3.8 Characteristics of Vehicles and Loading

The distribution of vehicles by age and manufacturer was obtained through interview which was conducted simultaneously with the Origin-Destination survey. The result of this interview is shown in Table V-3-15: average age of passenger cars is 6.41 years; that of van and pick-ups, 9.01 years; that of medium trucks, 11.02 years; that of large trucks, 9.51 years; that of truck-trailers, 12.63 years; and that of motorcycles, 4.08 years. Vehicles are generally quite old, and trucks are particularly old.

The same Table also shows the distribution of vehicles by manufacturer, showing incidence rates for the top three makes in each type of vehicle. It is observed that, while the top three makes represent more than 70% of all trucks, passenger cars and vans and pick-ups represent a high variety of makes and the top three makes of those types show low rates.

Table V-3-16 summarizes the distribution of vehicles by loading capacity. The average of van and pick-up trucks is 0.97 tons and the greatest number of these trucks fell in 1 to 2-ton class. The average loading capacity of medium trucks, which category covers from 3 to 8 tons, is 5.8 tons and the greatest number of them is of 5 to 8-ton class. The average of large trucks (8 to 11-ton) is 8.7 tons, and most of them from 8 to 9-tons, and that of truck-trailers (11-tons or larger) is 14.5 tons and most of them from 12 to 13-tons.

Table V-3-17 represents a summary of loading characteristics of passenger cars and trucks. Because some vans carry passengers and some vans carry cargo, they are broken into appropriate categories accordingly. The occupancy rate of taxi cabs is fairly high; average number of passengers per cab is 2.0 (average for occupied cabs being 2.6). Average number of passengers on vans is 2.8 (average for occupied vans being 3.9).

44% of cargo vehicles were loaded in average. Both the loaded vehicle rate and the cargo loading rate seem to be generally higher in the case of cargo vehicles with bigger loading capacity, indicating that those of higher capacity are utilized for higher efficiency. Average cargo loading rate of trucks and all other cargo vehicles is 0.337 (average for loaded vehicles being 0.703).

Table V-3-15 Distribution of Motor Vehicles by Maker and Age

Vehicle Make		Years in Service ~1	1	2	3	4	5	6	7	8	9	10	11~15	16~	Total Number	Total %	Average Years in Service
Car Taxi	Moris	84	434	288	285	216	176	160	227	134	55	188	169	136	2552	14.6	8.55
	Austin	100	536	430	326	257	133	87	113	47	59	49	84	60	2280	13.0	6.46
	Renault	163	631	599	341	168	88	33	51	29	16	26	21	47	2213	12.6	2.96
	Others	701	2371	2050	1474	893	747	432	429	350	145	261	314	307	10479	59.8	6.61
	Total	1048	3972	3367	2426	1539	1144	712	820	560	274	524	583	550	17524	100.0	6.41
Van, Pick-up	Bedford	9	24	61	65	44	20	11	38	17	14	12	22	29	366	16.2	11.02
	Commer	12	33	75	62	18	22	18	21	6	4	13	14	11	309	13.7	7.49
	Moris	2	17	32	28	33	23	26	38	22	7	11	25	24	288	12.7	11.62
	Others	42	224	226	224	159	77	65	62	50	27	48	25	70	1299	57.4	8.23
	Total	65	298	394	379	254	142	120	159	95	52	84	86	134	2262	100.0	9.01
Medium Truck	Bedford	18	64	106	93	118	89	61	69	30	27	30	70	51	826	54.7	9.89
	Moris	0	22	27	11	9	19	11	21	2	4	11	8	19	164	10.9	12.99
	Commer	4	4	10	19	14	22	7	16	4	0	5	9	18	132	8.7	14.13
	Others	12	34	40	45	56	32	15	40	17	13	24	24	35	387	25.7	11.56
	Total	34	124	183	168	197	162	94	146	53	44	70	111	123	1509	100.0	11.02
Heavy Truck	Bedford	6	13	15	21	19	10	12	17	1	0	8	2	5	129	42.0	7.78
	Leyland	3	3	1	9	11	7	9	9	2	1	4	5	4	68	22.2	9.41
	Commer	0	3	5	3	12	1	1	8	5	1	1	0	2	42	13.6	10.03
	Others	2	9	5	8	5	5	5	6	0	3	8	4	8	69	22.2	12.57
	Total	11	28	26	41	47	23	27	40	8	5	21	11	19	309	100.0	9.51
Truck-Trailer	Leyland	10	3	14	18	31	12	0	22	2	2	11	21	6	155	40.3	7.25
	Bedford	0	7	0	15	6	2	2	11	10	0	4	4	8	69	17.9	15.96
	Albion	0	6	0	0	0	1	4	24	4	0	0	0	15	54	14.0	31.45
	Others	3	8	6	10	31	6	10	3	7	4	10	6	3	107	27.8	8.78
	Total	13	24	20	43	71	21	16	60	23	6	25	31	32	385	100.0	12.63
Motorcycle	Honda	80	257	213	147	90	50	56	14	18	17	20	7	16	985	37.4	4.36
	Yamaha	58	142	102	73	10	7	5	4	3	4	0	0	4	412	15.6	2.77
	Vespa	16	74	60	62	27	35	20	19	15	6	12	22	7	375	14.2	5.32
	Others	128	212	231	121	61	22	34	16	5	9	7	3	16	865	32.8	3.85
	Total	282	685	606	403	188	114	115	53	41	36	39	32	43	2637	100.0	4.08
Total	No.	1453	5131	4596	3460	2296	1606	1084	1278	780	417	763	859	901	24624		
	%	5.9	20.8	18.7	14.0	9.3	6.5	4.4	5.2	3.2	1.7	3.1	3.5	3.7	100.0		

Table V-3-16 Distribution of Vehicles by Loading Capacity

	Van Pick-up	Medium Truck	Heavy Truck	Truck-Trailer	Total
0 1	951	0	0	0	951
1 2	1,075	0	0	0	1,075
2 3	110	0	0	0	110
3 4	22	105	0	0	127
4 5	0	71	0	0	71
5 6	0	388	0	0	388
6 7	0	303	0	0	303
7 8	0	404	0	0	404
8 9	0	0	142	0	142
9 10	0	0	66	0	66
10 11	0	0	37	0	37
11 12	0	0	0	16	16
12 13	0	0	0	102	102
13 14	0	0	0	17	17
14 15	0	0	0	48	48
15 16	0	0	0	20	20
16 17	0	0	0	49	49
17 18	0	0	0	12	12
18 19	0	0	0	5	5
19 20	0	0	0	9	9
$\geq$ 20	0	0	0	20	20
Total	2,158	1,271	245	298	3,972
Average Capacity (ton)	0,967	5,758	8,706	14,466	3,990

Note: Vehicles of which load contents and/or loading capacities were not clear in the field survey are excluded.

Table V-3-17 Loading Characteristics of Vehicles

Passenger Vehicles	Car	Taxi	Van	Motor-cycle
. Percentage of occupied vehicle	-	76.5	73.7	-
. Average number of passengers per vehicle1/	1.4	2.0	2.8	0.3

1/ excluding driver

Freight vehicles	Van, Pick-up	Medium Truck	Heavy Truck	Truck-Trailer	Total
. Percentage of loaded vehicles	41.2	45.4	45.7	56.7	44.0
. Average payload (ton)	0.55	3.79	5.91	11.81	3.06
. Average loading rate of loaded vehicle	0.55	0.66	0.67	0.84	0.70

CHAPTER VI COMPARISON AND SELECTION OF ALTERNATIVES

CHAPTER VI COMPARISON AND SELECTION OF ALTERNATIVES

1. Comparison of Alternatives

In view of the specific requirements of the Proposed Road, due consideration should be given to the following aspects in alternative planning:

1) Route location;
2) Bridges (construction site and structual types), and
3) Construction stages.

Full attention was paid on determination of the route location as the most important factor and subsequently other items were also examined as stated below.

1.1 Route Location

Preliminary studies were conducted on several alternatives prior to comparative studies.

Routes were basically aligned, first, on the plan maps (1:2,500 scale) and then field surveys followed to their locations and site conditions so as to find out the recommendable route.

As shown in Figure VI-1-1, comparative studies of alternatives were made separately along the two sections as follows:

The first section:	a route from Cassis Flyover Bridge in Port Louis to the S. Hill Intersection
The second section:	a route from the S. Hill Intersection to the Roundabout in Beau Bassin

1.1.1 Alternatives for the First Section

(a) Utilization of the present A_1 Road

(b) Utilization of the disused railway track originating from the Motorway

As a result of the preliminary study, it proved difficult to expand the width of the A1 Road in this section as detailed in the following paragraphs. Obviously, the A1 Road is unfit for the proposed road which is designed for a high standard four-lane highway. For this reason, there is no other course but to take up the second proposal as the alternative.

Regarding the A1 Road proposal, we made preliminary design and economic analysis as reference for future replacement of the G.R.N.W. Bridge which has been superannuated. Results are given in the appendixes appended at the end of the report.

The following paragraph explains the existing conditions of the first proposal and reasons for abandoning it as an alternative.

(1) Present Situation of the First Proposal and Reasons for Abandoning It as an Alternative

Present Situation

At present, the A1 Road is the only one that links the regions of Coromandel and Beau Bassin directly with Port Louis.

The A1 Road goes through urbanized areas with many houses built on each side of the road especially between the G.R.N.W. Bridge and the Flyover Bridge. In this section, the road has about three lanes but further width expansion is impracticable as it is so with a section between the G.R.N.W. Bridge and S. Hill due to the presence of sharp curves and steep slopes. As referred to in the following paragraphs, the G.R.N.W. Bridge located in the center of this section has a narrow width of 5.7 m which constitutes the most serious bottleneck in the way of traffic flow.

The G.R.N.W. Bridge has been utilized by traffic for over 50 years since it was built in 1923. The bridge is becoming increasingly superannuated and it is considered difficult for the bridge to meet increased traffic volume and load over the coming several decades.

Vehicles using the bridge must have been smaller and lighter at the time when the bridge was built, but at present, many problems have been encountered, such as insufficient width (W = 5.7 m) and excessive load applied on the old bridge, etc.

Reasons for Disqualifying the Proposal as an Alternative

As mentioned, the area along A_1 Road is packed with houses and buildings, further expansion of width beyond present three lanes is technically difficult.

In the neighborhood of S.Hill, some part of the road is graded more than 6% and major improvements of this grade is impossible due to an appreciable difference in elevation between the present bridge and S. Hill.

On the other hand, the proposed road is designed for a high standard four-lane highway and, therefore, the improvement of A1 Road alone will not be enough to meet these requirements.

For these reasons, a proposal for improving A1 Road in order to be qualified as the alternative route is unworkable.

(2) Proposal for Utilization of the Disused Railway Track

The disused A_1 Road railway track is linked with A_1 Road at the S. Hill Intersection and with the Motorway in Pailles.

As a result of technical studies, it proved that the disused railway has an appropriate alignment for the proposed road and, for this reason, we look upon the utilization of the disused railway track as the most realistic alternative route. However, the present railway bridge only allows two-lane traffic and we are therefore planning to build a new bridge in parallel with the existing one on its upstream in order to accommodate additional two lane traffic.

1.1.2 Alternatives for the Second Section

As the alternatives for the second section, we arrived at two proposals; 1) improvement of A_1 Road and 2) utilization of the disused railway track. In this connection, it is impractical to take up the first proposal due to a number of adverse factors that 1) districts along Al Road have been urbanized and packed with houses and buildings as evidenced by field surveys and aerial photos and 2) further expansion of road width is nearly impossible due to the difficulty of acquiring sites and huge financial expenditures involved.

As for the utilization of the disused railway track, four alternative routes were selected, as indicated in Figure VI-1-1, after preliminary studies.

(c) Route 1, to make almost full use of the disused railway track;
(d) Route 2, to be aligned with that proposed by the regional plan by M.O.H.L.T.C.P.;
(e) Route 3, to utilize the right of way of Chèbel Branch Road, and
(f) Route 4, which runs closer to Al Road.

Through the preliminary analysis and technical discussion with the M.O.W.'s engineering staff, the alternatives shown in Table VI-2-1 have been selected for detailed analysis. During such procedures, the plan (a) for the first section was dropped due to the reasons already mentioned and the plan to replace the present G.R.N.W. Bridge and construction of approach roads to and from the new bridge was adopted. Plan (c) for the second section was deleted because of too lengthy access to Beau Bassin and plan (e) for the same section was also dropped because it would increase the number of the crossings.

Figure VI-1-1 Location of Alternative Route

Scale - 1: 25 000

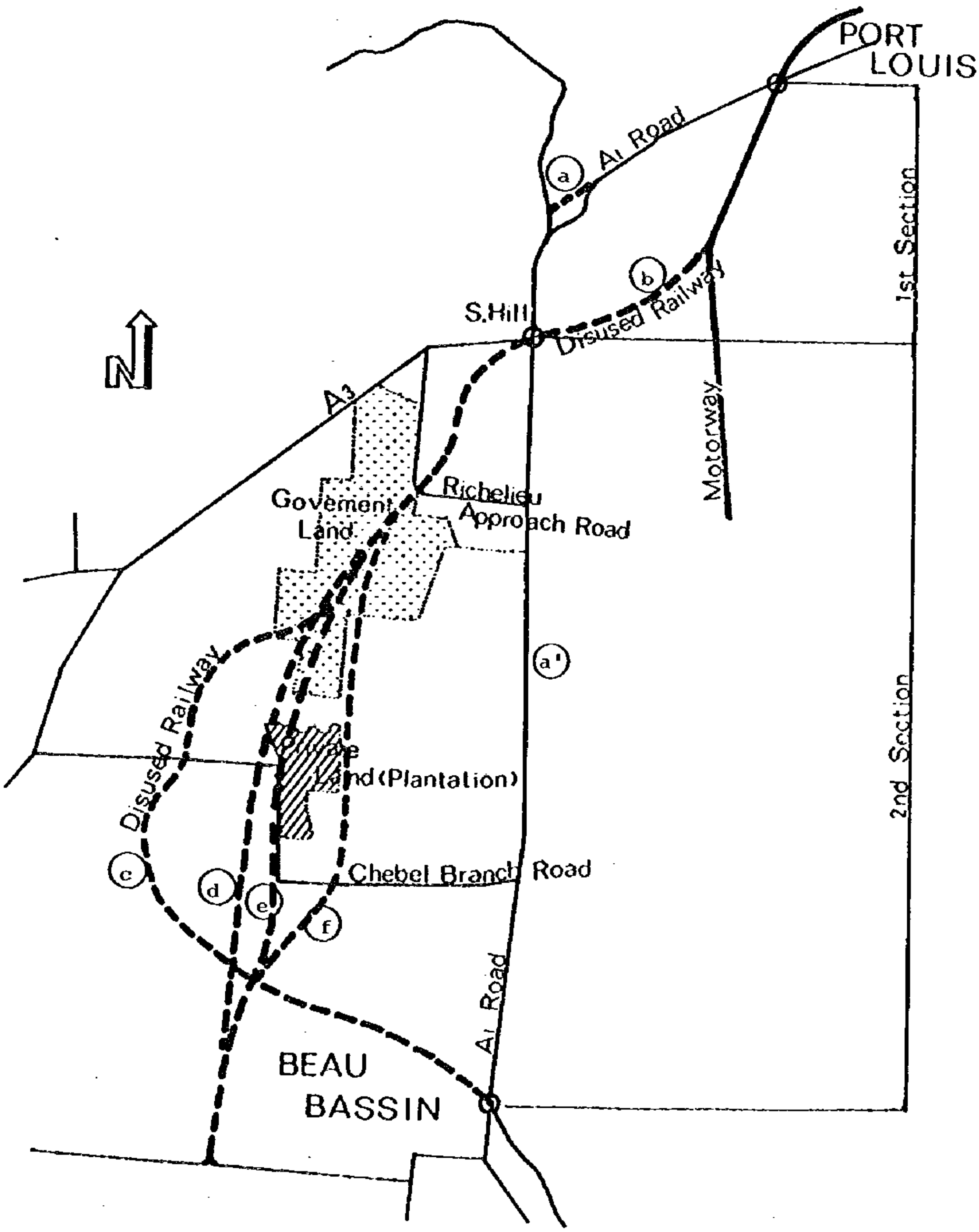


Table VI-1-1 Characteristics of Alternative Routes for Beau Bassin/S. Hill by Preliminary Study

Alternatives	c	d	e	f
Road Length: Main route Access	 5.3 km 1.8 km	 4.6 km 1.8 km	 4.7 km 1.6 km	 4.7 km 1.6 km
Vertical Alignment	. gradient: 2-4%	. max. gradient: 5.2% . length exceeding 5%: 750 m	. max gradient: 5.7% . length exceeding 5%: 1,100 m	. max. gradient: 6.5% . length exceeding 5%: 1,300 m
Horizontal Alignment	. relatively smaller radius curvature are often seen	good	good	fair
Adjustment with surrounding land use	negligible	negligible	subdivide a existing plantation	Route pass the Barkly C.H.A. Estate nearby
Number of Intersections with relevant road	. Richelieu Approach Road . Chebel Branch Road . Access Road to Beau Bassin	same as of c	Two junctions with Chebel Branch Road	same as of c

1.1.3 Summary of Alternative Route Locations

From the previous results of study, we take up the alternative routes as follows from a realistic point of view.

Link (A) : Substitution (on new installation) of G.R.N.W. Bridge on A_1 Road.

Link (B) : Utilizes G.R.N.W. Bridge as well as the disused railway track from M_1 to S. Hill.

Link (C) : Utilizes the disused railway track from S. Hill to the southern side of Richelieu Approach Road Intersection, then takes a short cut across the railway and re-crosses it in the northwestern part of Beau Bassin.

Link (D) : Like Link (C), it takes a short cut accross the disused railway, but is laid closer to A_1 Road by about 550 m than Link (C). It passes the western skirts of Petit Malabar and crosses the disused railway at a point about 100 m closer to Beau Bassin than Link (C).

