts of the materials do not satisfy the DOW specification limits of PI not more than 10 and

CBR value not less than 25 %. LL is almost out of the limit of not more than 30.

b) Cement Addition (Cement Stabilization)

To improve the quality of these naturally-occurring gravelly sand or clay materials such as reducing PI and increasing CBR, the addition of 1.5 percent by weight of portland cement, was already confirmed in the laboratory tests for the same type gravel-sand-clay materials on the Bereina section.

c) Subbase Design

As for the subbase design, Road Note 31 recommended that if good material is scarce, the quality of material beneath the top subbase can be relaxed, provided it has a CBR at least 8 % when tested at the worst moisture condition likely to occur. The sandy gravel materials from Malalaua hills have a soaked CBR of more than 8 %. For economy, the following subbase design was determined.

Upper subbase 100 mm : Cement treated sandy gravel with 1.5 % cement content.,

Lower subbase 140 mm: Non treated sandy gravel.

d) Subbase Material Specification for the Construction

Grading Requirements :

The grading of these sandy gravels is shown in Fig. 8-5*. These gradings are rather fine and mostly out of DOW specification grading range limits. For quality control during construction, a revised grading range for subbase material specification is necessary.

Following preferred grading requirements was proposed for the subbase of this section.

Grading Requirements for
Upper Subbase and Lower Subbase

Sieve Size	Percent by weight passing	
	DOW Specification Type B	Preferred Specification by JICA Study Team
75 mm	100	100
37.5 mm	60 - 100	80 - 100
19.0 mm	40 - 80	65 - 90
9.5 mm	30 - 60	50 -80
4.75 mm	20 - 45	40 - 65
2.36 mm	15 - 35	30 - 55
425 um	8 - 22	10 - 25
75 um	3 - 12	3 - 15

Upper Subbase material (Cement treated Subbase) :

Plasticity Index : the fraction of cement treated material passing 425 μm sieve shall have a Plasticity Index not greater than 10.

Soaked CBR : the cement treated material shall have a CBR value of not less than 25 % after 4 days soaking when compacted to at least 95 % maximum dry density as determined by AS 1289 Test NO. E1. 1.

Lower Subbase Material (Untreated Subbase) :

Soaked CBR : Sandy gravel shall have a CBR value of not less than 8 % after 4 Days soaking when compacted to at least 95 % maximum dry density as determined by AS 1289 Test No. E1. 1.

Prime Coat :

The compacted cement treated subbase shall be cured with a bituminous curing seal applied as soon as possible after the completion of final rolling. For this curing seal, DOW specification for Bituminous Prime Coat was applied for this design.

3.4 Base Course

a) Cement Treated Base Course

Crushed stone base course materials are generally difficult to obtain in the Lot II area. The available source is Ebao rock quarry which is outside of this lot and about 100 km from the lot center. Cement treated base course using sandy gravel from Malalaua hills was planned for this section.

b) Grading Requirements

Road Note 31 states that soil stabilized with cement is an excellent material for constructing the base of most bituminous-surfaced roads in tropical and sub-tropical countries if it has a uniformity coefficient greater than 5, and preferably greater than 10. Uniformity coefficient is given by $Cu = D_{60} / D_{10}$

where D_{60} : the particle size which 60 percent of soil particles are equal to or smaller than

where D_{10} : the particle size which 10 percent of soil particles are equal to or smaller than

The sandy gravels from Malalaua hills have D_{60} of about 5 - 10 mm, and D_{10} of about 0.2 - 0.6 mm. Therefore, Cu (D_{60}/D_{10}) is about 20 - 15 percent, that is greater than 10 percent. This means that the expected sandy gravel materials have good grading for cement stabilization.

In Japan, cement treated base course using pit-run sandy gravel is widely used in all type of road construction. Japan Road Association "Manual for Design and Construction of Asphalt Pavement" provides the following desirable grading range of materials for a cement treated base course.

Desirable Grading for Materials for Cement Treated Base Course
(Japan Road Association)

Sieve Size	Percent by weight passing
50 mm	100
40 mm	95 - 100
20 mm	50 - 100
2.5 mm	20 - 60
0.074 mm	0 - 15

The actual grading of Malalaua sandy gravel falls in this recommended grading range. However in order to assure the good quality of base course, the grading requirements of DOW specification for Base Course type B (38 mm) is to be applied for this design.

c) Cement Content

The cement content of cement treated soils is usually judged on the results of unconfined compressive strength tests of the cement treated samples compacted to the density expected to be attained in the field and cured seven days before testing.

This test was carried out based on the testing method AS 1141.51 unconfined compressive strength of compacted bound materials. Samples were compacted by the method AS 1289 Test No. E 2.1 (Modified compaction).
The compacted cement treated samples were cured 7 days in moist conditions at room temperature of 20 ~ 25°C and immersed in water 4 hours before testing.

Summary of Base Course Material Test Results

Items	Malalaua Sandy Gravel		DOW Specification for Non Treated Base
LL	24	28	not more than 25
PI	11	7	not more than 6
MMD t/m^3	2.140	2.030	
OMC %	8.9	11.1	
Soaked CBR %	15	23	no less than 80
Unconfined Compressive Strength kg/cm^2			Road Note 31 Cement treated Base Recommended Criteria
Cement 2 %	9.0	6.0	
3%	13.6	12.6	$18kg/cm^2$
4%	14.0	16.2	
5%		20.4	

The results shows that non treated material does not comply with DOW specification for Base Course. For cement treated base material, Road Note 31 recommends the criteria of unconfined compressive strength of $250lb/in^2$ ($18kg/cm^2$) after 7 days curing. From the test results, the cement content corresponding to the unconfined strength of 18 kg/cm^2 is about 4.5 % on the cement content - strength curves as shown in Fig. 8-6.

The actual cement content should be determined by conducting the necessary material laboratory tests and trial paving construction at the plant site or construction site prior to the commencement of the works to confirm the compatibility of materials and cement content with the project requirements, and the related experience in this country or neighboring countries.

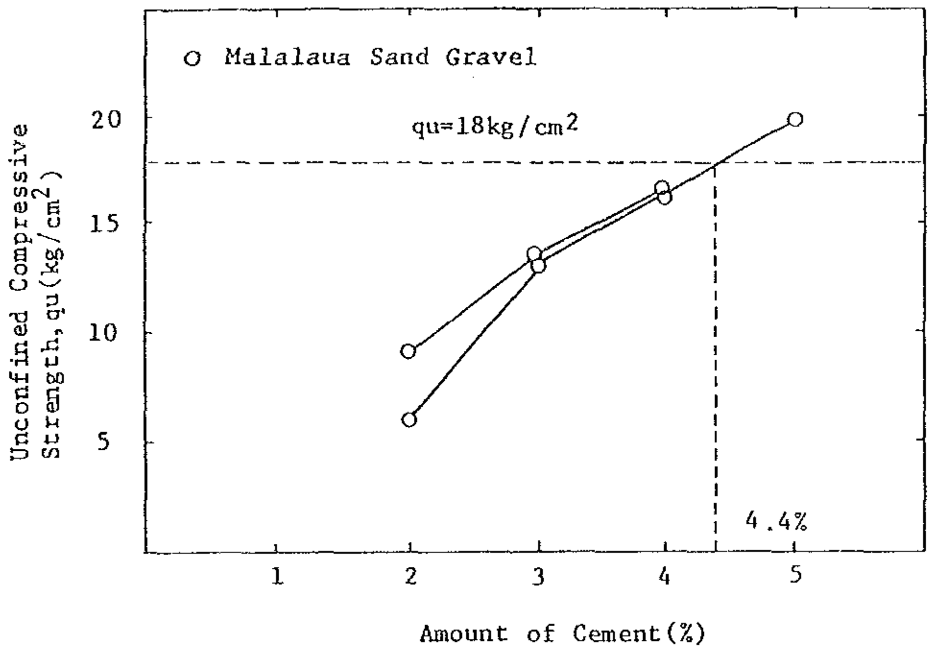


Fig. 8-6 BASE COURSE IMPROVED BY CEMENT

According to much experience in cement treated base course constructions using sandy gravel with light clays, a cement content of 3 percent to 4 percent is considered to be a suitable amount to attain a durable base course without any problem of causing shrinkage cracks.

The production of cement treated sandy gravel should be done by central plant mixing method. Sandy gravel from borrow pits should be screened into coarse and fine components by screens and, then blended in appropriate proportions having a combined grading which complies with the requirements.

d) Cement Treated Base Course Material Specification for the Construction

Grading Requirements :

Following DOW specification for Base course material type B (38 mm) was applied for this design.