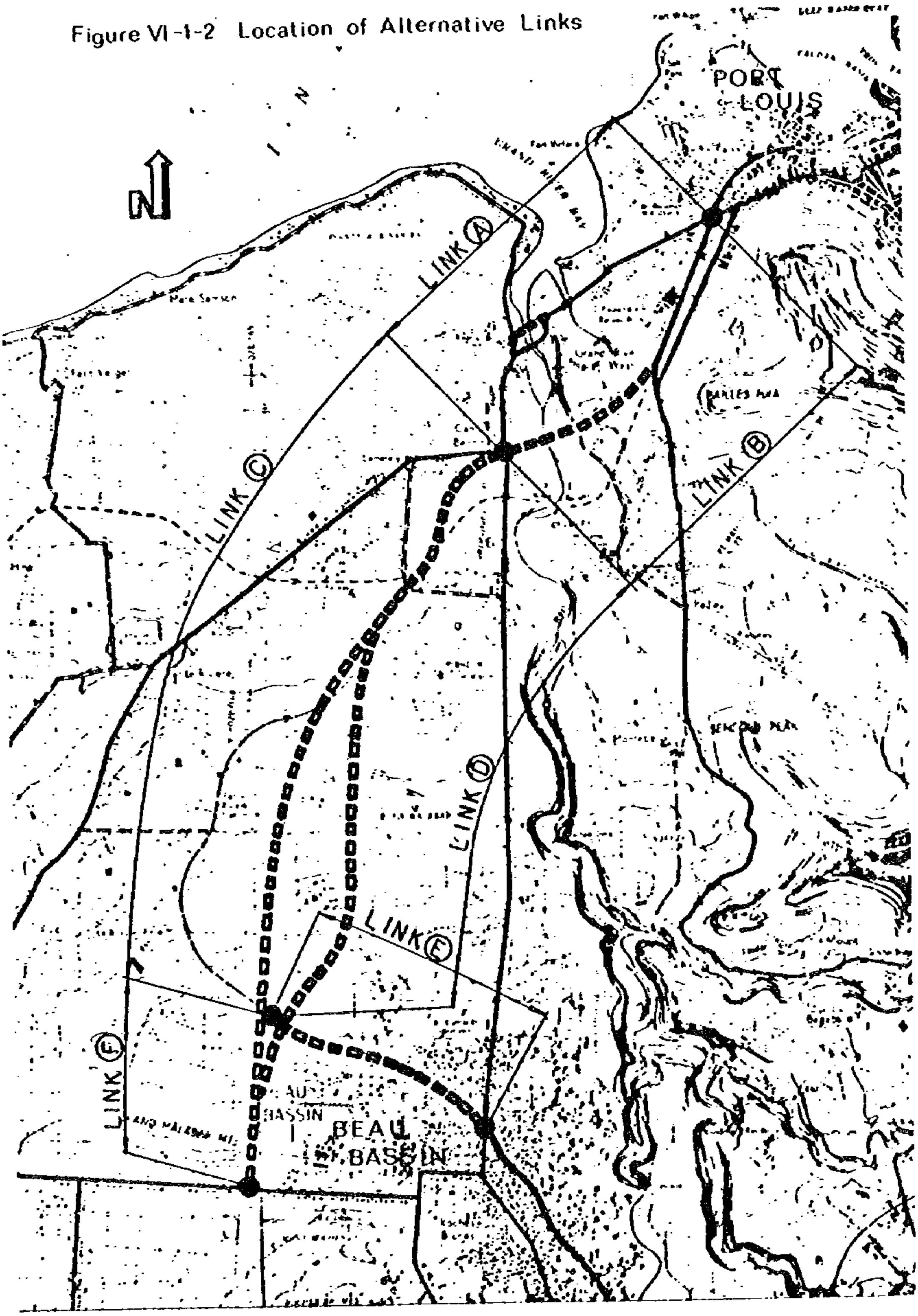
Link (E) : Links Roundabout with A_1 Road in the central part of Beau Bassin, utilizing the disused railway track.

Link (F) : Extends from the intersection of the disused railway track further southward down to Lower Plains Wilhems Cemetery Road.

As mentioned, we do not include Link (A) in the alternatives, but we treat it in the same way as for the alternatives for the purpose of conducting studies, for the reference's sake, on the replacement of bridges and assessment of potential impact on overall road network.

Results of studies on Link (A) are given in Appendix A appended in the last pages of this report.

Figure VI-1-2 Location of Alternative Links



1.2 Bridges (Construction Sites and Type of Structures)

Based on the location of alternative routes referred to in paragraph 1.1, studies have been conducted to identify appropriate construction sites, structural types and kind of bridges.

See respective paragraphs for details. For example, Chapter IX, paragraph 1. (Preliminary Road Design) deals with bridge construction sites and Chapter IX, paragraph 2. (Preliminary Bridge Design) deals with structural types and kind of bridges.

1.3 Construction Stages

Like the alternative routes, the construction stages applicable to the routes concerned are also subjected to comparative review, specifically, on the potential effect produced by the construction of the routes in a way to cope with the growth of traffic demand.

Studies are conducted on such alternatives as combining either a two-lane road (interim) or a four-lane road (final) with links in different ways. Chapter VIII (Traffic Forecast) and Chapter XI (Evaluation) deal respectively with these subjects in details.

2. Selection of Alternatives

In accordance with the concept of the alternatives with regard to various items which have been discussed, basic alternatives deemed appropriate are tabulated below.

The following combinations of the alternatives are made up of practically constructible routes which can be efficiently incorporated in the overall traffic network.

In the process of concrete studies, these basic alternatives will produce more than ten variations by being incorporated with their construction schedule.

Table VI-2-1 Basic Alternatives for Final Study

Alternatives	Notes	Links
P_2	Belle Village - S. Hill - Beau Bassin 2-lane road	Link B+C+E
P_2'		Link B+D+E
P_4	Belle Village - S. Hill - Beau Bassin 4-lane road	Link B+C+E+F
P_4'		Link B+D+E+F
P_A	G.R.N.W. Bridge (A_1 Road) Replacement	Link A

Note: P_2' and P_4' denotes western route plans, while P_2 and P_4 eastern route plans

Replacement of G.R.N.W. Bridge (A_1 Road) is shown in Appendix.

CHAPTER VII ENGINEERING ANALYSIS

CHAPTER VII ENGINEERING ANALYSIS

1. Outline

This Chapter describes the details of the survey executed by the Team and the analytical studies of the data obtained. The field surveys were accomplished by using the topographical maps (1:2,500 scale) and the aerial photos.

In leveling survey, the Team adopted the Bench Marks as basical elevation settled by Mauritius Government and covered the following essential spots:

- The proposed bridge sites at Grand River North West and St. Louis River,
- The places along the existing Motorway and A_1 Road, where the proposed road way join with and/or intersect.

The existing irrigation canals and the new roads under the city planning along the proposed road have also been checked. Bench Mark informations were given by the Ministry of Housing, Lands and Town and Country Planning.

2. Topography

The area through which the proposed road would run extends from Port Louis, the capital of Mauritius, to Beau Bassin and is comprised of the lava plateau, up-grading 3 to 4% from Port Louis to Curepipe, the central part of the Island, via Beau Bassin.

In this area, the Grand River North West, the longest river in Maunitius, and the St. Louis River are running in parallel. The starting point of the proposed road was decided at 2.4 km from the former river. Excepting these two rivers, the road is aligned through the sugarcane fields varying their altitudes gradually from 20 m to 210 m.

3. Geology and Soil

The following surveys and investigations were conducted for the purpose of determining the geology, soil and their distributions in the Project Area.

a. Surface geological survey
b. Subgrade soil survey (test pit, sampling and soil test)
c. Foundation ground survey at bridge construction sites (mechanical boring, standard penetration test, test pit, sampling and soil test)

Details of these surveys and test results are specified in Appendix VII-1, which are summarized as follows.

3.1 General Geology

In the surveyed area, bedrocks are mostly made up of older lavas of olivine basalt, which erupted in the early Tertiary era, (or older), underlying the layer of younger lavas consisting mostly of olivine basalt and dolerite which erupted in the Tertiary Pliocene or in the Quaternary Diluvium Era, with the layer of younger lavas covered with a thin layer of soil produced by weathering and degeneration of the younger lavas. Deposits are distributed over the river terrace and sediments on the riverbed.

3.2 Soil along the Proposed Road

Along the proposed road, older lavas bedrock underlies the layer of younger lavas covered on its top with a layer of weathered and degenerated soil of the younger lavas.

Of these, the older lavas are very hard, but plenty of cracks have been formed on the younger lavas which have been softened to a large degree. As the younger lava layer has a thickness of 5 to 10 m, earth-work will be directed at it as well as its surface soil, using a ripping method as a major means for excavation and in part blasting.

The top soil extending with 1 to 2 m thickness is classifiable into three categories according to color and particle size, or two categories from the viewpoint of soil engineering.

One of them is a quality soil with which a design CBR (California Bearing Ratio) of 9% is expectable, whereas the other is inappropriate as roadbed soil due to its design CBR of 1% and a swelling rate of 7 to 8%. The distribution of poor soil is limited only to a part and, as it is likely that weathered rocks are available in large quantities during construction, it is concluded that there is no particular soil problem provided these weathered rocks are effectively utilized.

3.3 Foundation Rock at Bridge Construction Sites

3.3.1 Foundation rocks at the disused railway bridges

The foundation rock is of older lavas (olivine basalt) underlying younger lavas (pillow lava). The riverbed lacks the younger lavas where deposits are placed directly on the layer of the older lavas. Furthermore, river terrace deposits are distributed on the banks of the river.

The older lavas are very hard with a N value of more than 50 and should have no problem as the foundation of structures. The younger lavas have been considerably weathered and softened, with a presumed N value of 30 to 50.

River deposits are mostly of hard and round gravels mixed with sand, silt and clay. This deposit layer is considerably compact, presumably containing more than 90% of gravels and a N value of more than 50. The thickness of this deposit layer is estimated at about 4 to 5 m. It was confirmed that river terrace deposits have a layer thickness of 4.3 m, in which gravels are dense in the lower parts and the N value exceeds 30. The lower part is also composed of sand, silt and clay, with the N value reaching about 10.

3.3.2 Foundation rocks at proposed sites for road bridges (downstream part of G.R.N.W. Bridge)

The foundation rock is of older lavas (olivine basalt) underlying the layer of younger lavas. Also, deposits are distributed on the beds of the Grand River North West and the St. Louis River.

The older lavas are very hard and show the N value of more than 50 and there should be no problem as to the foundation rock of structures.

River deposits have been distributed on the present riverbeds and lowlands (delta) between the Grand River North West and the St. Louis River. On the present riverbeds, deposits consist mostly of gravels and in part of sand and are soft and loose, with the N value showing 20 and a layer thickness of about 5 m. On the other hand, major elements of the deposits are sand and silty clay, with the N value of 4 to 12 and a layer thickness of 2.60 m, in the low lands between the Grand River North West and the St. Louis River. This deposit layer is considered to have insufficient strength to support important structures.

4. Distribution and Quality of Aggregates

Almost every where in the Island, there is a dense distribution of olivine basalt brocks called "stones" and as it is easy to secure such material, we assume that aggregates can be supplied in sufficient quantities. At present, large blocks of olivine basalt called stones are crushed into coarse and fine aggregates.

These olivine basalt brocks have considerably large values in density, specific gravity and compression strength and are appropriate as aggregates with extremely small effective porosity, low water content, and water absorption rate. Therefore, it follows that there is no particular problem in its quality as aggregate.

Relevant details are given in Appendix VII-1 with regard to their tests and investigations.

5. Hydrometeorology

5.1 Meteorology

This paragraph deals with general weather conditions prevailing in Mauritius.

5.1.1 Outline

Tropical and oceanic climate typical of the Southwest Indian Ocean prevails in Mauritius. As Mauritius is located about 900 km off Madagascar and 1,800 km off the African East Coast towards East, it is not affected by the continental winds.

Dense atmospheric moisture of the Indian Ocean is carried over to Mauritius by South-East Trade Winds, causing intensive rainfall in the south-eastern and central regions of the Island but less precipitation in the western and northwestern areas.

The southwestern part of the Indian Ocean is known as the area where cyclones occur. They mostly approach the Island from the southeastern direction and cause heavy downpour combined with hurricane when they directly land or approach the Island.

5.1.2 Temperature

The atmospheric temperature varies little throughout the year, with average temperatures showing 28°C (February) and 22°C (August) in Port Louis, and 22°C (February) and 17°C (August) in Curepipe respectively.

Maximum and minimum temperatures during the past decade (1961-70) were 35.6°C (March) and 12.2°C (July) in Port Louis, or 29.1°C (March) and 7.8°C in Curepipe.

5.1.3 Rainfall

Unbroken rainfall statistics have been kept in Line Barracks since 1853 and the characteristics of rainfall in Mauritius have been clarified for the past century.

The climate in Mauritius is classified into the rainy season (December through May) and the dry season (June through November), in which, the rainy season accounts for 70% of annual total rainfall. Even during the dry season, monthly rainfall averages as high as 120 mm or more in Curepipe in the central heights of Mauritius.

Annual rainfall, maximum monthly rainfall and minimum monthly rainfall during the past 30 years (1931-60) were 1,095 mm,179mm (January) and 23 mm (August) in Port Louis and 3,342 mm, 457 mm (January) and 126 mm (October) in Curepipe, respectively

5.1.4 Wind

The maximum instantaneous wind velocity recorded was 162mph (72.4m/sec) caused by cyclone "Jenny" which occurred on February 28, 1962.

5.1.5 Cyclone

Cyclones occur in the southern Indian Ocean during November through May; they initially proceed southwestward and then turn southeastward in the vicinity of Mauritius as a general pattern.

In case a cyclone hits Mauritius directly or passes nearby, it causes heavy downpours in the Island. When the cyclone "Daniel" hit the Island during January 17 through 19, 1964, 779.9 mm rainfall was recorded.

5.2 Hydrology

This paragraph deals with hydrological survey and analysis required for designning bridges, and road drainages to be installed along the projected road.

Eventual hydrological data required are the high water levels of rivers and of rainfall intensity.

5.2.1 River conditions

The Island as a whole is in the adolescent stage of topographical formation and rivers are shaped in a sharp V in spots where they rise and their protruded parts are linked together in the upper and middle streams.

During the dry season, there is little water to an extent that river-beds become dry but in the rainy season or when hit by a cyclone, there is an abundant flow of water in the rivers.

In most of the rivers, the formation of the delta has been prevented due to the high tide level and stream, but in the case of the Grand River North West, a delta has been formed in its estuary.

As the main watershed or ridge of mountains is located in the western part of the Island, rivers flowing westward are shorter than those running toward east.

5.2.2 Rainfall intensity

(1) Probable daily rainfall is assumed from the cyclone rainfall data (Table VII-5-1); rainfall intensity given during consecutive hours has been obtained as shown in Table VII-5-2 and Figure VII-5-1 (graphic representation);

(2) Rainfall intensity estimated from the maximum daily rainfall during the past 85 years and rainfall records (Table VII-5-3) during the past 85 years and 20 years were plotted and combined with entries on Figure VII-5-1 to develop Figure VII-5-2;

(3) The maximum daily rainfall (490mm) during the past 85 years is nearly equal to the daily rainfall(467.3mm) for the probability of 100 years; and

(4) From the above, we apply the estimated rainfall intensity for the probability of 100 years for river planning and the maximum rainfall intensity during 20 years by extrapolation for road drainage planning.

Table VII-5-1 Cyclone Rainfall

Year	Period	Cyclone	Station	Total Rainfall	Rainfall per Day
1959	Nov. 14	-	Pamplemousses	254.0 mm	254.0 mm
1960	Feb. 27 ~ 28	Carol	Vacoas	453.4	226.7
1961	Dec. 23 ~ 25	Beryl	"	716.1	238.7
1964	Jan. 17~19	Daniel	"	779.9	260.0
1967	Jan. 13~14	Gilberte	"	420.6	210.3
1968	Feb. 15	Ida	"	104.1	104.1
1970	Mar. 27~29	Louise	"	217.4	72.5
1971	Feb. 7~8	Helga	"	314.3	157.2
1972	Feb. 7	Dolly	"	146.3	146.3
1975	Feb. 5~6	Geruaise	"	408.6	204.3

Table VII-5-2 Rainfall Intensity

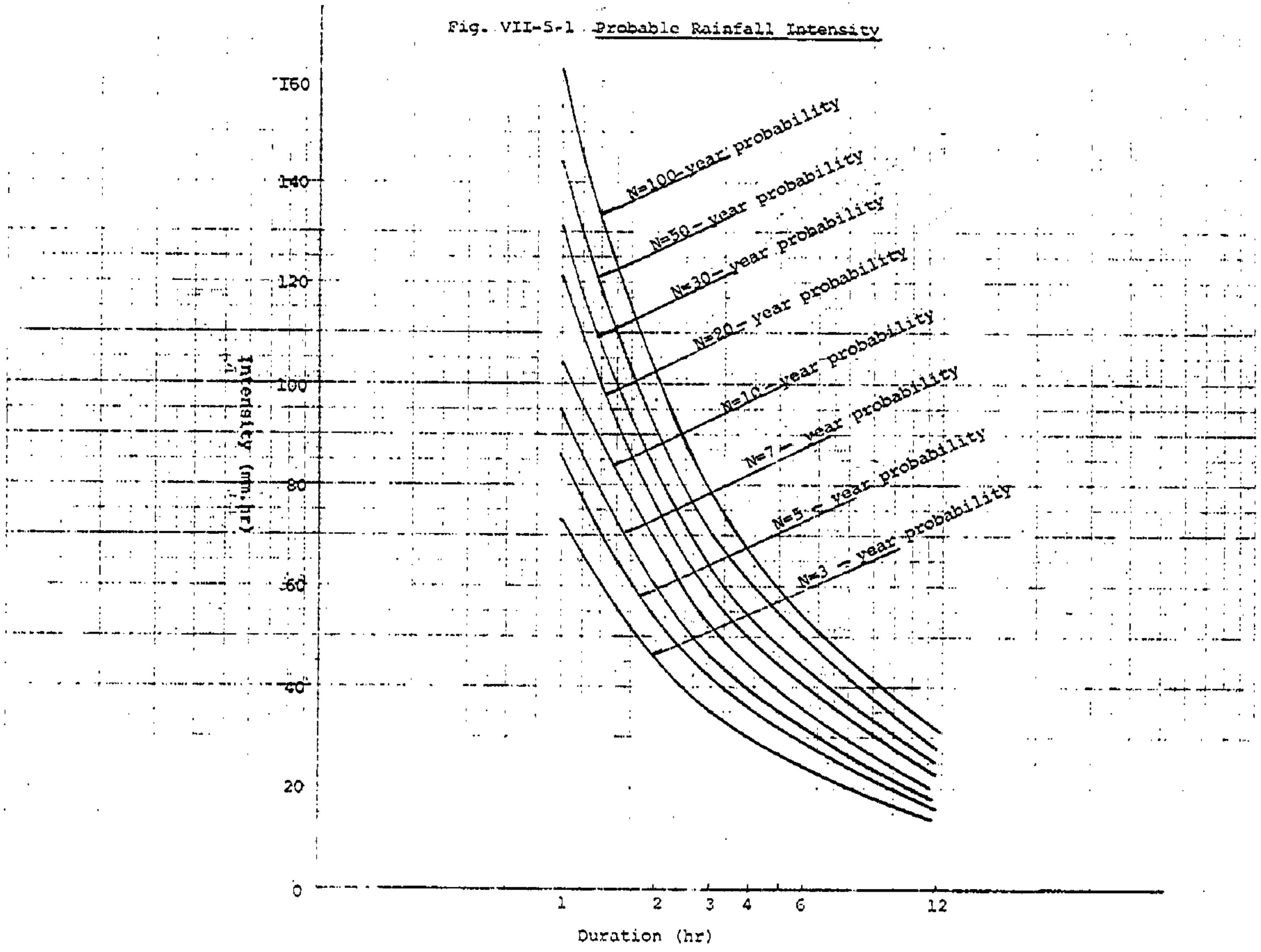
Unit : mm

Probable Year	100	75	50	30	20	10	7	5	3	2
24-Hour Rainfall / Duration	467.3	446.3	416.3	379.3	350.1	300.3	274.1	249.2	209.5	174.5
1 Hour	162	155	144	131	121	104	95	86	73	60
2 "	102	97	91	83	76	66	60	54	46	38
3 "	78	74	69	63	58	50	46	42	35	29
4 "	64	61	57	52	48	41	38	34	29	24
12 "	31	30	28	25	23	20	18	16	14	12
24 "	19	19	17	16	15	13	11	10	9	7

Table VII-5-3 Some Maximum Rainfall

Period	85 year		20 year			
Station	Pamplemousses Vacoas		Pamplemousses		Plaisance	
Duration	R (mm)	I (mm/h)	R (mm)	I (mm/h)	R (mm)	I (mm/h)
15 minutes	-	-	25	100	25	100
30 minutes	-	-	46	92	45	90
35 minutes	61	105	-	-	-	-
1 hour	82	82	76	76	79	79
2 hours	110	55	-	-	-	-
3 hours	150	50	-	-	-	-
4 hours	180	45	-	-	-	-
12 hours	250	21	-	-	-	-
24 hours	490	20	-	-	-	-