DOW Specification Grading Requirements
for Base Course (Type B)

Sieve Size	Percent by weight passing
37.5 mm	60 - 100
19.0 mm	40 - 100
9.5 mm	30 - 60
4.75 mm	20 - 45
2.36 mm	20 - 45
425 um	10 - 25
75 um	3 - 15

Plasticity Index : The fraction of cement treated material passing 425 µm sieve shall have a plasticity Index not greater than 6.

Unconfined Compressive Strength :

The cement treated material shall have an unconfined compressive strength of 18 kg/cm^2 after 7 days curing when compacted to at least 95 % maximum dry density as determined by AS 1289 Test No. E 2.1, and tested by AS 1141.51 unconfined compressive strength of compacted bound materials.

Prime coat:

Same as of the cement treated subbase.

Cutback bitumen : AMC 0 or mixture of approximately 78 parts aviation kerosene or power kerosene with 100 parts of Class 170 bitumen. Application rate, 0.4 ~ 0.8 l/m^2

Blotter material : Clean, dry sand or stone screening. Application rate, 0.3 ~ 0.5 kg/m^2.

3.5 Surface

Double Bituminous Surface Treatment (DBST) was designed based on DOW specification for Bituminous Surface Treatment.
The treatment will be as follows:

First seal:

Cover aggregate : 19 mm 65 - 85 m^2/m^3
Bituminous material : Bitumen 170 class 1.25 - 1.65 l/m^2

Second seal:

Cover aggregate : 9.5 mm 100 - 135 m^2/m^3
Bituminous material : Bitumen 170 class 0.6 -1.0 l/m^2

The actual application rate of bitumen binder and cover aggregates shall be determined by the method set out in Road Note 39 - Recommendations for Road Surface Dressing published by the TRRL, UK.

Cover aggregate :

Crushed stone produced from Tauri River coarse gravel is proposed based on the following test results:

Material Quality Test Results for Cover Aggregates

Test Items		Tauri River Coarse gravel	DOW Specification
Specific Gravity (SSD)	t/m^3	2.6	—
Water Absorption	%	2.9	5.0 Max.
Abrasion Loss	%	24	12 Max.
Sodium Sulphate Soundness	%	5.7	12 Max.

3.6 Proposed Pavement Design

The pavement design for the lot from Miaru river to Malalaua is proposed as shown in Figure 8-7.

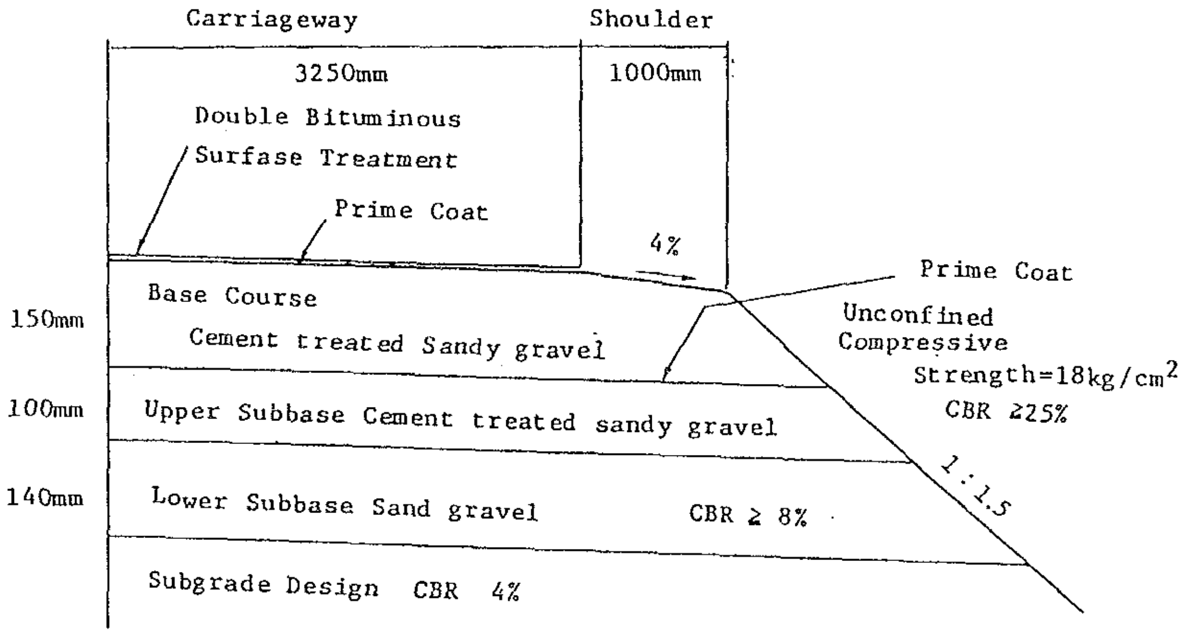


Fig. 8-7 PAVEMENT DESIGN PROPOSED FOR LOT II

CHAPTER IX METHOD OF QUANTITY COMPUTATION

1. QUANTITY OF ROAD WORKS

1.1 General

The main quantities of road works are not much different from those calculated by the Cardno & Davies Study. Methods or bases of quantity computation by the JICA study team are given below.

1.2 Base of Quantity Computation

1.2.1 Clearing & grubbing

The area of clearing extends to the borderline of the right of way : 40m as a standard, the grubbing area is;

1) up to 2.0m outside the formation width in general

2) up to 4.0m from toe of slope at sections where Geotextile is spread on soft ground.

The definition of Swamp/Land area of the clearing works were determined based on observations by the JICA study team during the Study.

The classification of Dense/Light jungle or Grass area of the clearing works were determined from the marks which are shown on the plan : Forest mark - Dense jungle, scattered trees - Light jungle and other - Grass.

1.2.2 Earth works

1) Sand mat

Depth of water in swamp and estimated settlement depth determined the area and thickness of sand mat layer.

Width of Geotextile is extended on both sides 2.0m from the toe of the slope.

2) Unsuitable material

Unsuitable material was estimated from the road surface conditions which were calculated by reference to the grubbing works, however, the location of unsuitable material is not indicated in the tender design.

It is planned that 1.3m of peat at Alika swamp shall be removed and included in the volume of the specified unsuitable material.

3) Earth work volume

The main earth work volume was computed by sections divided at change points but were cut and fill, and at bridge sites or where the thickness of sand mat layer changes.

The volume to be increased due to settlement is inclusive of settlement and the quantity certificate can be made on embankment site with observation of settlement. The settlement volume estimated is inclusive until pavement work is started on each soft ground embankment as discussed in Chapter VI.

4) Settlement observation

Settlement plates will be set up every 250m on the road centerline, and on both shoulders together with survey points for observation of any lateral movement of the road bank base.

1.2.3 Pavement works

The work volume was calculated by sections divided at bridge sections and totalled as the tender volume.
The quantity specified by range was calculated with the average value of the range with a suggestion by DOW.

The quantity of road signs is based on the A.S.

1.2.4 Pipe culvert works

Work quantities for pipe culverts were totalled by each diameter; each length of pipe was calculated from the center of pipe with reference to the slope of the embankments and the degree of skew to road centerline respectively. The effect of deformation due to settlement was not considered.

2. QUANTITY OF BRIDGE WORKS

2.1 General

Quantities of bridge works are quite different from those of the Cardno Davies Study due to changes in the type, width and available data for foundation design, cancelling of five (5) bridges was also a significant change.

For the acceptance of alternative bridge type which will be proposed by Contractors, bridge works have been completely separated from road works with preparation of a bridge section in each tender lot.

Each bridge section was extended in each direction from the abutments abutment by about 10m to include the associated road embankment, backfill to abutments and river protection works etc.

2.2 Main Works

Piling works, pile caps, abutments, steel girder works and slab concrete works were calculated respectively and totalled as tender quantities. Bridge accessories were also listed in the design.

2.2.1 Test piles

A testing pile was allowed for at each bridge, supply of steel pipe and driving of piles were included in the main pay items.

2.2.2 Drifting wood prevention pile

Drifting wood prevention piles were provided for at Miaru, Lake Kamu and Tauri bridges, however, quantities of the piles are included in the main pay items except for the rubber buffer.

2.3 Other Works

Excavation / Borrow, pavement works, and road signs were excluded from the quantities of bridge section and were compiled with the road works. Other related works were as follows:

2.3.1 Back filling to abutments

Quantities of road embankments in bridge sections were calculated together with the selected back filling volume behind each abutment.

Bearing units to prevent differential settlement were calculated by total length of timber, and the approach slabs to be set beneath the pavement were measured in terms of concrete and steel bars.

2.3.2 River protection works

Gabion and Reno mattresses were designed only for Taiena, Ungong and Agobino creeks to protect them from erosion due to quick flood flow. To prevent meandering of the Miaru River, the Reno mattress was designed only for the Malalaua side bank.

The detailed information as to quantity computation is given in the "Quantity Calculation Paper" in Attachment - 3 of the Appendix.

CHAPTER X COST ESTIMATES

The cost estimates have been made by JICA study team. Description of the Project cost was excluded from this "Design Report", and is reported in three (3) separate volumes :

A. Cost Estimates Report

The report describes the total project cost, basic data for cost estimates, estimated cost by Group of Specifications, Foreign Currency component of the Cost and the disbursement plan of the total project cost, an prepared under "Confidential" cover.

B. Priced Bill of Quantities

The priced Bill of Quantites is prepared in one volume for each Lot under "Confidential" cover.

C. Breakdown of Unit prices

The breakdown of each unit price is prepared in one volume for each Lot under "Confidential" cover.

CHAPTER XI IMPLEMENTATION OF THE PROJECT

1. GENERAL

Taking into account the necessary procedures, financial arrangements, preparatory works, tendering process and estimated progress of construction, as well as weather conditions on the site, the possible work schedule of the Project is shown as follows :

January, 1990	Completion of Detailed Design
February, 1990 Onward	Financial Arrangement, Land Acquisition, Prequalification
September, 1991	Completion of Land Acquisition
December, 1990	Tendering
September, 1991	Award of Contract and Commencement of Lot I and Lot II together
September, 1994	Completion of Lot I (36 months)
September, 1995	Completion of Lot II (48 months)

For the construction planning of the Project, the following are assummed :

(1) The Employer of the Project will be the Independent State of Papua New Guinea (PNG) Government.

(2) Consulting Engineering firms will be employed for supervision of the Project.

(3) Financial sources will be the national budget of PNG and a loan from OECF of Japan.

(4) The method of selecting Contractors will be international tendering. Contractors will be selected from the Eligible Source Countries as given in the "Guidelines for Procurement under the Loan (OECF)."

2. CONSTRUCTION LOTS DIVISION

2.1 Division into 2 Lots

The project road of about 80 km has been divided into 2 construction lots which are considered to constitute construction units suitable for international tendering.

The 2 lots on the project road are as follows :

Lot I : Bereina to Miaru River Section
CH.0+000 – CH.33+500 (L=33.500 km)

Lot II : Miaru River to Malalaua Section
CH.33+500 – CH.80+596 (L=47.096 km)

In dividing the road into the above 2 lots, the following factors have been taken into account :

(1) The character of each work site is quite different from the other ; one is a hilly road construction and the other is a swamp or soft ground road construction.

(2) Lot I can be approached by land access and Lot II mainly by water and air access, except for the existing road from Bereina to Apanaipi which can be used for both lots.

(3) Due to the location of available construction material sources in each work site, the construction of each lot can be independently managed without much interference with each other.

Although the work quantities are not balanced between the above 2 lots, it was considered preferable to divide the work into the hilly construction section and the swamp area construction section. The first section will cover the section from Bereina to Miaru River, 33.5 kilometers ; and the second section will cover from Miaru River to Malalaua, about 47 kilometers.

It has been agreed that the bridge works shall be combined with the road construction works.

If the bridge works were separated from the road works, it would be very difficult to control and supervise work progress and the construction schedule without claims arising from each contractor. Since the construction schedule will inevitably be affected by unforeseen weather conditions, especially in the wet season. Therefore the road and bridge work is combined for better coordination under one constructor for each lot to ensure smooth progress of the work as a whole.

Lot I will be relatively easy to construct. This will give a good opportunity to local contractors. Lot II being mostly in swamp and including six major bridges will be the most difficult section, and only experienced contractors will be considered.

2.2 Work Components of Each Lot

The construction work components in each lot are classified in the following groups :

Lot I
- Group 1 - General
- Group 2 - Preliminaries
- Road Work (Groups 3 - 18)
- Bridge No.1 - Taiena Bridge (Groups 3 - 18)
- Bridge No.2 - Agobino Bridge (Groups 3 - 18)
- Bridge No.3 - Ungongo Bridge (Groups 3 - 18)
- Group 19 - Dayworks

Lot II
- Group 1 - General
- Group 2 - Preliminaries
- Road Work (Groups 3 - 18)
- Bridge No.4 - Miaru Bridge (Groups 3 - 18)
- Bridge No.5 - Kapuri Bridge (Groups 3 - 18)
- Bridge No.6 - Lakekamu Bridge (Groups 3 - 18)
- Bridge No.7 - Tauri Bridge (Groups 3 - 18)
- Bridge No.8 - Makara Bridge (Groups 3 - 18)
- Bridge No.9 - Sappaharo Bridge (Groups 3 - 18)
- Group 19 - Dayworks

These Groups as stated above are described in the Specifications.

3. PLAN OF CONSTRUCTION METHOD

3.1 Basic Conditions to be Considered

The construction method and sequence have been planned to take into amount the Expected mode of construction and the target period for construction.

Taking into consideration, in addition to the above, the construction conditions such as availability of construction forces, weather conditions, geological site conditions, and the topographical site conditions, the mechanized construction method is applied in principle.

3.1.1 Workable days and hours for the construction works

A rainy season from December to April is observed in the Project area. The workable days are estimated on the basis of monthly rainy days reported at Bereina rainfall station from 1978 to 1987. The workable days are estimated to exclude suspended days due to rainfall, national holidays and Sundays as follows :

- Earthwork

Dry season	22 days / month
Wet season	14 days / month
Yearly	19 days / month

The earthwork on the land area is planned to be carried out through dry and wet seasons with 19 workable days / month

The earthwork on the swamp area is planned to be carried out during the dry season with 22 workable days / month (7 months)

- Bridge work

  Dry and wet seasons, 19 days / month

- Pavement work

  Dry season (7 months), 21 days / month

Actual working hours per day is planned to be 7 hours out of 8-working hours per one shift.

3.1.2 Coefficient of earth volume conversion

The following coefficients of earth volume conversion for earthwork planning are applied :

	Natural Volume	Loose Volume	Compacted Volume
Type A (rock)	1.00	1.60	1.05
Type B (ripping)	1.00	1.45	0.95
Type D (soil)	1.00	1.20	0.85

3.1.3 Construction materials

The sources of construction material are basically planned for each project site as follows :

a) Lot I work site

Fill material (embankment) :
Excavated material from the road along Inapi ridge

Subbase material :
Subbase borrow pit No. 1 (Bereina)
Subbase borrow pit No. 2 (Babanonga)

Base material :
Quarry site No. 1 (Eboa quarry)

Concrete aggregate, cover aggregate, rubble and cobble:
River deposit No. 1 (Angabangan river)

b) Lot II work site

Fill material (Embankment) :
Excavated material of Apanaipi ridge,
Palipalla hill and Ilavala hill

Fill material (embankment, cut in borrow) :

Stock pile No.1 (Spoil bank No. 1 for Lot I), CH33+400
Borrow pit No. 1 (Apanaipi ridge), CH34+200
Borrow pit No.2 (Palipalla hill), CH49+200
Borrow pit No.3 (Palipalla hill), CH54+000
Borrow pit No.4 (Ilavala hill), CH64+800
Borrow pit No.5 (Malalaua)

Sand mat material :

Sand borrow pit No. 1 (Mirapo - Iokea village)
Sand borrow pit No. 2 (Ilavala hill)
Sand borrow pit No. 3 (Koaru)

Subbase material :

Subbase borrow pit No. 3 (Malalaua village)

Base material :

Base borrow pit No. 1 (Malalaua)

Concrete aggregate, cover aggregate, rubble and cobble:

River deposit No. 2 (Tauri river)

Concrete aggregate for Miaru bridge and rubble/cobbles for Reno mattress of Alika swamp are planned to be supplied from River deposit No. 1 (Angabangan).

Locations of material sources discussed above are shown in Figure 11-1*.

3.2 Preparatory Works

3.2.1 Mobilization and access to site

Transportation routes for construction equipment and materials including bridge materials are available to the Project site as follows :

Lot I - Port Moresby to Bereina (Land transportation)
- Bereina to Babanongo (Land transportation)
- Bereina to CH9 + 000 (Land transportation)

Lot II - Port Moresby to Miaru River through Bereina and Iokea (Land transportation)
- Port Moresby to Lese Oarai (Sea and estuary transportation)
- Port Moresby to Terapo Mission (Sea and river transportation)
- Port Moresby to Port Kerema (Sea transportation)
- Port Kerema to Malalaua (Land transportation)

3.2.2 Construction of access road

The existing road of Bereina - Iokea - Apanaipi shall be first upgraded and improved before commencement of the main works. Other existing roads will be also improved and maintained during each construction period.
New access roads extended from the existing roads will be properly provided at the following work sites.