Fig. VII-5-1 Probable Rainfall Intensity



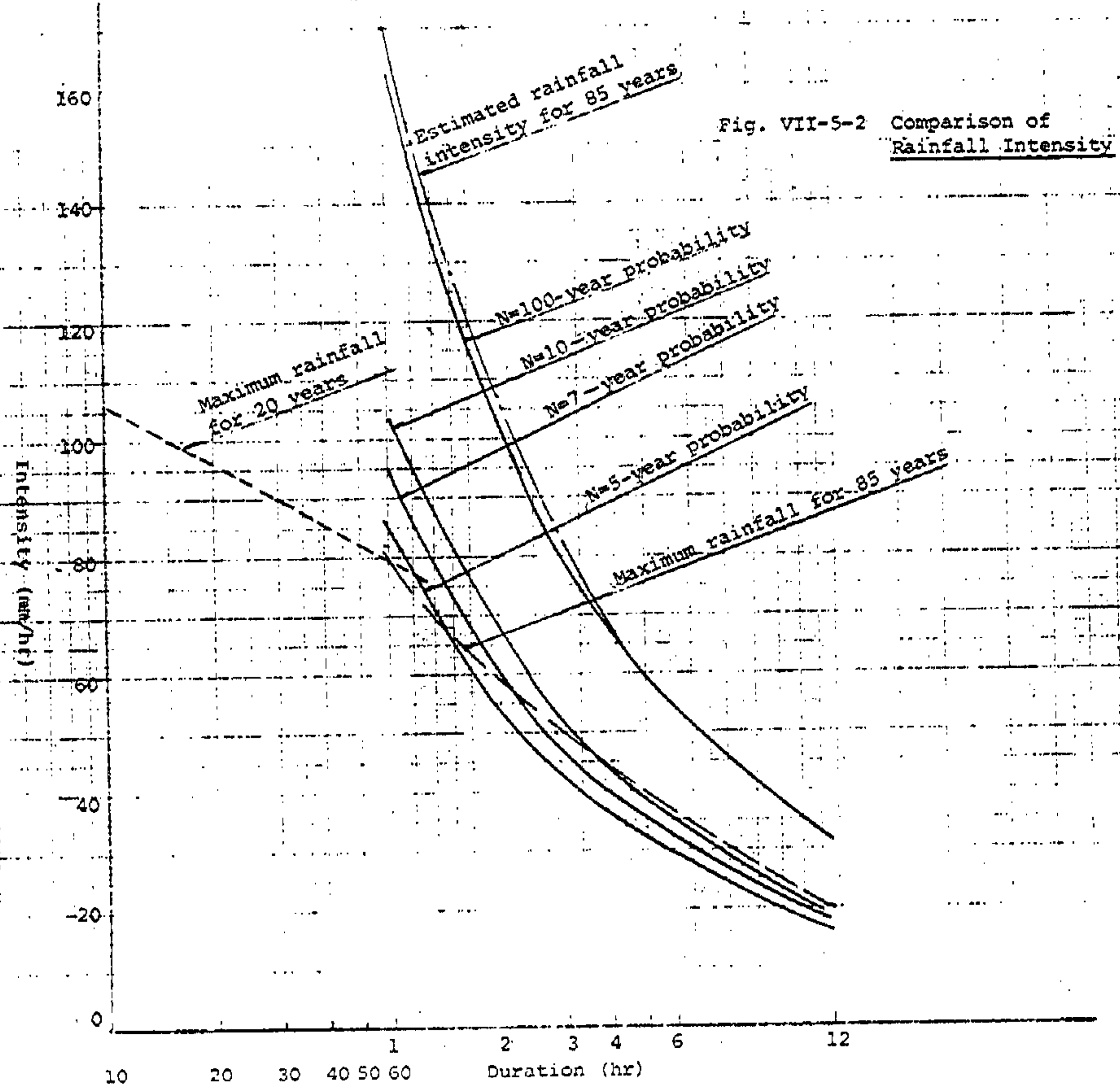


Fig. VII-5-2 Comparison of Rainfall Intensity

5.2.3 Design high water discharge

Concerning the Grand River North West and the St.Louis River, design high water discharge is determined on the basis of rainfall intensity for the probability of 100 years.

(1) A rational equation is used to determine the design high water discharge as follows:

$$Q = 1/3.6.f.r.A$$

where, Q: Discharge at out-let

f: run-off coefficient

r; Rainfall intensity (mm/hr)

A: Catchment area (Km^2)

(2) Run-off coefficient

The run-off coefficient is set at f=o.2 on the assumption that the projected area is a field type with water-penetrating soil and planted with few trees.

(3) Catchment area

The catchment areas are as follows:

G.R.N.W. A_1 = 116.8 km^2

St.Louis River A_2 =13.7 Km^2

(4) Time of inlet

The time of inlet is estimated at 20 minutes by taking into account potential utilization of the land in the future.

(5) Flowing velocity

The flowing velocity is estimated at 3.5 m/sec in consideration of the specific topography and grade (over 1/1,000).

(6) Flowing time

The flowing time is set at 101 minutes for G.R.N.W. and 59 minutes for the St. Louis River because total flow lengths are L_1 = 17,000 m (G.R.N.W.) and L_2 = 8,200 m (St Louis River).

(7) Calculation of high water discharge

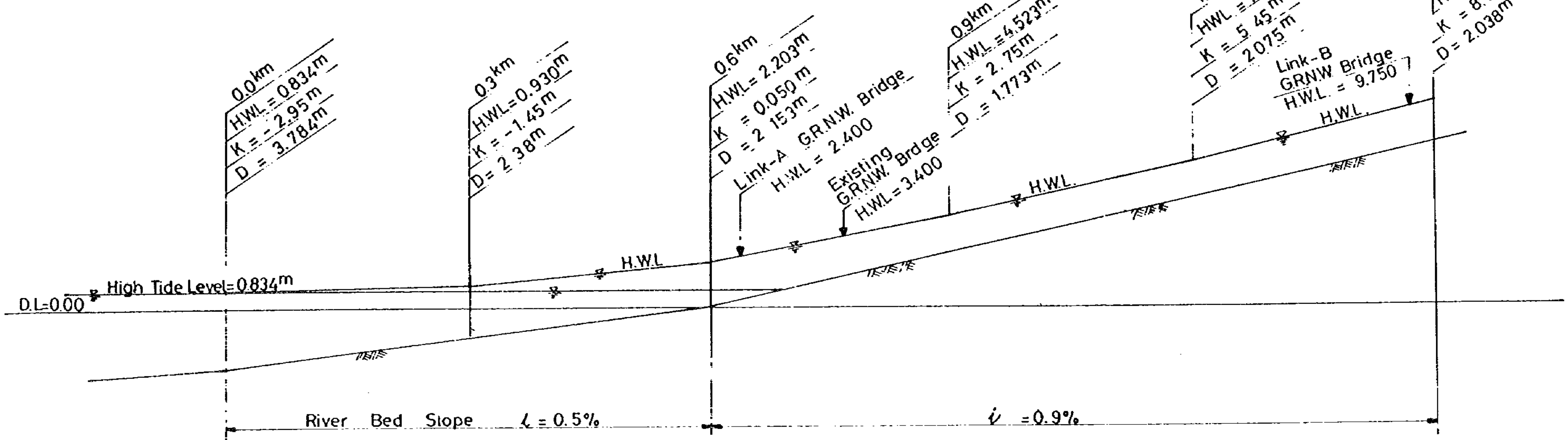
The high water discharge can be determined as follows on the basis of estimated rainfall intensity for probability of 100 years

G. R .N .W................Q'ty = 740 m^3/sec
St. Louis River...........Q'ty = 130 m^3/sec

5.2.4 Designed high water level (G.R.N.W)

Designed high water level can be determined from the design high water discharge by a non-uniform flow calculation method. (See Fig. VII-5-3)

Figure VII-5-3
High Water Level Of G.R.N.W.

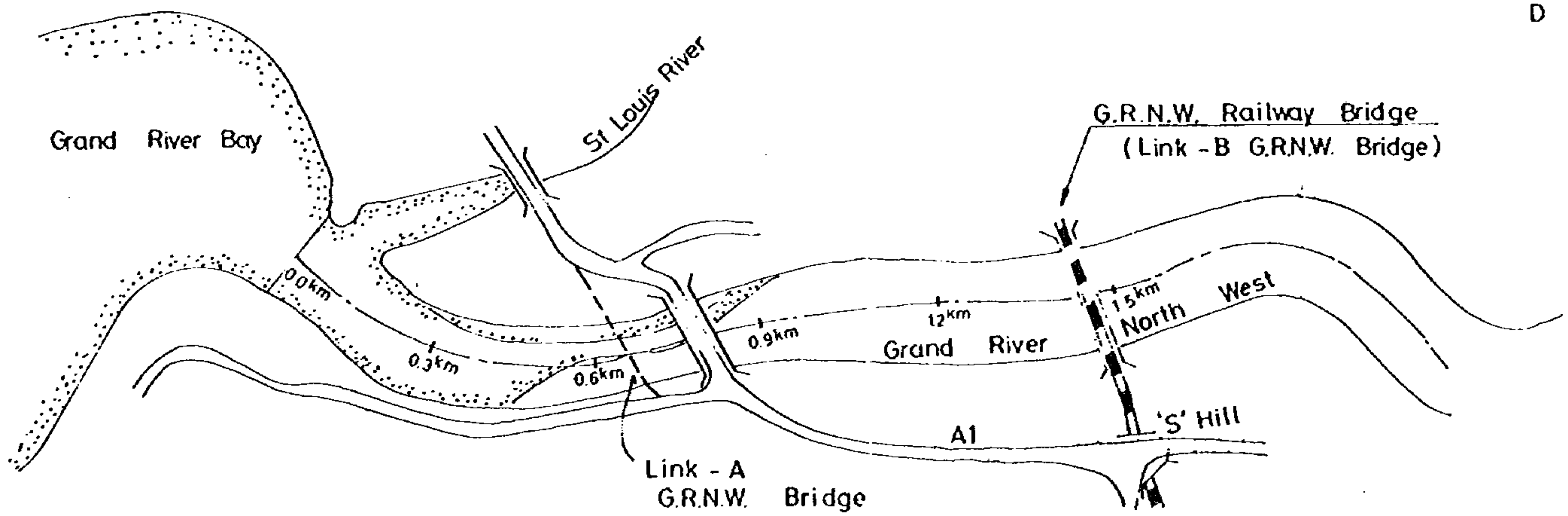


Note :

H.W.L = Elevation Of High Water Level

K = Elevation Of River Bed

D = Depth Of Water (H.W.L. - K)



6. Structures

This paragraph deals with the results of investigation concerning the existing conditions of structures such as the G.R.N.W. railway bridge, the St. Louis railway bridge on the disused railway track and other structures.

6.1 The G.R.N.W. Railway Bridge

It is a simply-supported, 7 through span bridge erected from 1911 through 1913 as a single track bridge.

The superstructure is the type of the two-girder bridge. Substructure is made of plain cobble-stone concrete with protective stones on its surface. The foundation is the type of spread footing.

(1) Superstructure

a. Measurement of dimensions
Measuring tools used are steel tape, a convex, a vernier gauge and a supersonic steel thickness measure meter. Typical dimension of superstructure are in Drg. 34/42.

b. Corrosion inspection

In corrosion checks, loose rust was removed with wire brush and hammer before inspecting the degree of corrosion.
Criteria A through D were established for classification of individual damages according to the status of conservation and damages.

A: Surface is preserved in a satisfactory condition with very little sectional damage if any.

B: Rusted, but plate surface is damaged 1 mm deep or less or corrosion developed in the direction of stress.

C: Plate surface is damaged more than 2 mm deep in a right angle with the direction of stress resulting in considerable sectional damage.

D: Badly damaged and corrosion has eaten through the steel plate or rivets are missing.

In corrosion checks performed according to these criteria, primary emphasis was placed on the girder flanges for each span. Table VII-6-1 shows relative results.

Ways and means for reutilizing the superstructure of this bridge are given in paragraph IX 3.2. In this connection, we must point out that the bearing shoes have deteriorated to the point prohibitive for reutilization.

Except limited locations, most of the rivet heads are in a relatively satisfactory condition and even in the worst case, only 5 per cent of the heads were found damaged.

c. Test pieces

On the Port Louis side abutment, 6 test pieces (45 x 6 cm)were taken from the end stiffener of the main girder by gas-cutting. Two pieces each were subjected to tensile, weldability and bend testing and two of them were further subjected to chemical composition analysis after rupture.
Table VII 6-2 shows results.

The results show that this steel conforms to the ultimate tensile strength of 28.0 ~ 33.0 ton/in^2 (4,400 ~ 5,197 kg/cm^2) as defined in the London County Council (General Powers) Act, 1909 and also to the prescribed allowable stress of 7.5 ton/in^2 (4,400 ~ 5,197 kg/cm^2) on tension, compression and bending as defined in the same Act.

As BS153 PARTS 3B and 4: 1972 prescribes the allowable stress according to the yield point, applicable steel should be GRADE 43-280 (280 $N/mm^2 \doteqdot 2,746\ kg/cm^2$), but its carbon content of 0.27 conforms to the requirements (below 0.30) of GRADE 43A (semi-killed, As rolled). Carbon equivalent defined in BS 4360 1972 para. 1-6-3 is as follows:

$$C.E. = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$

$$= 0.265 + \frac{0.54}{6} = 0.355$$

Therefore, the value of 0.41 prescribed for GRADE 43 C/D has not been exceeded. Phosphor (P) content does not exceed the values for GRADE 43 (0.050 to 0.060), but exceeds the value for JIS SM41/SM50 (0.040).

Sulphur content (S) is in far excess of the value for GRADE 43 (0.050 to 0.060) and therefore weldability is poor in respect of hot cracking and notch toughness.

Table VII-6-1 G.R.N.W. Disused Railway Bridge Corrosion Checks

Members \ Span		1st span Port Louis side	2nd span	3rd span	4th span	5th span	6th span	7th span "S" Hill side	Total
Upper flange (lower)	C	2	5	3	9	17	6	2	44
	D	2	0	0	0	0	0	0	2
Lower flange (upper)	C	7	2	1	1	0	1	2	14
	D	5	0	0	0	0	0	0	5
Stiffener	C	1	0	0	0	0	0	0	1
	D	7	8	2	1	0	0	0	18
Bearing shoes	C	0	0	0	0	0	0	0	0
	D	4	4	4	4	4	4	4	28

Note: 1) Flange corrosion was noted mostly in locations where stiffeners were attached due perhaps to accumulation of water and dust.

2) Stiffener corrosion is presumably attributable to the presence of residual stress concentration in the bending process.

3) Water and dust tend to accumulate near the bearing shoes, promoting corrosion. Most of the shoes have been virtually fixed and need to be replaced.

Table VII-6-2 Tabulation of Material Test Results

i) Tensile test

Test Piece	Dimensions			Tensile Test				Elongation	
	Gauge Length	Sectional Area		Yield Point		Tensile Strength		Actual	%
	m/m	Thickness x width	mm^2	Load (kg)	kg/mm^2	Load (kg)	kg/mm^2	m/m	
No. 1	200	16.1x40.5	652.1	19,100	29	30,200	46	251	26
No. 2	200	16.1x40.4	650.0	19,100	29	29,900	46	256	28

ii) Bending test

Test Piece	At Room Temperature		Cracking	Remarks
	Radius of Curvature	Bending Angle		
	(R)	(°)		
No. 3	1.5 t	180°	Nil	
No. 4	1.5 t	180°	Nil	

iii) Weldability test

Test Piece	Dimensions Sectional Area		Tensile Test Tensile Strength		Cutting Location
	Thickness x width	mm^2	Load (kg)	kg/mm^2	
No. 5	15.9x40.2	639.2	30,800	48	Welded part
No. 6	15.8x40.2	635.2	31,100	49	Welded part

iv) Chemical composition analysis

Test Piece	C (%)	Si (%)	Mn (%)	P (%)	S (%)	Ni (%)	Cr (%)	Mo (%)
No. 1	0.27	0.01	0.53	0.045	0.082	-	-	-
No. 2	0.26	0.01	0.55	0.047	0.078	-	-	-

(2) Substructure

a. Measurement of abutment and pier

Typical dimensions were measured with the aid of steel tapes and convex. Dry. 37/42 shows results. Also, refer to IX 2.5.2.

b. Structure and engineering survey

The periphery of two selective piers(P_1 and P_5) were excavated to check if the bridges were of the direct foundation type. Stones pitched on the surface were in part removed with the aid of breakers to confirm whether the interior of the body was made of plain concrete with cobbel stones.

c. Boring and test pitting

Boring was made near the abutment on the Beau Bassin side and test pitting was conducted in the vicinity of P_5 for elucidation of the properties of rock strata and geological structures. Paragraph VII 3.3 gives details.

d. Scouring and water depth

The depth of water was sounded near P_2 and P_3. Figure VII-6-1 shows results on P3.

In view of the fact that no powerful cyclones have hit during last ten years or so, rather large mossy stones were found at places, the influence of scouring is judged to be slight at present. Some provisions, however, should be made to protect the piers from potentially injurious scoaring in the future. These provisions are stated in paragraph IX.2.

e. Measurement of concrete strength

The strength of the caping beam concrete and cobbel-stone concrete in the body of P_6 was measured with the aid of a Schmit hammer. It is estimated that the former had a strength of 240 kg/cm^2 and the latter, 120 kmg/cm^2.

The body is protected with stones on its surface, and the cobblestone concrete showed no signs of ageing and was judged to have been in a satisfactory condition.

In case this bridge is employed as a road bridge, there will be no problem in concrete strength in view of the fact that compressive strength applied on the concrete is below σc = 15 kg/cm^2.

(3) Consideration

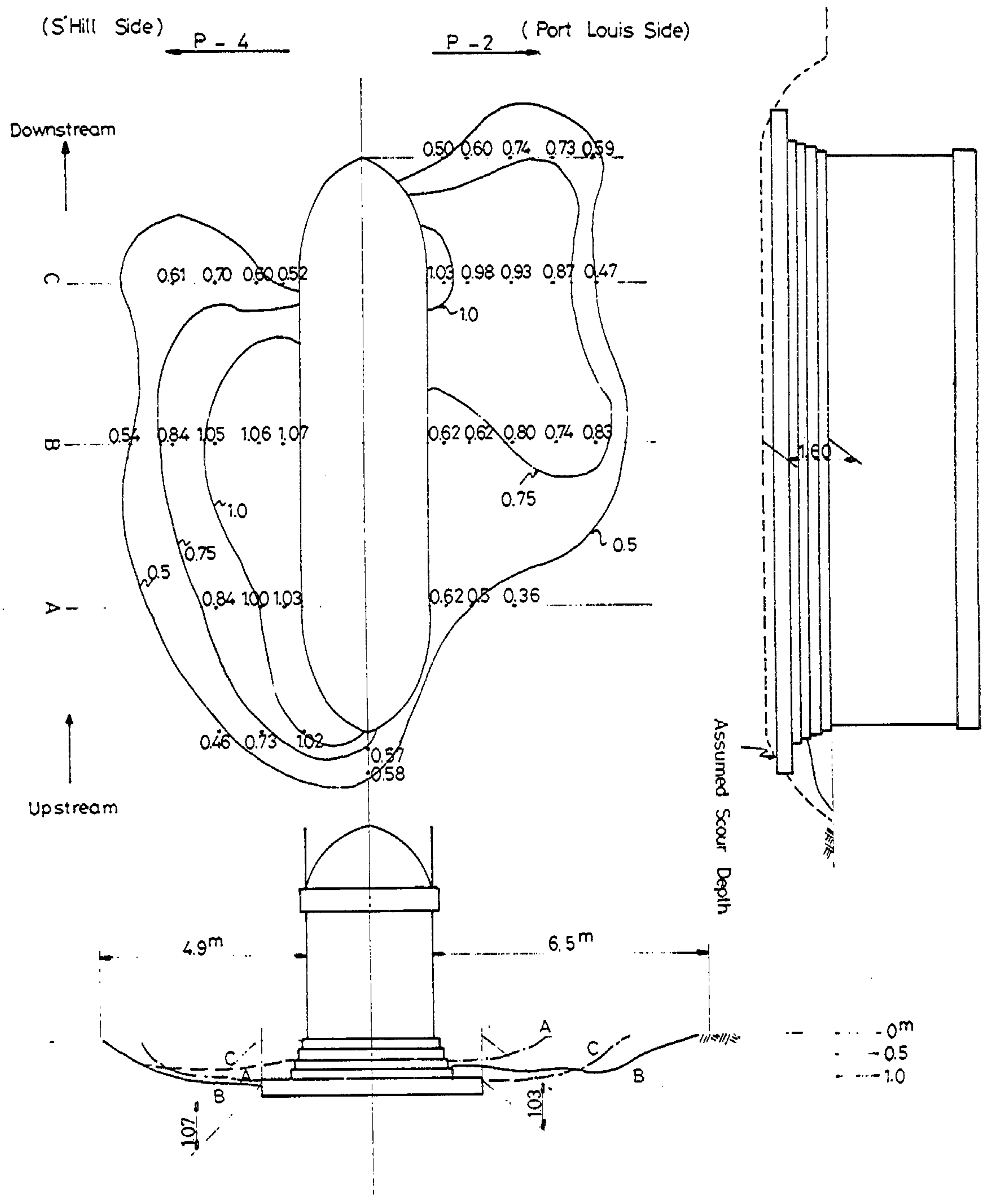
This bridge served as railway transport for as long as a half century after its completion. We conclude that, though partial renovation is necessary, the bridge has been maintained in a good condition which facilitates continued use.