Lot I : Babanongo, CH9 + 000 point, river deposit No. 1 (Angabangan river), quarry site No. 1 (Eboa quarry), subbase borrow pits No. 1 and No. 2 and CH30 + 000 from the end point.

Lot II : Lese Oalai, borrow pits No. 1 to No. 5, sand borrow pits No. 1 to No. 3, subbase borrow pit No. 3 and base borrow pit No. 1

3.2.3 Maintenance of existing roads

Maintenance of existing roads during construction is proposed as follows :

1) Maintenance of main road access (170 km) from Port Moresby to Bereina will be excluded from the duty of the Contractor for the Project. The road will be still in the hands of DOW.

2) Maintenance of the road from Bereina to Iokea (49 km) will also be excluded from the duty of the Contractor for the Project.

3) Maintenance of the road from Iokea to Apanaipi (13 km), however, will be handed over to the Contractor for the Project including initial improvement costs as an access road.

4) Other roads related to hauling of quarry material in Bereina to Miaru river section shall be included in the duty of the Contractor for the Project.

5) From the same point of view, maintenance of the road from Kerema to Malalaua (60 km) (up to the junction of road to Koaru) will be done by DOW including some improvement of bridges, if required.

6) Maintenance of the road (25 km) to Koaru village and to the end point of the Project in Malalaua town shall be included in the duty of the Contractor for the Project including the initial improvement cost for road and bridges to be required.

3.2.4 Site offices and camp

The site offices and camps for Contractors can be provided at various work sites as follows :

Lot I - Main office : Bereina
- Branch offices : Miaru river (Apanaipi), Babanonga and CH9 + 000 site

Lot II - Main office : Malalaua
- Branch offices : Terapo Mission, Lese Oalai and Miaru River (Kwaba school)

Engineer's offices and camps can be set up at following sites :
- Main office : Bereina for Lot I , Malalaua for Lot II
- Branch office : Miaru river (Kwaba school) and Terapo Mission

3.3 Clearing and Grubbing

The 40 m wide road site will be cleared along the alignment in advance of road excavation and embankment. The clearing and grubbing works will be carried out as follows :

a) Land area (Hilly construction section) :
Trees will be cut down by chain saw and manpower. Roots and grasses will be grubbed by 15 ton bulldozer, gathered and burned.

b) Swamp area :
Clearing and grubbing works will be carried out in the dry season in principle. In un-inundated areas, 10 ton low ground pressure type bulldozers are suitable. Trees, shrubs and grasses in water of 0.5 to 1.0 m depth will be cut by chain saw and manpower. Their roots will be taken out by using an amphibious soft ground vehicle with 0.4 m^3 backhoe and/or clamshell.

c) Alika swamp :
Grasses growing in water 1.8 m deep will be taken out from the river bed formation, by using a 0.4 m^3 clamshell crane set up on a pontoon. The grasses will be discharged onto small boats and removed.

3.4 Earthwork

3.4.1 Lot I work section

Bereina to Miaru river section is divided into four work sections ; Bereina to Taiena, Taiena to Agobino, Agobino to Ungongo and Ungongo to Miaru river.

The road embankment is to be performed by cutting and filling works. The excavated material shall be used for embankments as much as possible. As a result of mass curve planning and earth distribution, about 90 percent of cut material will be filled within the 500 m hauling distance. The earthworks can be operated through dry and wet seasons.

Bereina to Agobino section will be worked from three points : both sides and CH9 + 000 point simultaneously.

As for Agobino to Miaru river section, four work sites ; from Agobino side and two work sites from the end point can be considered seeing the topographical conditions which can provide access for temporary pilot roads. The above plan is illustrated in Figure 11-2*.

Earthmoving work will be planned by using an equipment fleet comprising a 21 ton bulldozer, 21 ton bulldozer with ripper, 2.3 m^3 tractor shovel and 11 ton dump truck. The excavated material will be spread by 11 ton bulldozer and compacted by 8 ton vibrating roller and 15 ton tired roller.

Surplus cut soil of 28,000 cu.m compacted volume will be hauled to the spoil bank No. 1, No. 2, and No. 3. This stocked material is to be partly used for the embankment of Lot II construction.

The earthworks of Lot I are scheduled to be completed by November in the 3rd year to secure the following pavement works :

3.4.2 Lot II work section

Miaru river to Malalaua section is divided into ten work sections ; beginning point to Miaru, Miaru to Alika, Alika swamp, Alika to Palipala hill, Palipala to Kapuri, Kapuri to Lakekamu, Lakekamu to Tauri, Tauri to Makara, Makara to Sappaharo and Sappaharo to Malalaua. About 60 percent of the road alignment of Lot II is situated in the swamp area.

Earthwork in the land area up to CH54+000 will be planned by the same method and same equipment fleet applied for Lot I construction. As a result of mass curve planning and earth distribution, the stockpile No. 1 and three borrow pits No. 1 to No. 3 are required.

Two work sites will be provided in Apanaipi ridge and four work sites including the two cut in borrow work sections (No. 2 & No. 3 pit) are necessary in Palipala hill. About 80 percent of the fill material will be transported within the 500 m hauling distance. This road section is scheduled to be completed by the mid-4th year.

Earthworks in the swamp area will mainly comprise road embankment and cut in borrow works. The cut in borrow material will be obtained from three borrow pits : No. 3 (Palipala hill), No. 4 (Ilavala hill) and No. 5 (Malalaua hill). The cut in borrow work will be carried out by using the same equipment as applied for the excavation work in the land area. The material hauled from borrow pits will be spread by 10 ton low ground pressure type bulldozer and be compacted by 15 ton tired roller. The embankment work will be made during the dry season of seven months. This road section will require three dry seasons from the 2nd year to the 4th year due to disturbance of the wet season and the bridge construction required.

Before starting the embankment work in the swamp area, geotextile sheeting and sand mat spreading will be carried out on the soft ground. The sand mat material will be transported from three sand borrow pits of No. 1 to No. 3. Sea sand will be loaded by 0.6 m^3 backhoe into 8 ton dump trucks for hauling to the embankment site. While Ilavala sand will be excavated and transported by the same equipment applied for the land area. The sand material will be spread and compacted by 10 ton low ground pressure type bulldozer.

The above plan is illustrated in Figure 11-3* and 11-4*.

3.5 Pavement Works

3.5.1 Subbase course

Subbase course work will be performed for two dry seasons on the last stage of the construction period. Two subbase borrow pits of No. 1 (Bereina) and No.2 (Babanongo) are available for Lot I construction and one subbase pit No. 3 (Malalaua) for Lot II. As for Lot II subbase course works, the material transportation will be planned to pass on the completed road embankment and completed bridge structures or their temporary stages.

a) Lower Subbase

The lower subbase material will be excavated in the borrow pits and hauled to the work site by 21 ton bulldozer, 2.3m^3 tractor shovel and 11 ton dump truck. The material will be spread by 3.7 m motor grader and compacted by 15 ton tired roller and 10 ton macadam roller.

b) Upper Subbase

The upper subbase material will be obtained from the same borrow pits as for the lower one and mixed with cement on site. The borrowed material of sandy gravel will be spread and leveled by 3.7 m motor grader. Cement will be spread uniformly on the road surface by manpower and spike harrow. Cement and sandy gravel soil will be mixed by 1.6 m wide road stabilizer and compacted by 15 ton tired roller and 10 ton macadam roller.

After clearing the upper subbase surface, the prime coating will be carried out. Cut-back bitumen will be melted in a 6,000 litre kettle and sprayed by 4,000 litre distributor and 600 litre engine sprayer. Blotter material of sand will be spread by manpower and rolled by 15 ton tired roller.

3.5.2 Base course

Following the subbase course work, the base course work will be carried out for two dry seasons. The base course material sources are located at the site No. 1 (Eboa Quarry) for Lot I and the base borrow pit No. 1 (Malalaua hill) for Lot II. The base course structure is planned to be crushed rock base course for Lot I and cement treated base course for Lot II.

The rock of quarry site No. 1 will be crushed by a portable crushing plant of 60 ton/hr. the raw rock material will be excavated by blasting after drilling by 10m^3/min crawler drill with 13 m^3/min air compressor. The rock fragments will be gathered, loaded and hauled to the crushing plant by 21 ton bulldozer with ripper, 2.3 m^3 tractor shovel and 11 ton dump truck.

As for the Lot II construction, the base course material will be combined by mixing with cement. The mixing will be made by 100 ton/hr portable mechanical stabilization plant. The hauling of sandy gravel to the plant will be carried out by the same equipment as used the lower subbase work.

Loading and hauling from both plants will be made by 1.6 m^3 wheel loader and 8 ton dump truck. The crushed rock will be spread by 3.5 m wide aggregate spreader in the Lot I. The mixed product for cement treated base course will be spread and leveled by 3.7 m motor grader in the Lot II. The rolling compaction will be made by 15 ton tired roller and 10 ton macadam roller.