As a result of steel tests, it was clarified that the entire superstructure has a sufficient strength and the substructures a corresponding strength. Whether the railway bridge can be converted into a road bridge or not should be decided through dynamic studies and economic analysis.

Figure V11 - 6 - 1

Present Scour Depth At P-3

S = 1 : 100



6.2 St. Louis Railway Bridge

This bridge is a deck bridge designed for double track, equipped with two parallel double girders. The structural details and the status of conservation of the girders vary between upper and lower streams.

(1) Investigation

As in the case of the G.R.N.W. railway bridge, dimensions and corrosion checks were made on the superstructure, including the confirmation of typical dimensions on substructures.

Drg.35/42,36/42 show the dimensions of upstream and downstream structures, while Table VII 6.3 shows the results of corrosion checks.

It is assumed that the upstream superstructure of this bridge was completed earlier than its downstream counterpart and does not incorporate solid structural design as viewed from the modern engineering technology.

Table VII-6-3 Corrosion Inspection Results on the Disused St. Louis River Railway Bridge

(Unit: location)

	Corrosion	Upstream Double Girder	Downstream Double Girder	Total
Upper flange (top)	C	4	20	24
	D	2	26	28
Bearing shoes	C	0	0	0
	D	4	4	8

Note: 1) Corrosion is in progress in the portions where cross beams are attached to the upper flange.

2) The bearing shoes have been placed under the same condition as that of the disused G.R.N.W. Railway Bridge and are in need of replacement.

6.3 Other Structures

(1) General

With other structures than those mentioned above, we conducted the review of drawings, field inspection and dimensional measurement, specifically on those structures installed along the disused railway track, the A_1 and M_2, typical examples of which are listed up in Table VII-6-4.

Relative bridge locations are specified in Figure VII-6-2.

(2) The estuary of the Grand River North West

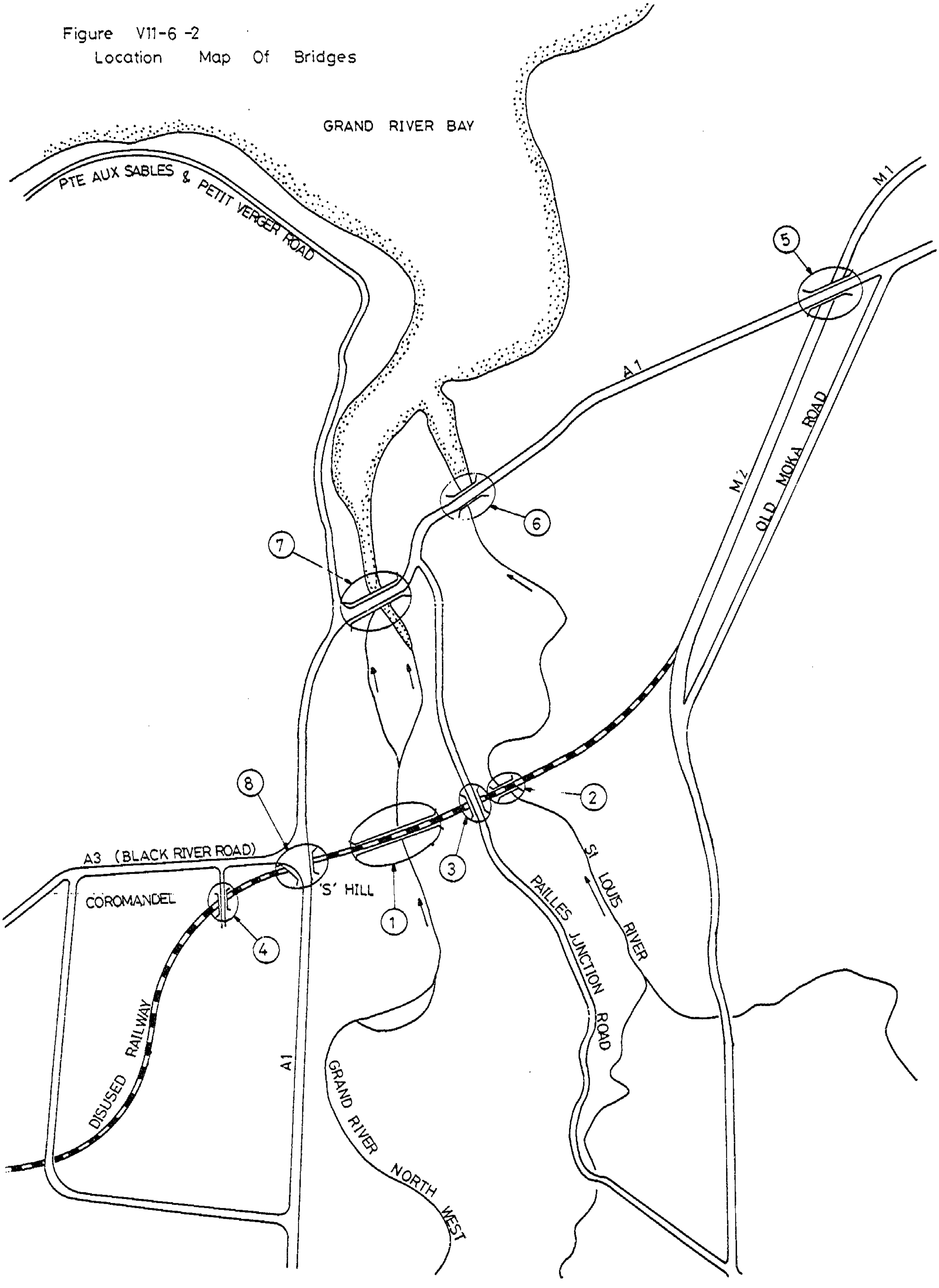
As there was an alternative plan of opening new routes downstream of the road bridge (on A_1 Road) spanning the G.R.N.W., we expanded the scope of survey including a boring operation.

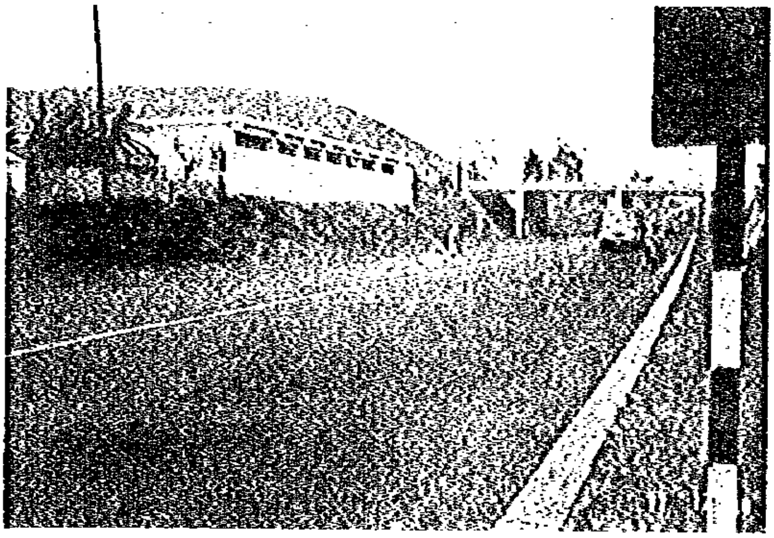
Also, we examined the difference in tide levels in Port Louis to obtain date forbridge spanning study in relation to the possibility of installing a new bridge and learned that the water level is 2,753'(0.84 m) at high tide and -0.645'(-0.20 m) at low tide, in January.

Table VII-6-4 Current Status on Bridges

Route	No.	Bridge Name	Type	Length (m)	Span (m)	Width (m)	Height of Substructure (m)	Present Conditions
Along the Disused Right of Railway	1	G.R.N.W. Railway Bridge	Steel plate girder for simply-supported bridge	193.0	7 @26.1	-	20.7 - 25.35	Refer to
	2	St. Louis Railway Bridge	Steel plate girder for simply-supported deck bridge	28.0	26.1	-	15.5	Refer to
	3	Pailles Over Bridge	Simply-supported reinforced concrete (T girder)	12.3	11.75	4.74	6.6	· Many cracks have been produced in the girder. · Concretes have been chipped off. Concrete-reinforcing rails are in part exposed. · Cracks have been produced in the wing as well.
	4	Coromandel Over Bridge	Simply-supported reinforced concrete slab bridge	5.8	5.3	3.45	5.7	· Slabs and abutments are stained with soot generated by the locomotive.
Along the A-1 Road	5	Route Royal Bridge	Pretensioning slab bridge	50.9	8.4 + 2 @16.9 + 8.4	10.8 (Footpath) 2 @1.9	7.3	Satisfactory
	6	Rouseel Bridge	Pretensioning slab bridge	24.3	4.3 + 15.1 + 4.3	9.55 (Footpath) 1.2	2.5	Satisfactory
	7	G.R.N.W. Bridge	Main span: simply-supported, curved-chord truss bridge Side span: stone arch bridge	137.3	48.9 + 3 @15.5	Main span 5.75 Side span 8.50	7.0 - 9.3	· The interior of truss panel points, bearing shoes and floor plates have been badly corroded.
	8	"S" Hill Over Bridge	Slab bridge with trough steel plate Reinforced concrete slab bridge	6.0	5.5	19.3	6.06 - 6.65	· Slabs and abutments are stained with soot generated by the locomotive.

Figure VII-6-2
Location Map Of Bridges



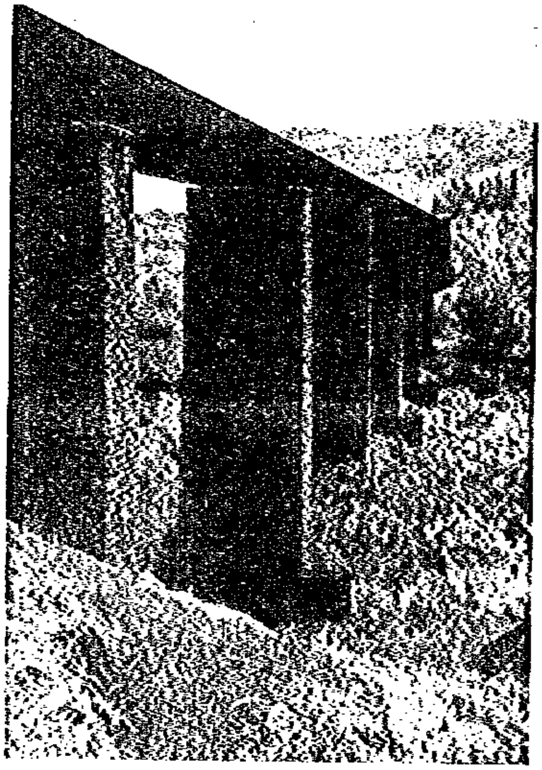


Motorway Flyover Bridge

At Flyover Bridge of Motorway

Motorway Belle Village

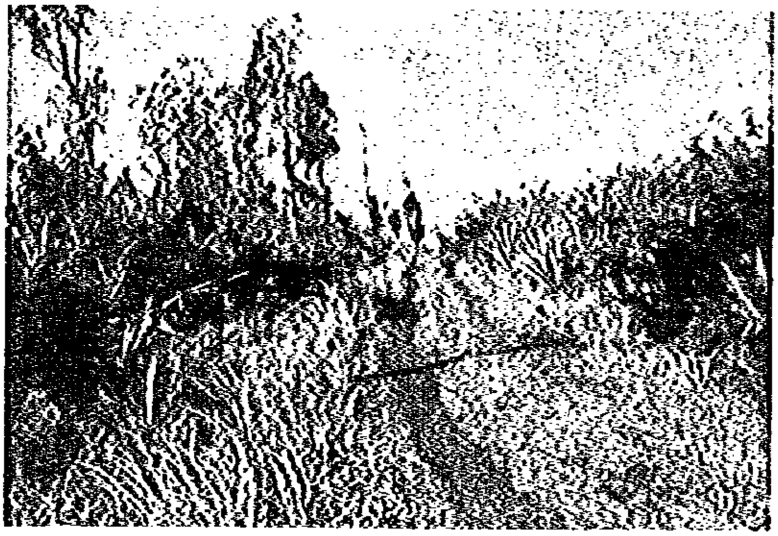
At Belle Village of Motorway



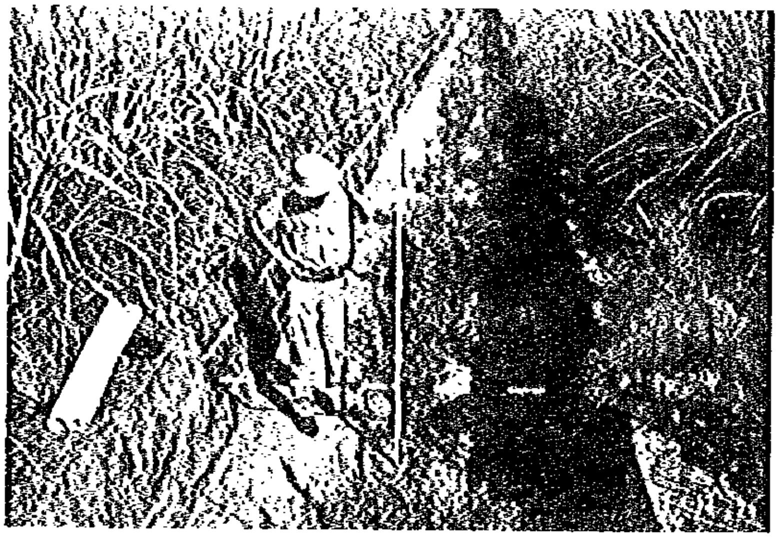
Disused Railway Bridge

Coromandel

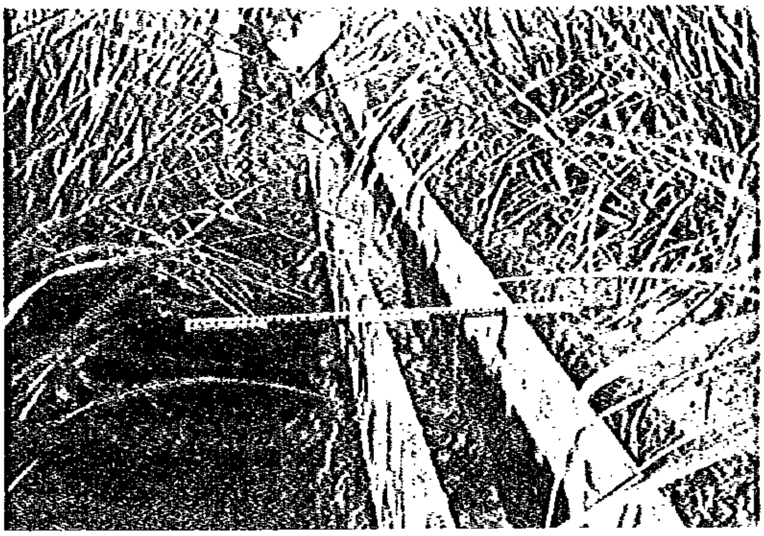
Disused Railway Track on the West Side of Coromandel



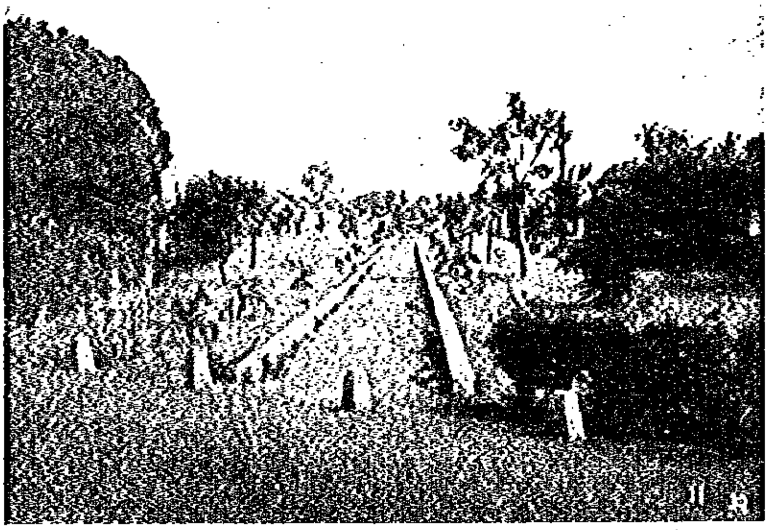
Disused Railway Track in Sugarcane Field



Irrigation Cannal Field Sugarcane



Irrigation Cannal in Sugarcane Field



Beau Bassin

Disused Railway Track near Beau Bassin Roundabout

CHAPTER VIII TRAFFIC FORECAST

CHAPTER VIII TRAFFIC FORECAST

The purpose of this Chapter is to estimate the volume of future traffic on the projected alternative networks, on the basis of the analysis of present traffic and economic parameters.

1. Methodology

The methodology of traffic forecast is outlined in the latter half of the overall traffic analysis framework as shown in Figure VIII-1-1.

1.1 Estimate of Future Traffic Demand

The volume and distribution of base, bus, airport, sugar, intra-link, and development traffic which will be generated in the future is estimated for years 1982, 1987, 1992, and 2002. The rate of traffic increase is estimated based on the estimated future values of economic indicaters (see Chapter IV).

1) Base Traffic

The total tripends of current (1977) base traffic estimated in Chapter V is projected toward the future years at the growth rate of total vehicle-miles estimated by type of vehicle in Chapter IV.2.

The generating and terminating trips of each zone was projected at either of the growth rate of night population or the growth rate of number of employment, depending on the period of hour. Because no major changes are expected in the future in the condition of transportation between zonal pairs, future distribution traffic volume is obtained by using the "present pattern method" and by accomplishing the convergence computation of the "Flator Method". The "present pattern method" obtains the future distribution traffic volume through computations repeated until the present distribution traffic volumes comes to coincide with future generation traffic volume and is so called because, under this method, the present distribution traffic volume is input as the hidden initial value upon the start of the convergence computation. The Flator Method is shown as follows:

$$Q_{ij}^{t} = Q_{ij}^{t-1} \cdot F_{i}^{t-1} \cdot F_{j}^{t-1} \cdot \left(\frac{L_{i}^{t-1} + L_{j}^{t-1}}{2} \right)$$

Where Q_{ij}^{t} : the traffic volume between i and j by t times of convergence computation.