After the base course layer, the prime coating will be carried out by same method as applied for the subbase course work.

3.5.3 Bituminous surfacing

Bituminous surfacing work will also be carried out over two dry seasons. The type of bituminous surfacing is planned to be the double bituminous surface treatment. Cover aggregate of 1st and 2nd layers are to be 19 mm size and 9.5 mm size respectively.

The cover aggregate sources are located at the river deposit No. 1 (Angabangan river) for Lot I and the river deposit No. 2 (Tauri river) for Lot II. The cover aggregate will be produced by 20 ton/hr portable crushing plant. These crushing plants will be installed at river deposit site No. 1 for Lot I and Terapo Mission for Lot II, the raw material of Lot II will be transported by 10 m^3 small ship through the Tauri river.

Bitumen heated by 6,000 litre asphalt kettle will be transported by 4,000 litre asphalt distributor and sprayed on the base course surface. The cover aggregate will be transported by using 1.6 m^3 wheel loader and 8 ton dump truck from the crushing plant.

Within 15 minutes after spraying bitumen, the cover aggregate will be spread by 2 m wide chip spreader and compacted by 15 ton tire roller and 15 ton tandem roller. The second layer bituminous surfacing will be carried out by the same surfacing method to follow the previous 1st layer work.

3.6 Bridge Works

3.6.1 Lot I Work site

Lot I includes three bridges ; bridge No. 1 (Taiena), bridge No. 2 (Agobino) and bridge No. 3 (Ungongo). All the bridge materials and equipment will be transported by trailer from Port Moresby.

Piling work of 500 mm dia. steel pipe will be made by 2.5 ton diesel pile driver with 35 ton crawler crane. The concrete for the abutment and deck slab will be mixed by 0.6 m^3 portable concrete mixer and be transported by 0.7 m^3 concrete dumper. the concrete aggregate will be obtained from river deposit No. 1 (Angabangan river).

H shaped rolled girder will be assembled at each bridge site and erected directly by 35 ton crawler crane set up at the creek banks.

3.6.2 Lot II work site

Lot II includes six bridges ; bridge No. 4 (Miaru), bridge No. 5 (Kapuri), bridge No. 6 (Lakekamu), bridge No. 7 (Tauri), bridge No. 8 (Makara) and bridge No. 9 (Sappaharo).

a) Bridges No. 8 and No. 9
   Both bridges will be constructed in early stages of the construction schedule to facilitate the embankment work for Malalaua to Tauri section. Bridge material and equipment will be transported by barge from Port Moresby to Port Kerema, and then by trailer from Port Kerema to the bridge sites.

   Piling work of 500 mm and 800 mm dia. steel pipe will be carried out by 3.5 ton diesel pile driver with 40 ton crawler crane. The piling equipment will be installed on a portable pontoon and the pier steel pipe will be driven from the pontoon. The concrete mixing and hauling will be made by 0.6 m^3 portable concrete mixer and 0.7 m^3 concrete dumper. After driving the steel piles located at the pier portion, the concrete and bars will be placed inside the piles. The concrete aggregate will be transported by river channel from Tauri plant.

H shaped rolled girders will be assembled at site and the erection of girders will be made by 35 ton crawler crane.

b) Bridge No. 4
The Miaru bridge is scheduled to be constructed at an early stage to commence the earthworks in Apanaipi ridge and Alika swamp sections. Bridge materials and equipment will be transported by trailer through Bereina-Iokea road from Port Moresby.

Piling work of 600 mm & 800 mm dia. steel pipe will be carried out by 3.5 ton diesel pile driver with a 40 ton crawler crane. A temporary staging of H-300×300mm piles will be required for this bridge construction. The piles for the temporary staging and bents will be driven by 60 kw vibrating hammer. Concrete works will be carried out by the same method. The concrete aggregate will be obtained from river deposit No. 1 (Angabangan) provided for Lot 1 construction.

Built up steel girder will be erected on the bents by 40 ton crawler crane.

c) Bridge No. 5
All staging materials and equipment used for the Miaru bridge construction will be transferred to the Kapuri bridge site through Iokea-Lakekamu-Kapuri. The bridge construction will be carried out by the same method applied for the Miaru bridge.

H-shaped rolled girder will be assembled on the temporary staging and erected by 40 ton crawler crane on the staging.

d) Bridges No. 6 and No. 7
The Lakekamu bridge is scheduled to be constructed in parallel

with Kapuri bridge construction to perform the embankment work between Lakekamu to Tauri section. After the completion of Kapuri bridge, all staging materials and equipment will be transferred to Tauri bridge.

Both bridges will require temporary staging in the rivers. This temporary staging is necessary for the transportation of embankment and pavement materials. H-300×300 mm piles will be driven by 3.5 ton diesel pile driver on the portable pontoon.

Piles of 800 mm dia. are planned for the steel pipe piling at abutments. Bored cast-in-place concrete piles of 1.000 mm will be installed at piers. After installing casing pipe in the water, the bored hole will be drilled in the limestone formation by mechanical pile boring machine. Reinforcing bars will be set up in the bored hole and casing, and will be concreted through a tremie pipe.

Built up steel girder will be erected by launching girder method.

3.7 Construction Equipment

The main construction equipment requirements are listed in Tables 11-1* and 11-2* respectively.

The types and number of construction equipment for Lot I are mainly based on the cutting and filling works in the hilly area. On the other hand, those for Lot II are mainly based on the needs of the swamp area works with due consideration of embankment and sand mat spreading.

The equipment required for pavement works will be a conventional pavement equipment fleet.

The bridge works can be undertaken by any modern mechanized construction method.

4. CONSTRUCTION SCHEDULE

4.1 Pre-Construction Program

Pre-construction activities will comprise detailed design, preparation of tender documents, financial arrangements, selection of consultants, pre-qualification of contractors, tendering, evaluation and award.

The detailed design and preparation of tender documents was completed by the end of January 1990. The financial arrangements are expected by the end of August 1990, six months after submission of the Implementation Program. The selection of consultant will also be made before starting tendering.

Pre-qualification of contractors will require three months before the tender call. The tendering will take ten months by the end of September in 1st year as shown in Fig. 11-5.

All required land acquisition and compensation shall be arranged by the DOW, before the commencement of construction works of Lot I and Lot II.

4.2 Construction Schedule and Target Date

The construction schedule is tentatively established in due consideration of weather conditions, construction methods, work progress rate, site conditions, etc.. The overall construction schedule is shown in Fig. 11-5. the commencement dates of construction works for Lot I and Lot II are scheduled at the October in 1st year. The time targets of the Project are summarized below.

Lot I :	Construction period	36 months (3 years)
	Commencement	October in 1st year
	Completion	September in 4th year
Lot II :	Construction period	48 months (4 years)
	Commencement	October in 1st year
	Completion	September in 5th year

DESCRIPTION	UNIT	QUANTITY
Detailed Design		
Financial Arrangement		
Selection of Consultant		
Prequalification		
Tender		
Land Acquisition and Compensation		
Lot I Bereina to Miaru River Section		
1. Preparatory Works		
Mobilization		L. S.
Temporary building		L. S.
Access road		L. S.
Borrow pit & quarry site		L. S.
2. Clearing and Grubbing	ha	142
3. Excavation	m^3	829000
4. Embankment	m^3	742000
5. Subbase Course	m^3	75 500
6. Base Course	m^3	44 900
7. Bituminous Surfacing, DBST	m^2	288 300
8. Drainage Work		
9. Road Furniture		
10. Bridge Work		
(1) Bridge No.1 (Tolena Bridge)	m	17
(2) Bridge No.2 (Agobino Bridge)	m	20
(3) Bridge No.3 (Ungongo Bridge)	m	20
Lot II Miaru River to Malalaua Section		
1. Preparatory Work		
Mobilization		
Temporary building		
Access road		
Borrow pit		
2. Clearing and Grubbing	ha	202
3. Excavation		
Excavation	m^3	261000
Cut in Borrow	m^3	392000
4. Sand mat	m^3	172000
5. Embankment	m^3	591000
6. Subbase Course	m^3	104000
7. Base Course	m^3	60900
8. Bitumenous Surfacing, DBST	m^2	395000
9. Drainage Work		
10. Road Furniture		
11. Bridge Work		
(1) Bridge No.4 (Miaru Bridge)	m	90
(2) Bridge No.5 (Kapuri Bridge)	m	64
(3) Bridge No.6 (Lakekamu Bridge)	m	120
(4) Bridge No.7 (Tauri Bridge)	m	120
(5) Bridge No.8 (Makara Bridge)	m	40
(6) Bridge No.9 (Sappahoro Bridge)	m	40

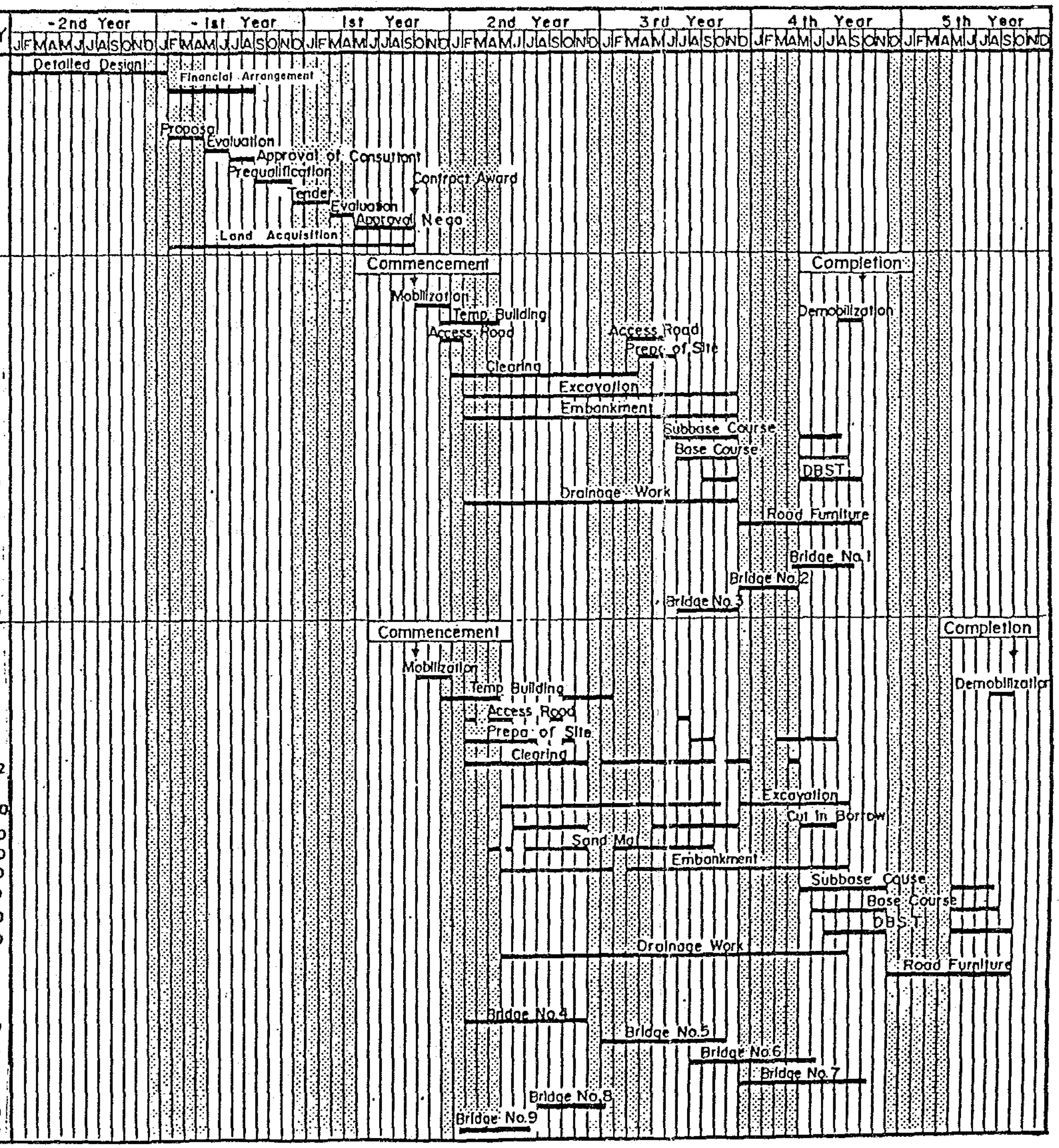


Fig. 11-5 PROPOSED CONSTRUCTION SCHEDULE FOR THE PROJECT

Each work item and it's construction period required are shown as follows.

Lot I :	Mobilization	3 months (October-December in 1st year)
	Clearing and grubbing	15 months (January in 2nd year-March in 3rd year)
	Excavation and embankment	22 months (February in 2nd year-November in 3rd year)
	Subbase course	9 months (June in 3rd year-Mid August in 4th year)
	Base course	9 months (July in 3rd year-August in 4th year)
	Bituminous surfacing	8 months (September in 3rd year-September in 4th year)
	Drainage work	22 months (February in 2nd year-November in 3rd year)
	Road furniture	10 months (December in 3rd year-September in 4th year)
	Bridge No. 1	5 months (July-November in 3rd year)
	Bridge No. 2	5 months (December in 3rd year-April in 4th year)
	Bridge No. 3	5 months (Mid April-Mid September in 4th year)

Lot II :	Mobilization	3 months (October-December in 1st year)
	Clearing and grubbing	23 months (February in 2nd year-April in 4th year)
	Excavation and embankment (inc. cut in borrow)	27 months (May in 2nd year-August in 4th year)
	Sand mat	15 months (April in 2nd year-September in 3rd year)
	Subbase course	10 months (May in 4th year-Mid August in 5th year)
	Base course	10 months (June in 4th year-August in 5th year)
	Bituminous surfacing	10 months (July in 4th year-September in 5th year)
	Drainage work	28 months (May in 2nd year-August in 4th year)
	Road furniture	10 months (December in 4th year-September in 5th year)
	Bridge No. 4	10 months (February-November in 2nd year)
	Bridge No. 5	10 months (January-October in 3rd year)
	Bridge No. 6	10 months (August in 3rd year-Mid June in 4th year)
	Bridge No. 7	10 months (December in 3rd year-Mid October in 4th year)
	Bridge No. 8	5 months (August in 2nd year-Mid January in 3rd year)
	Bridge No. 9	5 months (Mid January-Mid July in 2nd year)

The commencement date of both construction lots is scheduled at the around the middle of the dry season considering mobilization, and improvement of existing access roads which may be impossible to do in the wet season.

The construction works for Lot I will be carried out throughout the year, since the road alignment is planned on land. Also accessibility to work sites is not difficult from Bereina. The construction period is planned to be three years based on the new access points to be prepared.

The construction works for Lot II include road sections situated on land and in swamp. The embankment works in swamp are restricted by the wet season. The construction sequences for earthworks and pavement works are affected by the six bridges construction schedule. The first two dry seasons will be required for sand mat filling including geotextile sheeting and the embankment works will require three dry seasons after that. The last two dry seasons are scheduled for pavement works. The construction period is planned to be four years accordingly.

These plans discussed above were accepted by DOW upon the submission of the interim report No.2 in March 1989.

However, based on the request of PNG Government (Office of International Development Assistance) by the letter (13 - 5 - 2) dated 27 December 1989, an alternative extended construction schedule was included in Attachment - 4 of this report, and an alternative disbursement schedule was shown separately in the Cost Estimate Report.

CHAPTER XII TENDER DOCUMENTS

1. CONTRACT DOCUMENTS

The following volumes forming part of the tender documents have been prepared during the design period :

Volume I (Lot I and Lot II)

Invitation to Tender

Instructions to Tenderers

Appendix to Instructions to Tenderers

Form of Tender Security

Form of Performance Security

Form of Bank Guarantee for Advance Payment

Form of Agreement

Form of Certificate for Source, Origin and Eligibility

Conditions of Contract

Part I General Conditions

Part II Conditions of Particular Application

Volume II-1 (Lot I) and II-2 (Lot II)

Form of Tender
Appendix to Form of Tender
Schedules to the Tender
Conditions of Contract Part III nominated Borrow Pits
Bill of Quantities

Volume III-1 (Lot I) and III-2 (Lot II)

Specification

Volume IV-1 (Lot I) and IV-2 (Lot II)

Drawings

Volume V-1 (Lot I) and V-2 (Lot II)

Setting Out Co-ordinate Schedule

Volume I is common to the two construction lots and contains the Conditions of Contract which have been prepared based on the Papua New Guinea Department of Works Conditions of Contract for Civil Engineering Construction, 1st Edition, April 1988.

The other volumes have been prepared separately for each construction lot.

DOW has issued the Standard Specification for Road and Bridge Works in November 1978. The special specifications for the items not stipulated in the above standard Specification has been separately prepared for respective tenders. Contractors are, accordingly, requested to examine the two (2) volumes of Technical Specifications.

JICA study team, however, proposed to prepare one volume of Specification for each Lot including special items for each contract package and this was accepted by DOW.

2. PREQUALIFICATION DOCUMENT

The prequalification document is common to the two construction lots.