F_{j}^{t-1}: the modification coefficient of zone i obtained by t times of convergence computation.

L_{j}^{t-1}: the location factor of zone i obtained by t times of convergence computation.

$$F_{i}^{t} = \frac{Ki}{Q_{i}^{t}}$$

Where Ki : the future generating traffic volume in zone i (control total : exsogenous).

Q_{i}^{t} : Generating traffic volume obtained by t times of convergence computation.

$$Q_{i}^{t} = \sum_{j} Q_{ij}^{t}$$

$$L_{i}^{t} = \frac{Q_{i}^{t}}{\sum_{j} (Q_{ij}^{t} \cdot F_{j}^{t})}$$

Then, this computation is repeated until L_{i}^{t} comes to 1.00.

2) Bus Traffic

Assuming that there would be little change in the existing bus routes, the future volume of bus traffic is estimated based on the rate of growth of average vehicle-miles of buses in the country.

3) Airport Traffic

The feasibility of relocation of the airport and of the construction of new access road to the new airport was studied in 1975. Because airport traffic is small in volume and because most of it is expected to use Motorway, the future volume of airport traffic is estimated based on the results of the said feasibility study, even though it is analyzed separately from the volume of base traffic.

4) Sugar Lorry Traffic

The future sugar lorry traffic volume is estimated based on the estimates of future sugar production and the types of cargo vehicles to be used for transportation of sugar. It is assumed that the total volume of traffic will originate or terminate in Port Louis. Seasonal change is considered.

5) Intra-Link Traffic

It is assumed that the current volume of intra-link traffic will increase in the future at the pace equal to the national increases in the average vehicle-miles of each type of vehicle.

6) Development Traffic

In the methodology of the traffic analysis as a whole, all future development projects have been taken into consideration in terms of increases in the number of employment and population. Likewise, the development project planned for the project area (Pointe aux Sables) has been reflected in the future population and the number of employment within the area. Thus, development traffic is included within the base traffic.

1.2 Traffic Assignment

The future volume, speed, vehicle-mile, and running hour of traffic on each of the links proposed under the alternative networks are estimated by assigning the estimated future O-D traffic on the alternative networks.

1) Concept

Every trip is naturally made in the preference of a route that would cause a minimum of inconvenience, or on the road which gives the shortest time-distance. But, such route would depend on the level of traffic congestion on any alternative road or roads running between a given set of origin and destination. Concentration of traffic beyond certain level onto a road which was initially the route of the shortest time-distance would result in a drop in vehicle speed due to traffic congestion, which impaires the desirability of that route, and a detour would then be preferred. To reflect the actual flow of traffic, the origin-destination traffic volume would have to be assigned to each link which has specific ratio between the volume of traffic and road capacity (time-distance is measured based on geographical distance and vehicle speed which is considered correlative to the ratio between volume of traffic and road capacity) according to its road standard. Allocation or assignment of traffic volume would not be made all at once among the alternative routes according to pre-determined ratio; the total traffic volume would rather be divided into several or several tens of increments which would be given one at a time according to the congestion degree among the alternative routes which change from time to time. The shortest time-distance route may thus change from one to another. While there are varied methods available for this traffic assignment, ours is a kind of convergence computation and, therefore, the more number of increments (into which the total traffic volume is divided), the more appropriate results obtainable.

2) Speed-Congestion Curve

The correlationship between traffic/capacity and vehicle speed has been determined for each type of major roads primarily based on the Japanese Road Structure Order and the Highway Capacity Manual of the United States, incorporating, as required, the result of our on-site spot speed surveys.

3) Origin-Destination Traffic to be Assigned

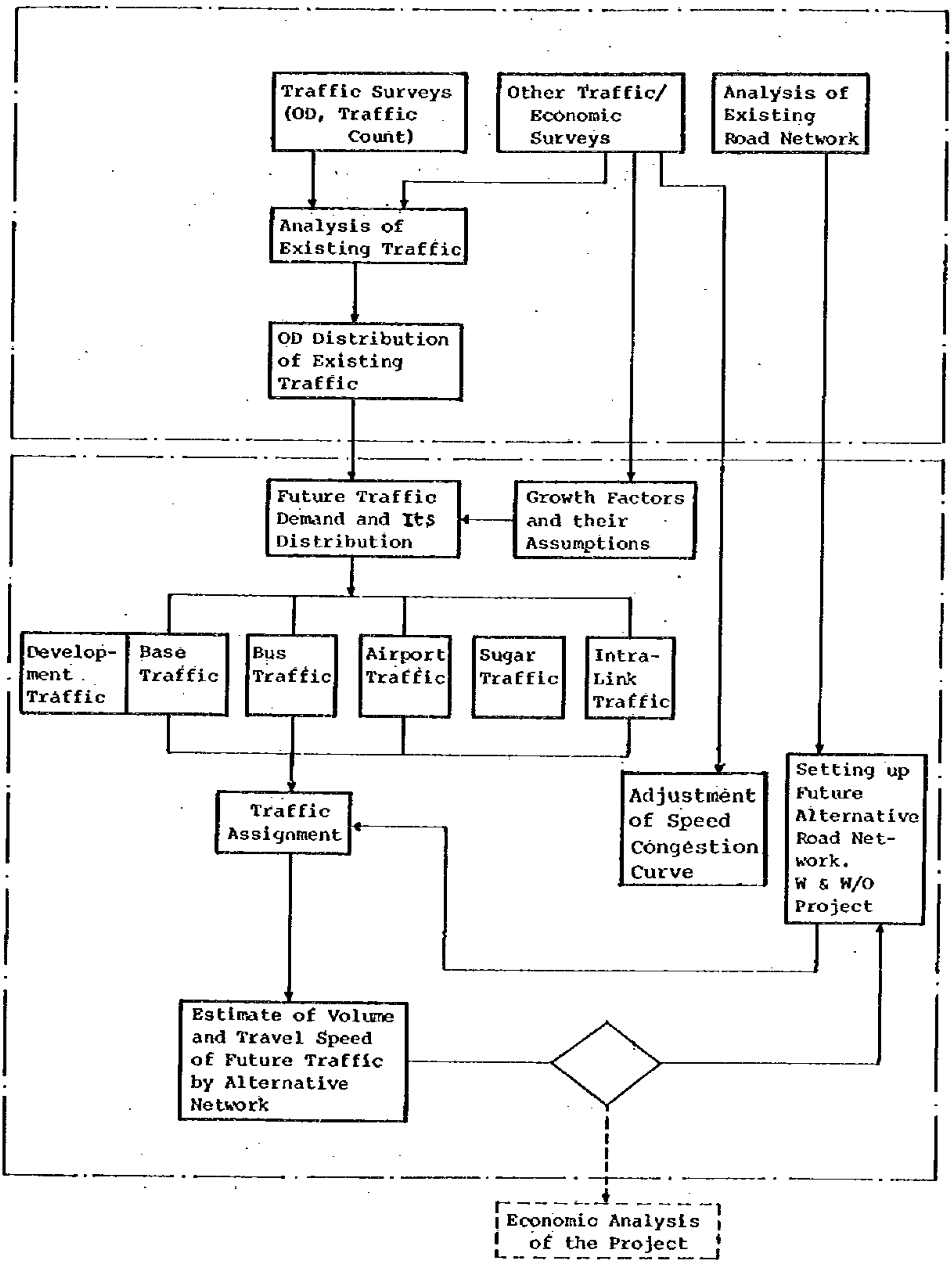
In view of that the chief role of the Project Road is the mitigation of traffic congestion, day is broken into morning peak, evening peak, and off-peak, hours for the purpose of O-D traffic assignment. The traffic is divided into five increments.

1.3 Road Networks for Traffic Assignment

O-D traffic is assigned to each of the projected alternative networks and to the network "without" the Project. A total of 12 alternative networks were analysed in an attempt to sort out the best or optimum alternative.

The details of the foregoing and the process of computations will be explained in the next Sub-Chapter.

Fig. VIII-1-1 Methodology of Traffic Analysis



2. Future Traffic Demand Forecast

2.1 Base Traffic

1) Total Tripends

It is conceivable that the future level of traffic will be determined by the future level of economic activities. As future number of motor vehicles and future vehicle-miles have been predicted in Chapter IV.1 on the basis of future population and future per capita GDP, the future number of total tripends are estimated, as presented in Table VIII-2-1-1 below, under the assumption that such number will be explained or determined by the growth rate of future vehicle-mile of each type of vehicle. No difference in the growth rate between the hours of day has been assumed.

Table VIII-2-1-1 Prediction of Total Tripends1/

Type of Vehicles	Period of Hour	1977	1982	1987	1992	2002
Car,Taxi	Morning Peak	2,544	3,745	5,403	7,609	14,162
	Evening Peak	2,511	3,618	5,221	7,352	13,684
	Off-Peak Hours	13,992	20,596	29,719	41,850	77,893
Van	Morning Peak	198	320	461	642	1,168
	Evening Peak	168	269	387	538	979
	Off-Peak Hours	1,797	2,908	4,187	5,828	10,602
Truck	Morning Peak	214	296	394	523	923
	Evening Peak	150	208	276	366	647
	Off-Peak Hours	1,924	2,663	3,538	4,700	8,298

1/ 12 hour daily average excluding Sunday.

2) Future Generating and Terminating Trips by Zone

The future volume of generating and terminating traffic for each zone can not necessarily be explained by one growth rate for all hours of day and all types of vehicles. For instance, in the case of passenger vehicles in the morning peak, generation would be explained by population but termination, by the number of workers while, in the evening peak, volume of generation would be explained by the number of workers but termination, by population. And, in case of cargo vehicles, it is believed that both generation and termination can be better explained by the number of workers in all hours of day. Therefore, future generation and termination of trips in each zone have been predicted at the growth rate (see Chapter IV.2.2) of economic indicators listed in Table VIII-2-1-2 and the total thus predicted has been adjusted to coincide with the total future tripends. Also, the estimated future volumes of generating and terminating traffic are presented by zone as in Appendix Table VIII-1.

Table VIII-2-1-2 Parameters used for Estimating the Growth of Zonal Traffic

Period of Hour	Type of vehicle	Originating Traffic	Terminating Traffic
Morning Peak	Passenger	Population	No. of jobs
	Freight	No. of jobs	No. of jobs
Evening Peak	Passenger	No. of jobs	Population
	Freight	No. of jobs	No. of jobs
Off-peak Hours	Passenger	Population	Population
	Freight	No. of jobs	No. of jobs

3) Future Traffic Distribution

The distribution of future base traffic has been estimated by applying the Flator Method, which was explained in 1. above in this Chapter. The estimation has been done by the type of vehicle, by period of hour and by year. Of the results of the estimate, the distribution of 12-hour traffic in 1992 is presented in Tables VIII-2-2 through VIII-2-5. The detail of the estimate is shown in Appendix Table VIII-2.

Table VIII-2-2 Distribution of 1992 Traffic 1/ (Cars)

Vehicles

O \ D	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	Total
01	16	1,514	1,629	2,330	3,123	2,029	2,657	1,651	27	127	128	41	4,562	261	487	185	86	2	20,855
02	1,736	-	-	-	-	-	779	-	-	-	-	-	-	-	-	-	-	215	2,730
03	2,561	-	-	-	-	-	265	-	-	-	-	-	-	-	-	-	55	209	3,090
04	2,763	-	-	-	-	-	341	-	-	-	-	-	-	-	-	-	100	210	3,414
05	3,766	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	539	4,305
06	2,539	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	316	2,855
07	2,893	905	718	427	183	160	-	788	-	-	60	-	-	-	-	-	143	256	6,533
08	1,514	-	-	-	-	-	713	-	-	-	-	-	324	-	261	-	229	114	3,155
09	134	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	38	172
10	301	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	24	325
11	103	-	-	-	-	-	-	-	-	-	-	-	-	-	-	125	-	23	251
12	95	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	103	198
13	3,378	-	-	-	-	-	-	287	-	-	-	-	-	-	-	-	-	488	4,153
14	449	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	73	522
15	862	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	129	991
16	327	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	72	399
17	89	153	38	55	66	32	37	28	-	-	-	-	-	-	-	-	-	-	498
18	5	276	192	317	319	122	435	262	23	24	10	42	265	-	72	-	-	-	2,364
Total	23,531	2,848	2,577	3,129	3,691	2,343	5,227	3,016	50	151	198	83	5,151	261	820	310	613	2,811	56,810

Table VIII-2-3 Distribution of 1992 Traffic 1/ (Vans)

Vehicles

O \ D	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	Total
01	1	741	190	286	206	213	370	211	-	15	23	20	227	87	204	62	8	-	2,864
02	784	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	112	896
03	152	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	30	182
04	181	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	76	23	280
05	211	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	39	250
06	235	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	16	251
07	76	558	35	-	-	-	-	55	-	18	-	-	-	-	-	-	22	3	768
08	99	-	-	-	-	-	-	-	-	-	-	-	215	-	-	-	-	15	329
09	12	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	12
10	23	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	23
11	32	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	11	43
12	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	10
13	372	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	53	425
14	45	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	8	53
15	194	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	194
16	28	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	28
17	11	-	-	-	-	-	24	67	-	-	38	-	-	-	-	-	-	-	140
18	-	79	29	15	15	16	79	-	-	2	4	15	-	8	3	-	-	-	265
Total	2,466	1,378	255	301	221	229	473	333	-	35	65	35	442	95	207	62	106	310	7,013

1/ 12 hour daily average excluding Sunday.

Table VIII-2-4 Distribution of 1992 Traffic 1/ (Trucks)

Vehicles

O \ D	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	Total
01	1	373	114	135	192	208	240	151	8	10	5	18	190	86	177	51	-	-	1,959
02	430	-	-	-	-	-	162	-	-	-	-	-	-	-	-	-	-	183	775
03	105	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	7	27	139
04	119	-	-	-	-	-	52	-	-	-	-	-	-	-	-	-	-	8	179
05	218	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	43	261
06	311	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	58	369
07	161	155	-	61	-	-	-	84	-	34	-	-	-	-	-	-	28	69	592
08	223	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	17	23	263
09	2	-	-	-	-	-	-	-	-	-	-	-	-	-	19	-	-	-	21
10	28	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	29
11	48	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	48
12	11	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	10	21
13	188	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	192
14	47	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	6	53
15	128	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	132
16	51	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	51
17	5	75	-	9	-	-	16	13	-	-	-	-	-	-	-	-	-	-	118
18	-	116	26	8	37	39	32	33	-	5	32	-	34	11	4	10	-	-	387
Total	2,076	719	149	213	229	247	502	281	8	49	37	18	224	97	200	61	52	436	5,589

Table VIII-2-5 Distribution of 1992 Traffic 1/ (Total)

Vehicles

	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	Total
01	18	2,628	1,933	2,751	3,521	2,450	3,267	2,013	35	152	156	79	4,979	434	868	298	94	2	25,678
02	2,950	-	-	-	-	-	941	-	-	-	-	-	-	-	-	-	-	510	4,401
03	2,818	-	-	-	-	-	265	-	-	-	-	-	-	-	-	-	62	266	3,411
04	3,063	-	-	-	-	-	393	-	-	-	-	-	-	-	-	-	176	241	3,873
05	4,195	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	621	4,816
06	3,085	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	390	3,475
07	3,130	1,618	754	488	183	160	-	927	-	52	60	-	-	-	-	-	193	328	7,893
08	1,836	-	-	-	-	-	713	-	-	-	-	-	539	-	261	-	246	152	3,747
09	148	-	-	-	-	-	-	-	-	-	-	-	-	-	19	-	-	38	205
10	352	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	25	377
11	183	-	-	-	-	-	-	-	-	-	-	-	-	-	-	125	-	34	342
12	116	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	113	229
13	3,938	-	-	-	-	-	-	287	-	-	-	-	-	-	-	-	-	545	4,770
14	541	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	87	628
15	1,184	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	133	1,317
16	406	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	72	478
17	105	228	38	64	66	32	77	108	-	-	38	-	-	-	-	-	-	-	756
18	5	471	247	340	371	177	546	295	23	31	46	57	299	19	79	10	-	-	3,016
Total	28,073	4,945	2,972	3,643	4,141	2,819	6,202	3,630	58	235	300	136	5,817	453	1,227	433	771	3,557	69,412

1/ 12 hour daily average excluding Sunday.

2.2 Bus Traffic

The estimation of future bus traffic has been done under the following basic assumptions:

1) The current bus routes will not be changed basically in the future.

2) Growth of bus traffic volume will be in indirect proportion to the growth of average vehicle-miles of all buses on the Island.

The completion of the Project Road can very likely result in the establishment of a new bus route on that road for its entire extension or a part of it, as well as in some modification in the existing bus routes. To develop predicted bus routes for each of the alternative networks, however, will call for an exorbitant amount of work resulting at so many predictions with insufficient dependability, as anticipated changes in government and bus company policies to which the future bus routes will be highly susceptive will have to be input in the computation. In addition, it is unlikely that population distribution and urban structure will be drastically changed in the foreseeable future. Therefore, it has been assumed that the future bus routes will be the same as the existing routes.

While the frequency of bus service will change keeping pace with changes in population distribution and the growth rate of population in the project area, such changes would differ from one zone to another. Future volume of bus traffic has been estimated, therefore, based on the future growth of average vehicle-miles of all buses on the island, as estimated in Chapter IV.2.2., in the view that bus service of each route is not limited to the inhabitants of any particular zone but covers the population of a wider area and that future growth of the population of the project area roughly coincides with that of the entire island.

As a result, the volume of future bus traffic has been estimated for each of the major road sections as presented in Appendix Table VIII-3.

2.3 Sugar Lorry Traffic

The future volume of sugar lorry traffic has been estimated separately from all others for the following reasons:

1) That the on-site traffic survey was taken at the time when sugar industry traffic in terms of the volume of cargo was not in normalcy.

2) Sugar is being transported by relatively distinct means and ways of transportation whose origin and destination are well defined.

Average annual production of sugar during the past five years has been almost constant at about 698,000 tons except for 1975, when the crop was damaged by cyclone. Because further enlargement in the real size of sugar-cane field cannot be expected, any future sugar production increase will have to depend on soil improvement, expansion of irrigated area, fertilizer, extermination of insects and diseases, and other productivity increase efforts. The Five-Year Plan (1975-1980) provides for the implementation of a series of such productivity improvement measures under the objective of accomplishing the production of 800,000 tons of sugar by 1980 by the average annual increase of 3%. Performance has so far been lagging behind the Plan goals due partly to the cylone damage in 1975. However, it has been assumed that all such measures will fundamentally be implemented as planned, and the future sugar production has been estimated under the growth rates as presented in Table VIII-2-6 below. Future increases in sugar production in each zone have been assumed to be the same as increases in the national total and, thus, estimated as presented in Appendix Table VIII-4.

Table VIII-2-6 Forecasted Sugar Production

	Base Year (1976)		1982		1987		1992		2002
Cane production (000 tons)1/	6,231								
Sugar production (000 tons)	698		790		851		894		940
Average annual growth rate (%)		2.0		1.5		1.0		0.5	

1/ Cane production was estimated by applying the same sugar recovery rate as of that of 1976.

The same rate of molasses production to sugar production has been assumed for all zones. Port Louis is considered the destination of sugar industry traffic regardless of origin. Most frequently, sugar is transported by 12 ton truck-trailers and molasses by 7.5-ton tank trucks, and the volume of production has been translated into the number of trucks accordingly. It has been assumed that input materials are all transported to sugar mills via return trip of these trucks. The results of estimation is presented in Appendix Table VIII-4.

2.4 Airport Traffic

It is expected that the Plaisance Airport will be relocated to the northern part (near Belle Vue Maurel) of Flacq District in the near future. Because it is highly possible that the pattern of airport traffic will be substantially changed upon relocation, the future volume of airport traffic has been estimated as stated below in view of the availability of the under-mentioned feasibility studies:

1) Traffic to be generated by airport: Estimates by "Access Road to the New Airport--Feasibility Study" by the Bureau Central D'Etudes pour les Equipments D'Outre-Mer (BCEOM) (see Table VIII-2-7) has been used.

2) Under the assumption that the relocation of the airport will take place in or about 1985, estimates for 1987, 1992, and 2002 have assumed the emergence of the new airport. At this time, traffic distribution follows the pattern presented in said Feasibility Study Report, provided, however, that the zones used by the Report, which differ from the zones of this project to a fair degree, have been reorganized by the following indicators depending on the purpose of trip:

   Tourists: The number of hotel beds (see Table III-3-4)

   Non-Tourist Passengers : Population (see Table IV-2-4)

   Local visitors to airport: Population (same table)

   Airport staff: Population (same table)

   Cargo: The number of workers (see Table IV-2-6)

3) As for the volume of airport traffic in 1982, the volume estimated by the said Report has been distributed by the current pattern (see Table V-3-12).

Table VIII-2-7 Forecast of Airport Generated Traffic 1/

vehicles/day

Passenger Type	Vehicle Type	1982	1987	1992	2002
Tourist	Car	379.0	496.0	570.0	670.0
	Bus	39.6	51.6	59.0	76.0
Non-Tourist	Car	508.0	837.2	1,264.4	2,236.4
Passenger	Bus	2.8	4.8	6.8	10.8
Visitor	Car	353.2	533.0	731.0	1,151.0
	Bus	2.4	3.4	4.4	6.4
Airport	Car	224.8	320.4	451.2	757.2
Staff	Bus	6.8	8.0	8.0	8.0
Goods	Truck	21.4	50.4	144.2	490.2
	Car	1,465.0	2,186.6	3,016.6	4,814.6
Total	Truck	21.4	50.4	144.2	490.2
	Bus	51.6	67.8	78.2	101.2

1/ in average daily two-way traffic

Source: Modification of the Results of "Access Road to the New Airport - Feasibility Study" (BCEOM)

For this purpose, buses have been ignored in the absence of basic data needed for the prediction of traffic distribution and because the number of buses running to and from the airport is relatively small.

The distribution of future airport traffic is presented by zone and by year as shown in Appendix Table VIII.5.

2.5 Intra-Link Traffic

The future volume of intra-link traffic has been assumed to increase in direct proportion to the growth of vehicle-miles of each type of vehicle as estimated in Chapter IV.2.2.

The intra-link traffic is not necessarily limited to those which move only within a link but includes all the traffic which do not come under any of the traffic discussed so far in the above. For instance, traffic moving between Beau Bassin and Curepipe is treated as intra-link traffic of each link concernred. Although the volume of local trips can be explained by the population or the number of workers of the particular area and the volume of longer trips by the average vehicle-miles of all traffic on the island, said treatment has been used here in the absence of breakdown of intra-link traffic. Also, motorcycle traffic, which is substantial, has been considered in estimating the future volume of intra-link traffic. The result of this estimation is presented in Appendix Table VIII-6.

3. Traffic Assignment

3.1 Method and Conditions of Assignment

1) Method

While the basic concept of traffic assignment was discussed in 1. above in this Chapter, the method of assignment is illustrated in detail in Figure VIII-3-1.

The traffic assignment is accomplished roughly in two steps. In the first step, bus, sugar lorry, airport, and intra-link traffic are assigned. In the case of bus and intra-link traffic, the values of estimate done in Chapter VIII.2. are assigned to each link under the assumption that their routes remain the same irrespective of the level of congestion in the link. Sugar lorry and airport traffic are assigned by the demand assignment method (under which origin-destination trips are assigned to the route of minimum time-distance by the initial speed established for each road shown in Figure VIII-3-2). With regard to the assignment of hourly traffic, bus and intra-link traffic are assigned in accordance with the current hourly variation and airport and sugar lorry traffic is assigned to off-peak hours.

In the second step, base traffic is distributed under the actual assignment method with the traffic volume assigned to each link and each period of hour in the first step as the given volume of traffic. The passenger car conversion equation, Speed-Congestion Curve, networks to which traffic is assigned, link conditions, and traffic capacity computation formula are explained in detail in the following sub-chapters. This assignment methods have been tested based on the volume of traffic in 1977 and the various conditions mentioned above and are presented briefly in Appendix VIII-7.

The resultant output:

(a) Traffic volume (actual munber)

(b) Traffic volume in terms of P.C.U.

(c) Vehicle speed

(d) Congestion ratio

(e) Vehicle-hours

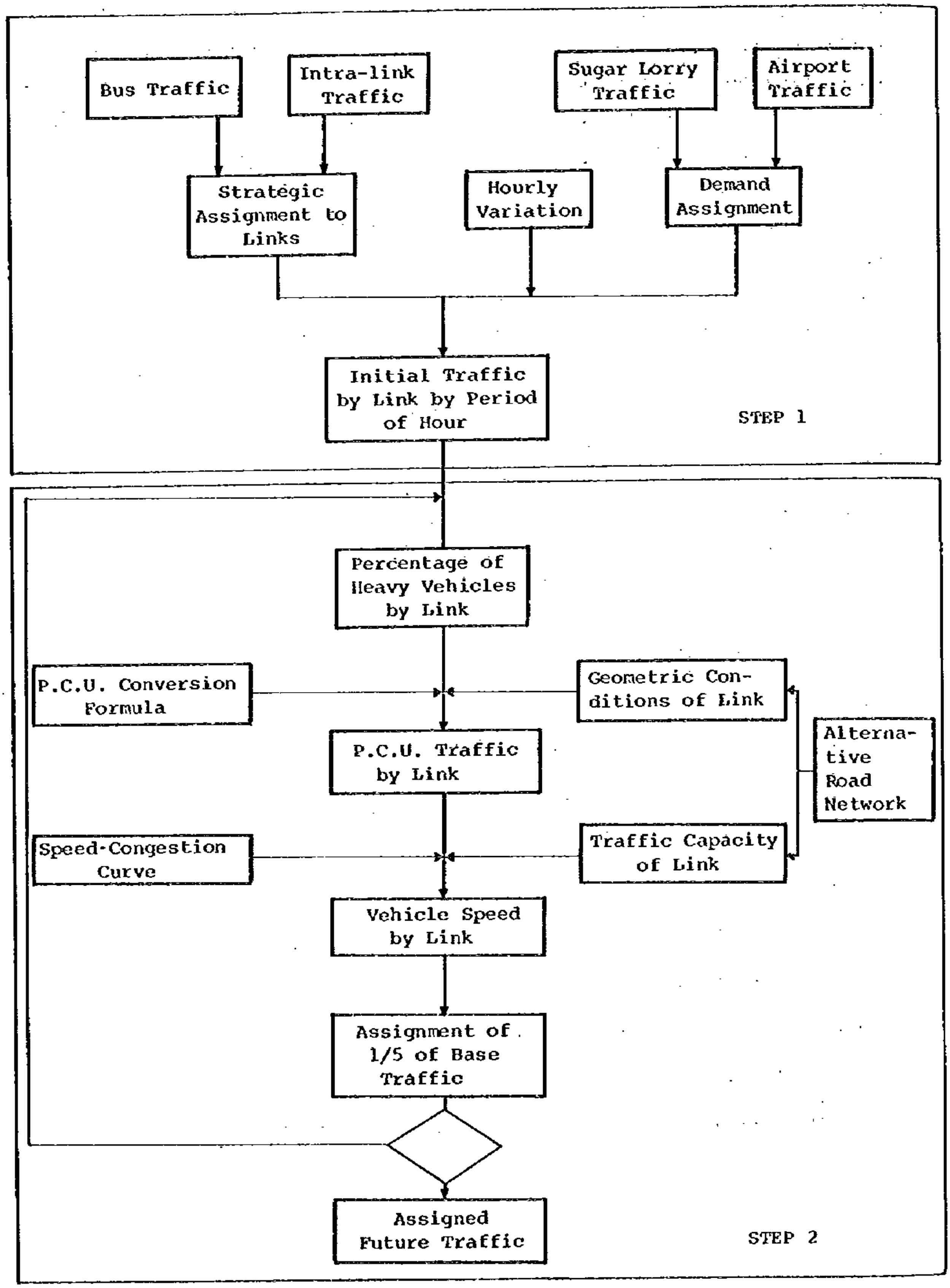
(f) Vehicle-miles

for each of the combinations of:

(1) Year: 1982, 1987, 1992 and 2002.

(2) Period of Hour: Morning peak, evening peak, off-peak hours

Fig. VIII-3-1 Methodology of Traffic Assignment



(3) Type of vehicle: Passenger cars (including taxi cabs), vans, trucks

(4) Link: Number differs from alternative network.

2) Conversion Equation of Passenger Car Unit (P.C.U)

Because the volume of traffic of various types of vehicles must be converted into the number of passenger cars under the assignment method used here, the following equation has been arrived at by accomplishing multiple regression analysis based on the P.C.U. conversion ratio of large vehicles which is shown in Appendix Table VIII-8. The equation assumes different conversion rates depending on the longitudinal gradient of the road, the length of gradient section, and the percentage of large vehicles in the traffic, and the values presented in Appendix Table VIII-8 have been derived from various surveys and research conducted in Japan.

Two-lane Road:

$$K = 1.1272 + 0.3906C + 0.3431L - 0.8429M \quad (r = 0.9565)$$

Multi-lane Road:

$$K = 0.7599 + 0.3626C + 0.3121L - 0.2871M \quad (r = 0.9520)$$

Where

K : P.C.U. conversion coefficient

C : Longitudinal gradient (%)

L : Gradient length (Km)

M : Rate of large vehicles in traffic

3) Computation of Traffic Capacity

The traffic capacity of road is computed by the following equation:

$$Q = B \times \alpha \times \beta \times C$$

Where

Q : Traffic capacity (number of vehicle/hour)

B : Basic traffic capacity (vehicle/hour)

α : Adjustment rate by the width of drive lane

β : Adjustment rate by lateral clearance

C : Adjustment rate by roadside conditions

(a) The following value is used as the basic traffic capacity (B):

Two-way total of 2-lane road:	2,500 vehicles/hour
Per lane on multi-lane road :	2,500 vehicles/hour

(b) The following values are used as adjustment rate by the width of lane (α):

Lane width (m) :	3.5	3.25	3.0	2.75
Adjustment Rate:	1.0	0.94	0.85	0.77

(c) The following values are used as adjustment rate by lateral clearance (β), with the distance between the road curb and an obstacle as the lateral clearance:

Lateral Clearance (m)			1.75	1.50	1.25	1.00	0.75	0.50	0
2-lane road:	lack on one side	:	1.00	0.98	0.96	0.93	0.91	0.88	0.85
	lack on both sides:		1.00	0.96	0.92	0.86	0.81	0.75	0.70
Multi-lane : road	lack on one side	:	1.00	1.00	0.99	0.98	0.97	0.95	0.90
	lack on both sides:		1.00	0.99	0.98	0.97	0.94	0.90	0.81

(d) The following values are used as adjustment rate by roadside conditions (C), depending on the degree of urbanisation of road-side area:

Degree of Urbanisation		Adjustment Rate
3. Not at all	:	1.00
2. Some	:	0.90
1. Fully	:	0.80

In the calculation of traffic capacity, the following consideration is given to the climbing lane:

Two-lane + Climbing Lane:

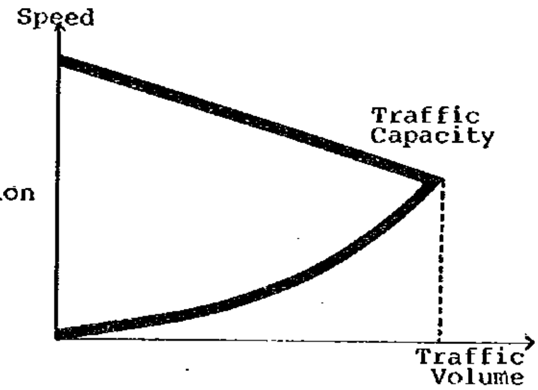
The same traffic capacity is assumed as 2-lane roads, but reduction in capacity (increase in traffic) in ascending direction on account of the mixture of large vehicles will be mitigated by the use of climbing lane by large vehicles. Thus,in ascending direction, one large vehicle is counted as one passenger car, and, in decending direction,the regular P.C.U. conversion rate for two-lane road is used.

Four-lane + Climbing Lane:

Under the same idea as above, the same traffic capacity is assumed as 4-lane roads, each large vehicle is counted as one passenger car in the ascending direction, and the regular conversion rate for multi-lane roads is used in descending direction.

4) Speed-Congestion Curve (Q-V Equation)

Generally speaking, the average speed of vehicles is strongly related to the volume of traffic on the road:



as the level of traffic rises, the speed drops, and when the traffic volume reaches the capacity of road, both the volume of traffic and speed start declining. This speed-congestion relationship can be illustrated by a curve presented in the above chart. However, because roads of different standards and different road conditions should perform on different speed-congestion curves, speed survey was conducted on the A_1 Road (at Grand River North West Bridge and Coromandel) and on Motorway. Particularly in the case of the A_1 Road, on which traffic congestion has already been observed, 5-minute observation at Grand River North West Bridge resulted in a finding that when the volume of traffic is 1.5 times the capacity, speed drops down to about 15 Km/hour. In contrast to this 5-minute observation on the short distance, actual congestion ratio (traffic volume-traffic capacity) during one hour of peak time was 1.26, indicating that traffic congestion is actually occurring in this hour.

With this in mind, we have obtained the congestion curves of Figure VIII-3-2 by adjusting them in such a manner as to bring the result of traffic assignment to the existing network which is shown in Appendix VIII-7. whould come to the proximity of the actual level of traffic observed.

Such curve has been obtained for each of the three roads of different standards, but in all cases speed diminishes as congestion builds up until congestion ratio reaches 1.0, after which speed drops rapidly down to only 5.0 Km/hour.at the congestion ratio of 1.3. These curves, which indicate that traffic must be moving at the speed of about 10 Km/hour on Grand River North West Bridge where congestion ratio is 1.26, are believed realistic.

Traffic cannot in reality increase further when vehicle speed has dropped down to about 5 Km/hour. But because the estimation is extremly difficult for the economic loss which is attributable to the unsatisfied traffic demand, it is assumed for the purpose of this project that the volume of traffic moving at the speed of 5 Km/hour continues to increase after congestion ratio of 1.3 has been reached, and the amount of additional vehicle operating costs incurred due to congestion is regarded as the imputed economic loss.

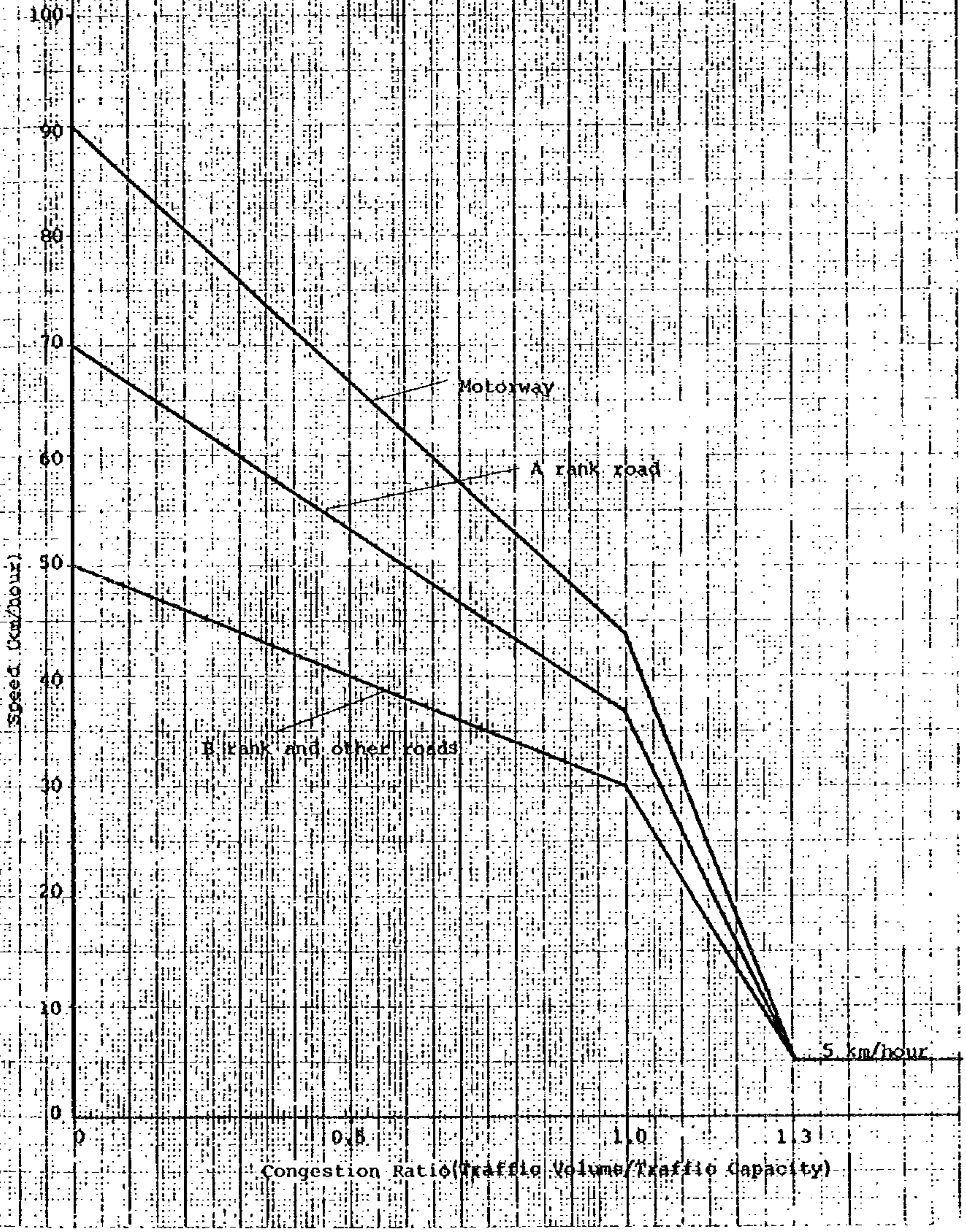
5) Implementation of Related Projects

With regard to related projects, the followings have been assumed for the purpose of traffic assignment:

(1) Airport: The new airport will be opened in 1985.

(2) Development in Pointe aux Sables will progress gradually and will be completed by 1992.

Fig. VIII-3-2 Speed-Congestion Curve



3.2 Networks Subject to Traffic Assignment

Traffic has been assigned to each of the 6 alternative road networks, that is, the combination of the existing network with each of the projected alternatives, as well as the existing network by itself.

Each of these networks have been broken into links, each of which has relatively homogenous road conditions. See Appendix Figure VIII-9 and Appendix Table VIII-10 for the conditions (number of lanes, the length of link, longitudinal gradient, lane width, lateral clearance, roadside condition, traffic capacity, the availability of climbing lane) of each network and each link.

Table VIII-3-1 Alternative Network for Traffic Assignment

Alternative Case	Description
W.O.	o existing network.
P_2	o existing network with a new two-lane road link between Belle Village (M2) and Beau Bassin via S. Hill
P_4	o existing network with a new dual carriageway road link between Belle Village and Beau Bassin.
P_A	o existing network with a new G.R.N.W. bridge in replacement of the existing road bridge along Al Road.
$P_2 + P_A$	o alternative network case P_2 with a new G.R.N.W. bridge but without the existing bridge.
$P_4 + P_A$	o alternative network case P_3 with a new G.R.N.W. bridge but without the existing bridge.

3.3 Result of Assignment and Evaluation of Alternative Networks

The traffic volume and congestion ratio on each of the alternative networks are shown in Appendix Figures VIII-11 and VIII-12, respectively.