This documents has been prepared based on the DOW prequalification forms normally used for DOW's construction projects, together with minor additions and modifications required for this Project.

3. DRAWINGS

The drawings for the tender documents have been presented in two volumes as stated above.

The list of the drawings are listed below :

DRAWINGS LOT- I

TITLE OF DRAWING	DRAWING NO.
GENERAL DRAWINGS	
SITE AND LOCALITY PLAN	A1 / 87760
ABBREVIATION AND LEGEND	A1 / 87761
PLANS LAYOUT, COORDINATES OF CONTROL POINTS AND INTERSECTION POINTS	A1 / 87762
STANDARD DRAWINGS	
TYPICAL CROSS SECTION (FILL & CUT AND FILL SECTION)	A1 / 87763
TYPICAL CROSS SECTION (CUT SECTION)	A1 / 87764
TYPICAL PAVEMENT SECTION FOR ROAD CH.0+000 – CH.33+500	A1 / 87765
SUPERELEVATION	A1 / 87766
INTERSECTIONS	
INTERSECTION CH.0+200 & CH.0+260 PLAN, LONGITUDINAL AND CROSS SECTIONS	A1 / 87767
INTERSECTION CH.1+450 PLAN AND LONGITUDINAL SECTIONS	A1 / 87768
INTERSECTION CH.1+450 CROSS SECTIONS (A-LINE)	A1 / 87769
INTERSECTION CH.1+450 CROSS SECTIONS (B-LINE)	A1 / 87770
INTERSECTION CH.33+425 PLAN, LONGITUDINAL AND CROSS SECTIONS	A1 / 87771
ROAD FURNITURE	
STANDARD GUARD RAIL	A1 / 87772
GUARD RAIL DETAILS, APPROACH FOR TWO WAY BRIDGE	A1 / 87773
ROAD EDGE GUIDE POST AND ROAD EDGE MARKERS	A1 / 87774
SCHEDULE OF ROAD EDGE GUIDE POST CH.0+000 – CH.33+500 1/2	A1 / 87775
SCHEDULE OF ROAD EDGE GUIDE POST CH.0+000 – CH.33+500 2/2	A1 / 87776
PAVEMENT MARKINGS	A1 / 87777
SCHEDULE OF PAVEMENT MARKINGS CH.0+000 – CH.33+500	A1 / 87778
ROAD SIGNS	A1 / 87779
ROAD SIGN FOR BRIDGE APPROACH AND INTERSECTIONS	A1 / 87780
SCHEDULE OF ROAD SIGNS CH.0+000 – CH.33+500	A1 / 87781

DRAWINGS LOT- I

TITLE OF DRAWING	DRAWING NO.
DRAINAGE	
STANDARD CULVERT HEADWALLS	A1 / 87782
CULVERT BEDDING, SUBSOIL DRAIN AND STANDARD INLET PIT	A1 / 87783
CULVERT SCHEDULE, CH.0+520–CH.12+333	A1 / 87784
CULVERT SCHEDULE, CH.12+760–CH.24+091	A1 / 87785
CULVERT SCHEDULE, CH.24+517–CH.32+950 AND ON SIDE DITCH	A1 / 87786
OTHERS	
EARTHWORKS SCHEDULE, CH.0+000–CH.33+500	A1 / 87787
SPOIL BANKS NO. 1,2 & 3 FOR LOT- I /STOCKPILE NO.1 FOR LOT-II	A1 / 87788
RIVER DEPOSIT NO. 1 AND QUARRY SITE NO. 1	A1 / 87789
SUBBASE BORROW PIT NO. 1 AND NO. 2	A1 / 87790
PROJECT NOTICE BOARD	A1 / 87791
ENGINEER'S OFFICE ACCOMMODATION	A1 / 87792
PLAN, ELEVATIONS, SECTIONS AND DETAILS	A1 / 87793
DOOR & WINDOW SCHEDULE, STAIR DETAILS, SECTION AND JOINERY ELEVATIONS	A1 / 87794
PLANS, ELEVATIONS, ELECTRICAL LEGEND AND WIRING DIAGRAM	A1 / 87795
SECTION DETAILS	A1 / 87796
* RENO MATTRESS AND GABION	A1 / 88072
LIST OF PLAN AND LONGITUDINAL SECTIONS	
FROM CH. 0+000–CH. 0+600	A1 / 87797
TO CH. 33+000–CH. 33+600	–A1 / 87845
CROSS SECTIONS	
FROM CH. 0+000–CH. 0+350	A1 / 87846
TO CH. 33+300–CH. 33+500	–A1 / 88009

*. . . . THE DRAWING FROM LOT-II

DRAWINGS LOT- I

TITLE OF DRAWING	DRAWING NO.
BRIDGES	
BRIDGE NO. 1 - TAIENA BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1 / 88010
GENERAL ARRANGEMENT	A1 / 88011
ABUTMENT CONCRETE & REINFORCEMENT DETAILS	A1 / 88012
CONCRETE DECK DETAILS	A1 / 88013
STEELWORK DETAILS	A1 / 88014
HANDRAILING / IMPACT ANGLE DETAILS	A1 / 88015
BAR BENDING SCHEDULE NOTES & STANDARD DETAILS	A1 / 88016
BEARING BP.B - 103 (FIXED)	A1 / 88017
BEARING BP.B - 104 (MOVABLE)	A1 / 88018
RIVER BANK PROTECTION, BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1 / 88019
BRIDGE NO. 2 - AGOBINO BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1 / 88020
GENERAL ARRANGEMENT	A1 / 88021
ABUTMENT DETAILS	A1 / 88022
CONCRETE DECK DETAILS	A1 / 88023
STEELWORK DETAILS	A1 / 88024
HANDRAILING / IMPACT ANGLE DETIALS	A1 / 88025
BAR BENDING SCHEDULE NOTES & STANDARD DETAILS	A1 / 88026
BEARING BP.B - 103 (FIXED)	A1 / 88027
BEARING BP.B - 104 (MOVABLE)	A1 / 88028
RIVER BANK PROTECTION, BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1 / 88029
BRIDGE NO. 3 - UNGONGO BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1 / 88030
GENERAL ARRANGEMENT	A1 / 88031
ABUTMENT PLAN, ELEVATIONS & DETAILS	A1 / 88032
CONCRETE DECK DETAILS	A1 / 88033
STEELWORK DETAILS	A1 / 88034
HANDRAILING / IMPACT ANGLE DETAILS	A1 / 88035
BAR BENDING SCHEDULE NOTES & STGANDARD DETAILS	A1 / 88036
BEARING BP.B - 103 (FIXD)	A1 / 88037
BEARING BP.B - 104 (MOVABLE)	A1 / 88038
RIVER BANK PROTECTION, BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1 / 88039

DRAWINGS LOT-II

TITLE OF DRAWING		DRAWING NO.
GENERAL DRAWINGS		
	SITE AND LOCALITY PLAN	A1 / 88051
*	ABBREVIATION AND LEGEND	A1 / 87761
	PLANS LAYOUT, COORDINATES OF CONTROL POINTS AND INTERSECTION POINTS	A1 / 88052
STANDARD DRAWINGS		
	TYPICAL CROSS SECTION (FILL & CUT SECTION)	A1 / 88053
	TYPICAL CROSS SECTION (SAND MAT t=0.5m & ALIKA SWAMP)	A1 / 88054
	TYPICAL CROSS SECTION (SAND MAT t=1.0m)	A1 / 88055
	TYPICAL CROSS SECTION (SAND MAT t=1.0m, t=0.5m)	A1 / 88056
	TYPICAL PAVEMENT SECTION FOR ROAD CH. 33+500 TO CH. 80+596	A1 / 88057
*	SUPERELEVATION	A1 / 87766
ROAD FURNITURE		
*	STANDARD GUARD RAIL	A1 / 87772
	GUARDRAIL & FENDER POST DETIALS (APPROACH FOR SINGLE LANE BRIDGE)	A1 / 88058
*	ROAD EDGE GUIDE POST AND ROAD EDGE MARKERS	A1 / 87774
	SCHEDULE OF ROAD EDGE GUIDE POSTS CH. 33+500 – CH.80+596 1/3	A1 / 88059
	SCHEDULE OF ROAD EDGE GUIDE POSTS CH. 33+500 – CH.80+596 2/3	A1 / 88060
	SCHEDULE OF ROAD EDGE GUIDE POSTS CH. 33+500 – CH.80+596 3/3	A1 / 88061
*	PAVEMENT MARKINGS	A1 / 87777
	SCHEDULE OF PAVEMENT MARKINGS CH. 33+500 – CH.80+596	A1 / 88062
*	ROAD SIGNS	A1 / 87779
*	ROAD SIGNS FOR BRIDGE APPROACHES AND INTERSECTIONS	A1 / 87780
	SCHEDULE OF ROAD SIGNS CH.33+500 – CH.80+596	A1 / 88063
DRAINAGE		
*	STANDARD CULVERT HEADWALLS	A1 / 87782
	CULVERT HEADWALLS IN ALIKA SWAMP	A1 / 88064
*	CULVERT BEDDING, SUBSOIL DRAIN AND STANDARD INLET PIT	A1 / 87783