1) Traffic Volume at Major Points

Table VIII-3-2 shows the future traffic level at major points. There is little difference between the alternative cases in the rate of increase in the volume of traffic at Belle Village on Motorway. This is naturally because, while the Project Road is to be constructed to accommodate trips of a relatively short length, Motorway is used for the purpose of longer trips.

The volume of traffic on G.R.N.W. Bridge, which is the most serious bottleneck in the existing network, is shown to increase gradually and reach three times the current level in year 2002, under Case W.O. Such impossible level of traffic has been shown because of non-availability of a substitute route. While similar increase of traffic volumes are shown under Cases W.O. and P_A' reduction of traffic up to 1992 is shown under other Cases, clearly indicating a shift of traffic and the improvement of congestion in this section.

Traffic on A_1 Road at Coromandel is shown to increase under all Cases other than Cases P_2, P_4, P_2+P_A, and P_4+P_A; these four cases assume the complete opening of the Project Road in 1982 and under such assumption the traffic is shown to decrease. This means that traffic at this particular point may not be improved unless Motorway and Beau Bassin are connected by a two-lane road or a dual-carriageway road.

As for the volume of traffic on the Project Road at G.R.N.W. New Bridge, Alternative Cases P_2, P_4, P_2+P_A, and P_4+P_A show a substantial yearly increases, indicating a substantial degree of traffic diversion from the existing network. However, the volume of traffic differs little between the completion of 2-lane road and the completion of dual carriageway road in this section except in year 2002.

Table VIII-3-1 Estimated Future Traffic[1/] at Major Sections of Alternative Network

Alternative Network (Case No.)	Year	Coromandel (A_1)	G.R.N.W. Bridge (A_1)	Belle Village (M_2) to P. Louis	Belle Village (M_2) from P. Louis	G.R.N.W. New Bridge (Project road) to P. Louis	G.R.N.W. New Bridge (Project road) from P. Louis
W.O. (Existing Network)	1977	18,014	15,829	8,321	8,218		
	1982	22,492	20,902	14,426	13,660		
	1987	26,118	26,873	20,943	20,504		
	1992	27,936	34,466	28,973	28,287		
	2002	32,679	59,918	49,623	49,427		
P_A (with New G.R.N.W. Bridge)	1977	-	-	-	-		
	1982	22,861	21,268	14,144	13,614		
	1987	26,690	28,007	20,693	19,935		
	1992	28,343	36,221	28,521	27,885		
	2002	32,679	59,720	49,715	49,684		
P2 (with two-lane road between M2 and Beau Bassin)	1977	-	-	-	-	-	
	1982	15,897	6,683	13,692	13,150	15,115	
	1987	20,392	9,183	19,298	18,584	22,706	
	1992	23,179	12,620	25,612	24,904	32,930	
	2002	29,174	38,447	43,221	43,081	40,058	
P4 (with dual-carriage way road between M2 and Beau Bassin)	1977	-	-	-	-	-	-
	1982	15,897	6,683	13,589	13,018	7,578	7,210
	1987	20,368	9,122	19,130	18,407	11,385	11,043
	1992	22,845	10,460	25,542	24,675	17,466	17,044
	2002	29,174	17,221	45,467	42,811	32,814	33,633
P_2+P_A	1977	-	-	-	-	-	
	1982	15,897	6,683	13,692	13,150	15,115	
	1987	20,392	9,183	19,298	18,584	22,706	
	1992	23,179	12,723	25,632	24,904	32,835	
	2002	29,174	41,318	42,706	42,109	39,672	
P_4+P_A	1977	-	-	-	-	-	-
	1982	15,897	6,683	13,589	13,018	7,578	7,210
	1987	20,368	9,122	19,130	18,407	11,385	11,043
	1992	22,845	10,460	25,542	24,675	17,466	17,044
	2002	29,174	19,467	43,746	42,811	32,713	31,597

1/ 12 hour traffic in P.C.U.

Traffic volume of two lane road includes that of both direction.

2) Evaluation of Networks

The influence of this project reaches not only the above-mentioned points but also the entire network. This effect is shown in Appendix Figure VIII-12 in terms of the congestion ratio of each link of every alternative network. As stated earlier, congestion ratio of 1.0 means that any further congestion results in rapid decline in the speed of vehicle. At the congestion ratio of 1.3, the speed drops to 5 km/hour and traffic snarl begins. So, the following criteria have been assumed in evaluation of each alternative network:

(1) Links in which congestion ratio is 1.0 or higher in off-peak hours cannot perform normal functions.

(2) Links in which congestion ratio is 1.3 or higher in peak hours cannot perform normal functions

The number of links which cannot perform normal functions and, therefore, constitute a traffic bottleneck, for each alternative network is shown in Table VIII-3-2.

Table VIII-3-2 Number of Bottleneck Links on Major Roads by Alternative

Year	Road	Alternative Network Case					
		W.O.	P_A	P_2	P_4	P_2+P_A	P_4+P_A
1982	A_1 Road	2	1	0	0	0	0
	Motorway	0	0	0	0	0	0
	Project Road	-	0	0	0	0	0
1987	A_1 Road	7	7	0	0	0	0
	Motorway	0	0	0	0	0	0
	Project Road	-	0	0	0	0	0
1992	A_1 Road	10	8	5	2	3	2
	Motorway	1	1	0	0	0	0
	Project Road	-	0	4	0	4	0
2002	A_1 Road	18	17	18	16	17	15
	Motorway	6	6	6	3	6	4
	Project Road	-	0	9	0	9	0

According to the above observation, the following can be asserted:

(1) The number of bottlenecks will continue to increase each year in the future, if no additions are made to the existing roads. There is little difference between alternatives, but this is because it has been assumed that the volume of traffic can increase indefinitely at the vehicle speed of 5 km/hour. Actually, congestion ratio at bottlenecks is far greater than those of other Alternative Cases. Therefore, the number of bottlenecks will actually increase in a wider area than shown.

(2) Only by construction of a new G.R.N.W. bridge instead of the existing G.R.N.W. Bridge, under Case P_A, congestion on A_1 Road will be little improved.

(3) The plan to connect Belle Village-S. Hill-Beau Bassin by a 2-lane road under Case P_2 is considered sufficient to meet the volume of traffic up to 1987, but bottlenecks will occur not only on A_1 Road but also on the Project Road thenceforth.

(4) The plan to connect Belle Village-S. Hill-Beau Bassin by a dual carriage way road under Case P_4 will be sufficient to meet the traffic volume up to 1992, because in 1992 one of the two bottlenecks will be eliminated by making the S. Hill Junction a separate grade intersection and the other bottleneck will reach congestion ratio of 1.4 in Rose Hill city only in peak hours.

(5) Cases P_2+P_A, and P_4+P_A are same as Cases, P_2 and P_4, respectively, except that Case P_A has been added in each case, and contribute little to the mitigation of traffic congestion as a whole.

It is concluded that the satisfactory mitigation of traffic congestion between Port Louis - Beau Bassin which is one of the main objectives of the project cannot be expected, regardless of whether the measure referred to in paragraph 2 is taken or G.R.N.W. Bridge, one of the worst bottleneck on the A_1 Road, is replaced.

(6) All of the Cases will be short of meeting the volume of traffic in year 2002, when a number of bottlenecks will have occurred; Bottlenecks will occur on all of Al Road, Motorway, and the Project Road except under Cases P4 and P_4+P_A.

(7) Bottlenecks in 2002 under Cases P_4 and P_4+P_A will occur on Motorway between Motorway Junction and Cassis Flyover (congestion ratio being 1.5 in peak hours) and the most of remaining bottlenecks will occur on Al Road and adjacent roads in the areas from Beau Bassin through Rose Hill to St. Jean Roundabout.

(8) In view of the above, at least the following steps will have to be taken in order to accomplish this project while avoiding the occurrence of bottlenecks:

① Completion of Case P_2 by 1987 (around 1985), and
② Completion of Case P_4 by 1992 (around 1990).

CHAPTER IX PRELIMINARY DESIGN

CHAPTER IX PRELIMINARY DESIGN

1. The Preliminary Design of Road

In preparing the preliminary design, the following considerations were taken into account:

- Design standards in force in Mauritius.
- Traffic regulations and drivers' traffic habits in the country.
- Use of construction materials and equipment which are readily available locally at reasonable prices.
- Roads will be designed on the basis of the standards applicable to the existing motorways.

1.1 Outline

This Chapter describes the preliminary design for the estimation of the construction costs, one of the key elements for economic evaluation of the Project. The chapter deals with roads and bridges separately in view of determining the possibility of using the existing railway bridges.

1.2 Design Standards

The design standards adopted for the Southern Entrance Road, Access Road to the New Airport and similar projects in Mauritius were evaluated in order to select the design criteria for the present road project. As a result, it was concluded that these design criteria conform substantially to the British standards for roads in urban areas and road layouts in rural areas. However, the application of the British standards to the present project will obviously present some difficulties because of the particular topographical features of Mauritius. The design criteria applicable to the project were analyzed in the light of the following standards:

(1) Roads in Urban Areas (Ministry of Transport, Scottish Development Department, the Welsh Office).

(2) Layout of Roads in Rural Areas (Ministry of Transport, Scottish Development Department, the Welsh Office).

(3) A policy on Geometric Design of Rural Highways (AASHO).

(4) A policy on Design of Urban Highways and Arterial Streets (AASHO).

(5) Road Structure Ordinance (Government of Japan).

The principal design standards were established by consultations with Ministry of Works engineers.

Outlined below are the circumstances in which some of the standards were decided.

(a) Minimum curve radius

It is a primary condition to use the existing railway bed for the road construction. If the railway bed is to be used as far as possible the minimum curve radius will be Rmin. = 500 m.

(b) Maximum longitudinal grade

The standard longitudinal grade refers to a grade which permits a heavy vehicle with 10 P.S/ton to go up an ascent at nearly half the design speed. The topographical features of this island country make it difficult to provide such a grade. For this reason, the steepest longitudinal grade of 5% has been proposed as a standard grade for the project. For some road sections where the reference grade of 5% cannot be adopted for topographical reasons, it has been proposed to provide a steeper grade. Construction of ascending lanes is proposed for the steeper section.

(c) Design speed

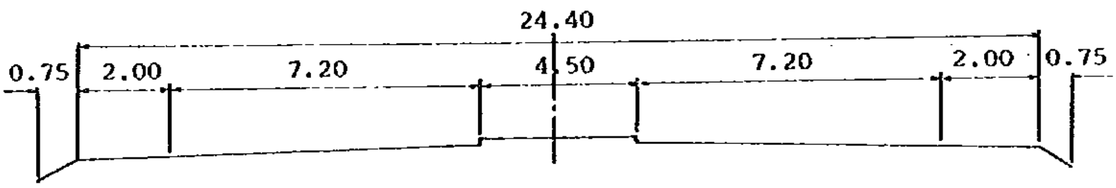
The design speed of 100 km/hr is to be adopted in compliance with the requirements of the competent Mauritian authorities. However, a lower design speed may be considered for specified areas, such as the neighborhood of roundabouts and the interchange at Coromandel.

The design conditions adopted for the project are listed below:

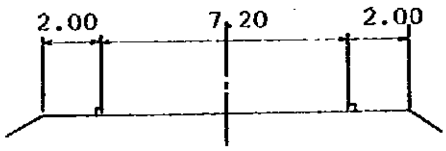
	Main Road	Access Road
Design speed	100 km/hr	40 km/hr
Road Reserve (from edge to edge)	15 m	
Width of carriageway	7.2 m	7.2 m
Width of shoulder	2.0 m	2.0 m
Sight distance	200 m	
Minimum curve radius	500 m	
Maximum longitudinal grade	5%	4%
Motorway cross slope	2.5%	2.5%
Vertical clearance	H = 5.5 m	H = 5.5 m
Design load for Bridges	BS - 153	
Width of carriageway on Bridge	7.2 m	
Width of Sidewalks on Bridge	0.6 - 1.35 m	

The composition of the motorway and pavement widths is illustrated below. The standard cross sections are contained in the drawing set.

(1) Main Road



(2) Beau Bassin Access Road



1.3 Location of Routes

The route locations and the geometrical structures were determined in respect of the selected alternative plans.

The following factors were considered in selection of the route locations. The results of technical analyses based on field investigations were also reflected in evaluation of the route locations.

(1) Topographical features

(2) Geology

(3) Meteorological and hydrological conditions

(4) Compatibility with land use plans

(5) Need for massive structures

(6) Environmental conservation

(7) Geometrical structures

(8) Relative ease of execution

A 1/2,500 scale topographical map was consulted in determining the route locations. The selected route locations are shown in the relevant drawings contained in the drawing set.

The route conditions in each link are outlined below.

Link B

Link B will connect the proposed motorway with S. Hill on A_1 Road. It will branch off from the motorway near Belle Village (by St. Louis Generating Power Station) and continue to S. Hill over the disused railway. This link will use the disused railway bridges spanning the St. Louis River and the Grand River North West (the abutment and the pier of the bridge will be repaired for reuse). The railway bridges have enough width to provide for a 2-lane motorway bridge. However, another bridge will have to be constructed to provide an additional two lanes in future years.

The controlling factors in the choice of the site for a new bridge included the following:

(1) The site should be at the branching-off point of the motorway.

(2) Taking into account an extension in the direction of Beau Bassin, the additional two lanes will preferably pass S. Hill in parallel with the two lanes across the disused railway bridge.

In that section of the new motorway passing S Hill, vertical clearance must be secured in relation to the level and the longitudinal grade of A_1 Road. Taking this into consideration the proposed motorway should run immediately south of the crossing with the disused railway bed (nearer to Beau Bassin). The distance between S.Hill and the left bank of the Grand River North West is only about 200 m, and it is not desirable from the driver's point of view to provide a horizontal alignment for this short section. Therefore, it is recommended that the new motorway run immediately upstream of the railway bridge across the Grand River North West. Technical analysis of a pier construction close to the existing railway bridge showed that the distance between the center lines of the proposed two lanes should be 18 m or more (See XI-2, Preliminary Design of Bridge). The distance between the center lines of the additonal two lanes would be 18 m at the pier nearer to the left bank of the Grand River North West.

In that section of the additional 2-lane bridge passing under A_1 Road, the bridge would be limited in length and have a width as close to the standard width as possible. In this case, the new bridge would be somewhat similar to the disused railway bridge and the distance between the center lines of the additional two lanes on the right bank of the Grand River North West would be about 20 m. For these reasons, the horizontal alignment on the side of Port Louis would shift slightly to the south of the disused railway, and the distance between the center lines of the lanes will be about 30 m in the vicinity of the bridge across the St. Louis River.

As to the alignment of Link B, the disused railway bridge will be used for the lanes to Port Louis and the additional lanes to Beau Bassin will be newly built immediately south of it. The lanes to Port Louis will be designed with a curve of R=1,000 m, and the lane to Beau Bassin, with a compound curve of R=1,000 m and R=1,300 m.

The longitudinal alignment starts with a line tangential to the level of the motorways, running substantially along the railway bed level. The longitudinal alignment of the disused railway between the St. Louis River and the Grand River North West is almost horizontal. The proposed longitudinal alignment for this section is level and the alignment of the section is the railway bed level plus about 50 cm to allow for pavement thickness.

Link B crosses the motorway at its end nearer to Port Louis and crosses A_1 Road at its end nearer to Beau Bassin.

The roundabout type will be adopted for the crossing with the motorway, and when the motorway provision is made for a grade-separated crossing between the roundabout and the link connecting the Port Louis Ring Road when completed, with the proposed motorway.

Since the disused railway bed and A_1 Road form a grade-separated crossing, it is planned that the new motorway will pass under the route. A1 Road and the new motorway will be connected by a rampway, as a rule. However,

A_1 Road and A_3 Road cross each other on separate grades, and the anticipated traffic over the proposed motorway, coupled with the existing traffic, will produce a very complex traffic flow. For this reason, a rampway correcting the proposed motorway will be only for the purpose of coping with traffic flow between Port Louis and Beau Bassin.

Link C

This link starts at S. Hill on A_1 Road by using the disused railway and shortcuts the disused railway from the south of its crossing with Richelieu Approach Road until it crosses the disused railway again on the West of the Barkly CHA Housing Estate in the Beau Bassin area.

On the west of the Coromandel industrial zone, the link will use the disused railway bed almost completely (curve radius R=500) until it will cross the Richelieu Approach Road (Richelieu Roundabout). Near this roundabout, there are two controlling points, the one is the Ministry of Agriculture and Natural Resources building located on the lefthand side, and the other is "water", on the righthand side of the proposed motorway. The motorway will have to pass between these two points without affecting them. The link will then bypass a livestock farm located some 2.2 km away from S. Hill, through the west of the farm. Thus, the link will deviate westward from the railway bed at about 1.7 km from the Hill and cross it at some 2.2 km distance from the Hill where the railway bed curves sharply to the west. Then, after shortcutting the curve of the railway bed, Link C will meet the Petite Riviere Approach Road at about 3.2 km from S. Hill (Petite Malabar Roundabout). In this section, housing location (located 3.1 km away from S. Hill) is a controlling point which will be by-passed in the east. In the eastern part of this controlling point, these spread privateowned lands about 350 m wide (planters with more than 100 arpents). Care has been taken to avoid trespassing on these private lands.

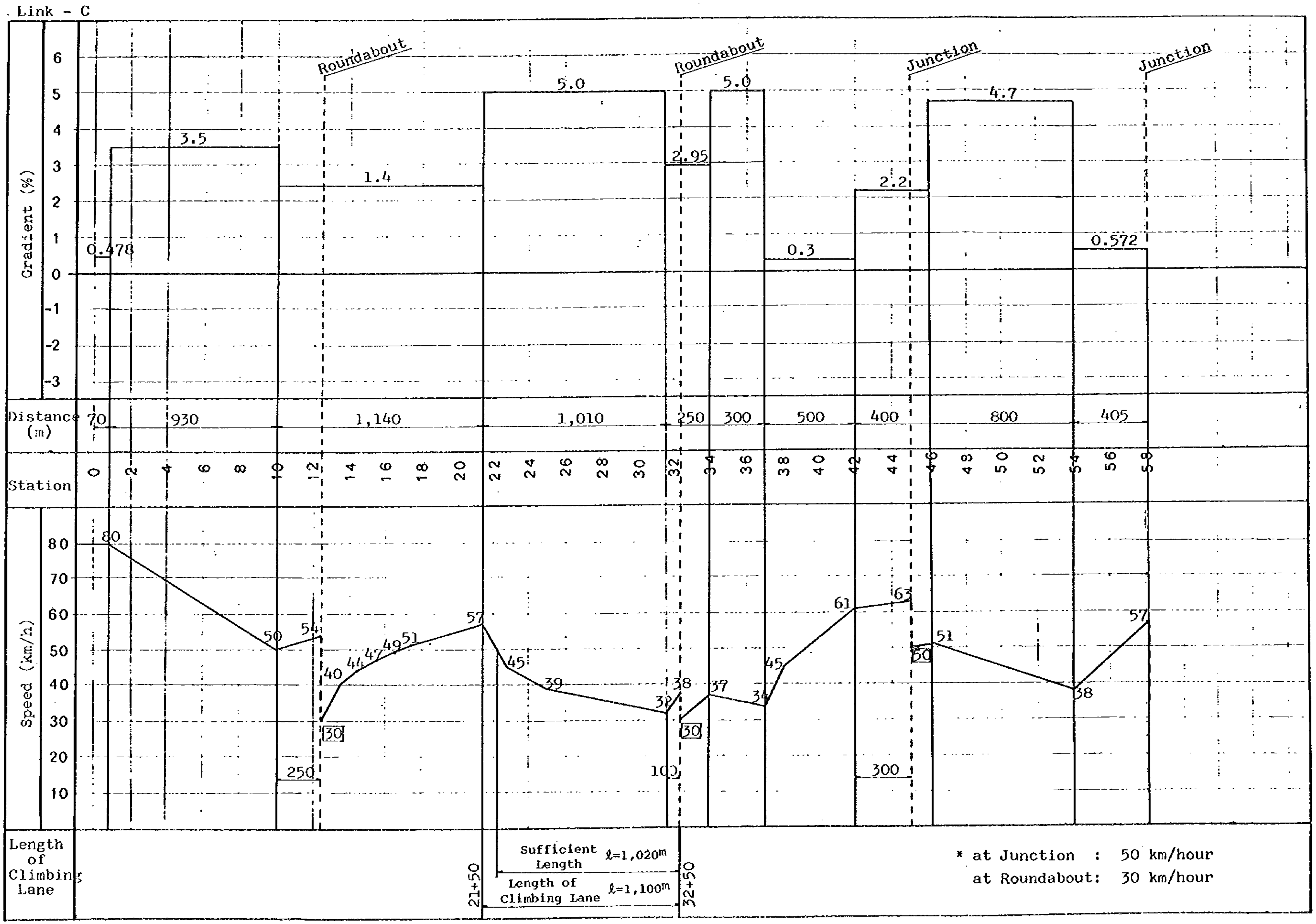
Thence Link C will run in north-south direction and meet the railway bed at its end. In this section, the motorway will bypass "water" (located about 3.7 km from S. Hill), in its west. The link's cross with the railway bed will take place at about 300 m west of the Barkly CHA Housing Estates. The crossing, which will form a T crossing in the future, will be a channelization crossing (Barkly Junction).