*. . . . THE DRAWING FROM LOT - I

DRAWINGS LOT- II

TITLE OF DRAWING	DRAWING NO.
CULVERT SCHEDULE CH.33+530 – CH.41+015	A1 / 88065
CULVERT SCHEDULE CH.41+150 – CH.53+350	A1 / 88066
CULVERT SCHEDULE CH.53+845 – CH.60+450	A1 / 88067
CULVERT SCHEDULE CH.60+500 – CH.64+270	A1 / 88068
CULVERT SCHEDULE CH.65+260 – CH.79+140 AND ON SIDE DITCH	A1 / 88069
OTHERS	
EARTHWORKS SCHEDULE CH.33+55 – CH.80+596	A1 / 88070
SETTLEMENT PLATE AND DISPLACEMENT PEG	A1 / 88071
RENO MATTRESS AND GABION	A1 / 88072
BORROW PIT NO. 1	A1 / 88073
BORROW PIT NO. 2 - 1 & 2 - 2	A1 / 88074
BORROW PIT NO. 3 - 1 & 3 - 2	A1 / 88075
BORROW PIT NO. 4 AND SAND BORROW PIT NO. 2	A1 / 88076
BORROW PIT NO. 5, BASE BORROW PIT NO. 1, SUBBASE BORROW PIT NO. 3 AND SPOIL BANK NO. 4	A1 / 88077
SAND BORROW PIT NO. 1 & NO. 2 AND RIVER DEPOSIT NO.2	A1 / 88078
* SPOIL BANK NO. 1, 2 AND 3 FOR LOT - I / STOCK PILE NO. 1 FOR LOT - II	A1 / 87788
* PROJECT NOTICE BOARD	A1 / 87791
* ENGINEERS OFFICE ACCOMMODATION	A1 / 87792
* PLAN, ELEVATIONS SECTIONS AND DETAILS	A1 / 87793
* DOOR & WINDOW SCHEDULE STAIR DETAILS SECTION AND JOINERY ELEVATIONS	A1 / 87794
* PLANS, ELEVATIONS, ELECTRICAL LEGEND AND WIRING DIAGRAM	A1 / 87795
* SECTION DETAILS	A1 / 87796
PLAN AND LONGITUDINAL SECTIONS	
FROM CH. 33+500 – CH. 33+700	A1 / 88079
TO CH. 80+100 – CH. 80+596	– A1 / 88147

*. . . . THE DRAWING FROM LOT - I

DRAWINGS LOT-II

TITLE OF DRAWING	DRAWING NO.
CROSS SECTIONS	
FROM CH. 33+500–CH. 33+700	A1/88148
TO CH. 80+500–CH. 80+596	–A1/88248
BRIDGES	
BRIDGE NO. 4 - MIARU BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1/88249
GENERAL ARRANGEMENT	A1/88250
ABUTMENT PLANS, SECTIONS & DETAILS	A1/88251
PIER DETAILS	A1/88252
DECK SLAB DETAILS	A1/88253
STEEL WORK DETAIL SHEET 1	A1/88254
STEEL WORK DETAIL SHEET 2	A1/88255
HANDRAILING/IMPACT ANGLE DETAILS	A1/88256
BAR BENDING SCHEDULE SHEET 1	A1/88257
BAR BENDING SCHEDULE SHEET 2	A1/88258
BEARING BP.B - 103 (FIXED)	A1/88259
BEARING BP.B - 104 (MOVABLE)	A1/88260
RIVER BANK PROTECTION, BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88261
BRIDGE NO. 5 - KAPURI RIVER	
GENERAL NOTES AND DRAWING LIST	A1/88262
GENERAL ARRANGEMENT	A1/88263
ABUTMENT PLAN, SECTION & DETAILS	A1/88264
PIER DETAILS	A1/88265
DECK SLAB DETAILS	A1/88266
STEEL WORK DETAILS SHEET 1	A1/88267
STEEL WORK DETAILS SHEET 2	A1/88268
HANDRAILING/IMPACT ANGLE DETIALS	A1/88269
BAR BENDING SCHEDULE SHEET 1	A1/88270
BAR BENDING SCHEDULE SHEET 2	A1/88271
BEARING BP.B - 101 (FIXED)	A1/88272
BEARING BP.B - 102 (MOVABLE)	A1/88273
BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88274

DRAWINGS LOT-II

TITLE OF DRAWING	DRAWING NO.
BRIDGE NO. 6 - LAKEKAMU BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1/88275
GENERAL ARRANGEMENT	A1/88276
ABUTMENT PLAN, SECTION & DETAILS	A1/88277
PIER DETAILS	A1/88278
DECK STEELWORK GENERAL ARRANGEMENT	A1/88279
GIRDER DETAILS SHEET 1	A1/88280
GIRDER DETAILS SHEET 2	A1/88281
GIRDER LAUNCHING DETAILS	A1/88282
GIRDER ERECTION PROCEDURE	A1/88283
DECK SECTIONS	A1/88284
DECK CONSTRUCTION PROCEDURE	A1/88285
DECK SLAB DETAILS	A1/88286
HANDRAILING/IMPACT ANGLE DETAILS	A1/88287
BAR BENDING SCHEDULE SHEET 1	A1/88288
BAR BENDING SCHEDULE SHEET 2	A1/88289
BEARING BP.B - 104 (MOVABLE)	A1/88290
BEARING BP.B - 117 (FIXED)	A1/88291
BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88292
BRIDGE NO. 7 - TAURI BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1/88293
GENERAL ARRANGEMENT	A1/88294
ABUTMENT PLAN, REINFORCEMENT & CONCRETE DETAILS	A1/88295
PIER DETAILS	A1/88296
DECK STEELWORK GENERAL ARRANGEMENT	A1/88297
GIRDER DETAILS SHEET 1	A1/88298
GIRDER DETAILS SHEET 2	A1/88299
GIRDER LAUNCHING DETAILS	A1/88300
GIRDER ERECTION PROCEDURE	A1/88301
DECK SECTIONS	A1/88302
DECK CONSTRUCTION PROCEDURE	A1/88303
DECK SLAB DETAILS	A1/88304
HANDRAILING/IMPACT ANGLE DETAILS	A1/88305
BAR BENDING SCHEDULE SHEET 1	A1/88306
BAR BENDING SCHEDULE SHEET 2	A1/88307
BEARING BP.B - 104 (MOVABLE)	A1/88308
BEARING BP.B - 117 (FIXED)	A1/88309
BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88310

DRAWINGS LOT-II

TITLE OF DRAWING	DRAWING NO.
BRIDGE NO. 8 - MAKARA BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1/88311
GENERAL ARRANGEMENT	A1/88312
ABUTMENT PLAN, SECTIONS & DETAILS	A1/88313
PIER DETAILS	A1/88314
DECK SLAB DETAILS	A1/88315
STEEL WORK DETAIL SHEET 1	A1/88316
STEEL WORK DETAIL SHEET 2	A1/88317
HANDRAILING/IMPACT ANGLE DETAILS	A1/88318
BAR BENDING SCHEDULE SHEET 1	A1/88319
BAR BENDING SCHEDULE SHEET 2	A1/88320
BEARING BP.B - 101 (FIXED)	A1/88321
BEARING BP.B - 102 (MOVABLE)	A1/88322
BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88323
BRIDGE NO. 9 - SAPPAHARO BRIDGE	
GENERAL NOTES AND DRAWING LIST	A1/88324
GENERAL ARRANGEMENT	A1/88325
ABUTMENT PLAN & DETAILS (MALALAUA ABUTMENT)	A1/88326
ABUTMENT PLAN & DETAILS (BEREINA ABUTMENT)	A1/88327
PIER DETAILS	A1/88328
DECK SLAB DETAILS	A1/88329
STEEL WORK DETAIL SHEET 1	A1/88330
STEEL WORK DETAIL SHEET 2	A1/88331
HANDRAILING/IMPACT ANGLE DETAILS	A1/88332
BAR BENDING SCHEDULE SHEET 1	A1/88333
BAR BENDING SCHEDULE SHEET 2	A1/88334
BEARING BP.B - 101 (FIXED)	A1/88335
BEARING BP.B - 102 (MOVABLE)	A1/88336
BEARING UNITS, BACKFILL TO BRIDGE ABUTMENT AND OTHERS	A1/88337