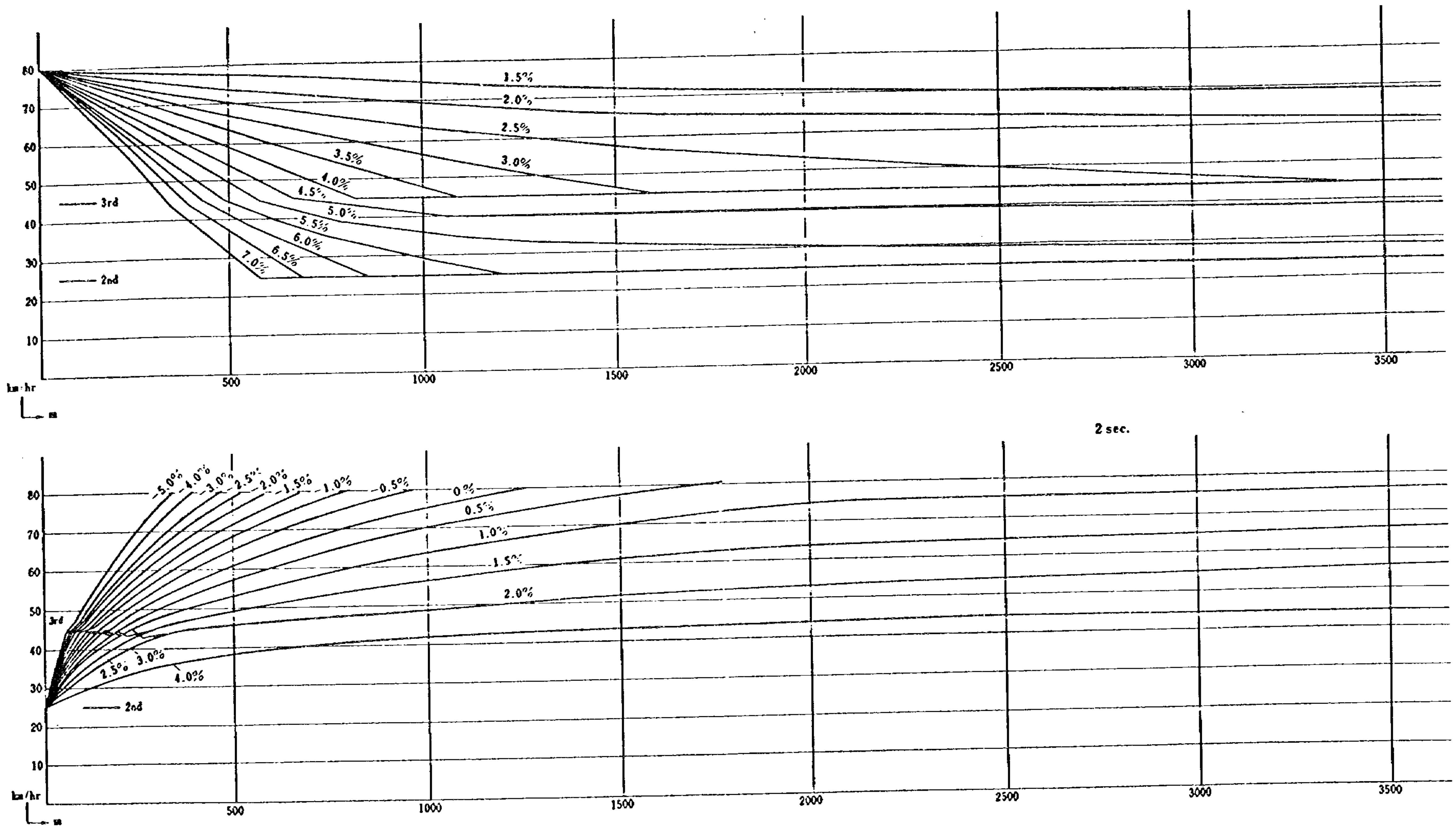
Link C will have a 3.5% uphill grade as far as the Richelieu Roundabout, since in S. Hill it will pass under A_1 Road and continue into the tableland. Because the section between point 2,1 km from the hill and a point 0.5 km beyond the Petite Malabar Roundabout has a rapid ascent, the link will have a 5% longitudinal grade. Climbing lanes are considered necessary for this section, and this will be analyzed later. Since the 5% grade in the vicinity of the Petite Malabar Roundabout is too steep, it has been proposed to build a section with a grade below 3% near the roundabout.

The section between a point 0.5 km beyond the Petite Malabar Roundabout and Barkly Junction is less steep with the grade of about 0.3%. In the vicinity of the Barkly Junction, it is necessary to raise the design road level so as to keep the cutting heights at minimum for the access road to Beau Bassin (Link E) and Link F to the Banbous Branch Road. It is also necessary to avoid producing bad effects on the crossings and to prevent the filling height from being excessive. Consequently, a 2.2% longitudinal grade has been adopted.

Given below is a chart showing the results of the analysis of the required length of the climbing lane.

Fig. IX-1-1 Estimated Figure for Climbing Lane Length





Speed - Distance Curves of a Typical Heavy Truck
Operating on Various Grades

Power : 10PS/ton
Efficiency : Top 90% 3rd 2nd 85%, low 80%
Maximum Speed : 80km/h (Top)
Road Surface Resistance: $r = 10+10i+0.0016\ V^2$
Transmission gear ratio: 1 + 1,775: 3,200: 5,333

Link D

This link is an alternative to Link C. Like the latter, the link will originate at "S" Hill and terminate at Barkly Junction.

Between "S" Hill and the Richelieu Roundabout, Link D takes the same route as Link C but thenceforth it will run further east than Link C. Running eastward from the railway bed at the Richelieu Roundabout, it will by-pass the livestock farm (controlling point on Link C) in the east. It will skirt along the east end of the Government-owned lands (Crown Lands) with a width of about 300 m which are located some 2.5 km away from "S" Hill. Some 0.3 km beyond, the link will run toward south across a strip between the western end of the Petite Malabar Mountain and the eastern end of the privately-owned estates (controlling point on Link C). In this area, Link D will be situated about 500 m further east than Link C.

After crossing the Chebel Branch Road (Petite Malabar Roundabout) some 3.8 km away from "S" Hill, Link D will shift slightly to the west and meet the railway bed, some 100 m further east than Link C (Barkly Junction).

In this section, houses are located some 3.6 km away from S. Hill. The controlling points include the following:

(1) It is planned that Link D will run to the east of the houses.

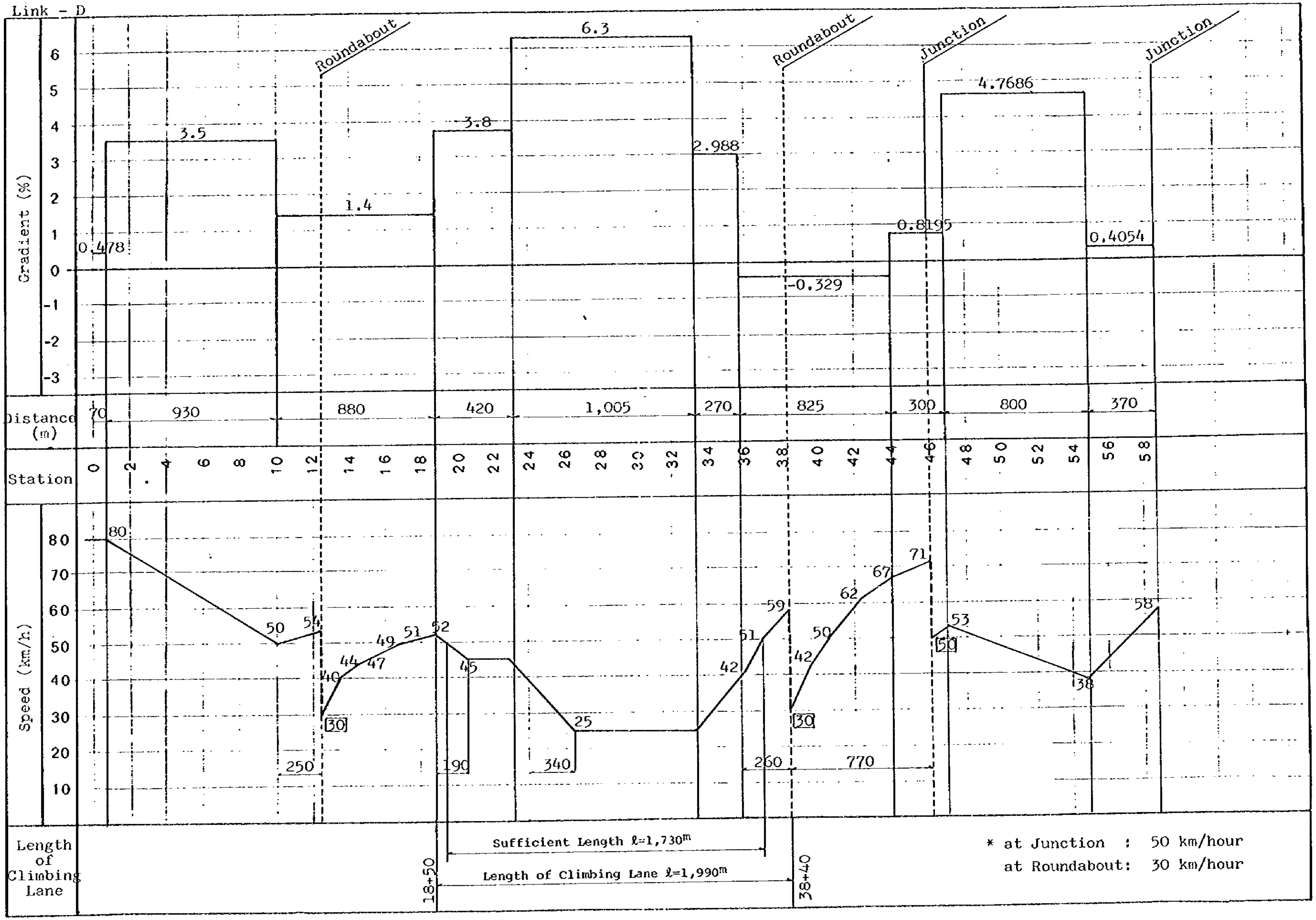
(2) The Barkly Junction is located about 200 m distant from the CHA Housing Estate.

As to the longitudinal alignment, the link will have a 3.5% uphill grade until the Richelieu Roundabout as in the case of Link C. After passing the roundabout, Link D will continue with a 1.4% grade as in Link C until a point 1.9 km away from S. Hill. Beyond that point, Link D will continue some 400 m with a 3.8% grade and the grade will increase to 6.3% in the section extending from the west of the Petite Malabar Mountain to the vicinity of the Petite Malabar Roundabout. The sharp grade of the link is due to the fact that the area has a steep gradient.

Beyond the Petite Malabar Roundabout, Link D will descend with a downhill grade of 0.329% until it reaches the Barkly Junction.

Possibility of constructing climbing lanes in the steeper section of Link D has been studied. The results of this study are shown in Fig. IX-1-2.

Fig. IX-1-3 Estimated Figure for Climbing Lane Length



Link E

This link will be an access connecting the Barkly Junction with the heart of the urban area of Beau Bassin.

It will use the railway bed throughout. Since the railway bed constituting the link is lined with dwelling houses, the maximum possible road structure will be a 2-lane roadway with a sidewalk.

Longitudinally, it is planned that the railway bed level will be raised to about 50 cm above the existing ground level to allow for a pavement thickness. The link will have an uphill grade of nearly 3.5%.

Link F

This link will connect the Barkly Junction and the Bambous Branch Road.

It will run along the western end of the urban area of Beau Bassin. Taking into consideration its future extension to the south of the city, it is planned that the link will run 300 to 350 m west of the CHA Housing Estate and the Central Prison. Because of the Grand Malabar Mountain located to the west of the link, it will skirt along the eastern foot of the mountain.

Longitudinally, the link will ascend for about 900 m beyond the Barkly Junction with a grade of 4.7%, which will then decrease to some 0.5% over the next section of about 400 m in length leading to the Bambous Branch Road.

The junction with the Bambous Branch Road (Grand Malabar Junction) where Link E will terminate will be a T crossing in the immediate future. If the proposed motorway is extended further south in the more distant future, it will form a crossing with the Bambous Branch Road. Taking this into consideration, the junction will have an adequate area to meet future requirements.

1.4 Design of Pavement

The pavement structural design was reviewed in reference to other alternative designing methods. For the designing method 'AASHTO Interim Guide for Design of Pavement Structures 1972' and 'Asphalt Pavement Manual 1975 of the Japan Road Association' were adopted.

Design Conditions

Traffic volume:	1/2 × (15,678+35,422) = 25,655 vehicles per day
Mixed rate of large-size vehicles:	10 %
Heavy-vehicle traffic volume:	2,565 vehicles per day in one-way traffic
Equivalent single axle load:	8.2 t (18 kips)

Examination by AASHTO Method

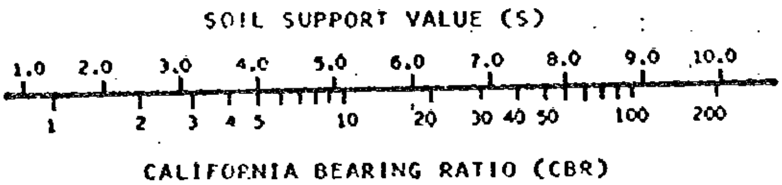
AASHTO Method is a designing method using various relevant factors such as the service index of surface (Pt), the soil support value (S), the structural number (SN), the structural layer coefficients (An) and the regional factor (R).

Service index of surface	Pt = 2.5
Soil support value	S = 4.9 (CBR = 9.0)

(From the relative chart of soil support value California Bearing Ratio - CBR)

Regional factor	R = 0.5
Structural number	SN = 4.4

Structural Layer	Thickness	Structural	Number SN
Surface course (2-layer)	10 cm	0.44	1.76
Base course (Bituminous-treated)	8 cm	0.34	1.09
Subbase course	35 cm	0.11	1.54
	53 cm		4.4



Correlation Between Soil Support Value and CBR, Utah Department of Highways

Structural Layer Coefficients Proposed by AASHO Committee on Design, October 12, 1961

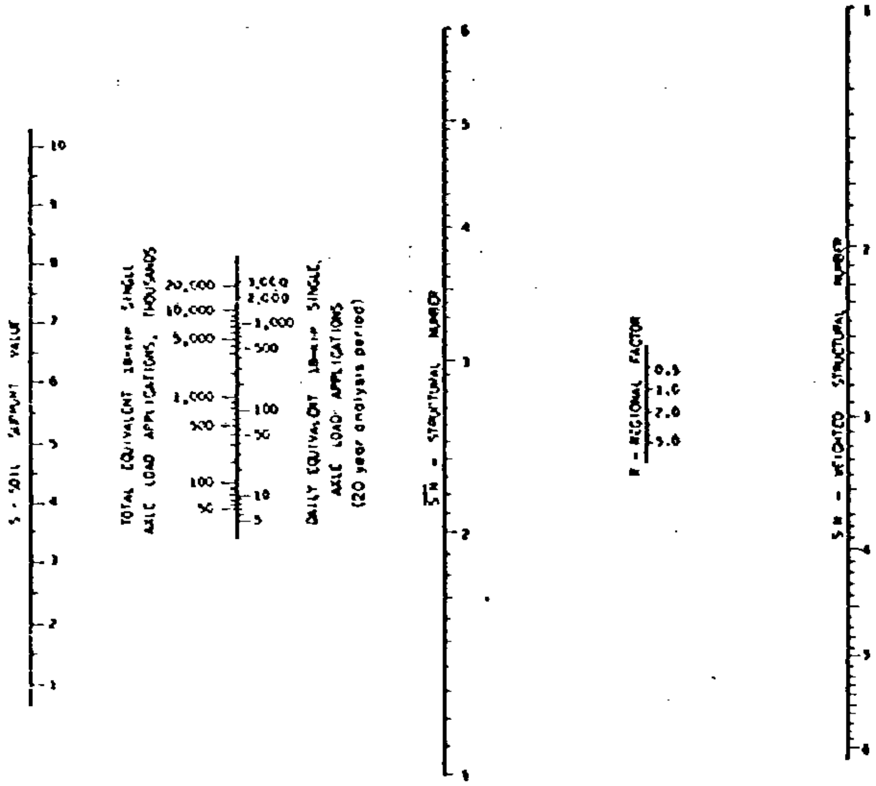
Pavement Component	Coefficient[4]
Surface Course	
Roadmix (low stability)	0.20
Plantmix (high stability)	0.44*
Sand Asphalt	0.40
Base Course	
Sandy Gravel	0.07[3]
Crushed Stone	0.14*
Cement-Treated (no soil-cement)	
Compressive strength @ 7 days	
650 psi or more[1] (4.48MPa)	0.23[3]
400 to 650 psi (2.76 to 4.48MPa)	0.20
400 psi or less (2.76MPa)	0.15
Bituminous-Treated	
Coarse-Graded	0.34[3]
Sand Asphalt	0.30
Lime-Treated	0.15-0.30
Subbase Course	
Sandy Gravel	0.11*
Sand or Sandy-Clay	0.05-0.10

* Established from AASHO Road Test Data

[1] Compressive strength at 7 days.

[3] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.

[4] It is expected that each state will study these coefficients and make such changes as experience indicates necessary.



Design Chart for Flexible Pavements, p_t = 2.5

Method from the Asphalt Pavement Manual

The method by use of the Asphalt Pavement Manual of Japan is a designing method using the structural number, on the basis of the design factors such as the heavy vehicle traffic volume per day in one direction and the design CBR for subsoil.

Classification by traffic volume	Heavy vehicle traffic volume (Vehicle/day in one direction)
A	250 and below
B	250 ~ up to 1,000
C	1,000 ~ up to 3,000
D	3,000 and over

Target value for T_A and total thickness

T_A: Required thickness for pavement entirely with heating asphalt compound for surface and base courses.

Table Target Values for TA and Total Thickness

Design CBR	Target values (cm)							
	Traffic A		Traffic B		Traffic C		Traffic D	
	T_A	Total thick-ness	T_A	Total thick-ness	T_A	Total thick-ness	T_A	Total thick-ness
2	21	62	29	74	39	90	51	105
2.5	20	55	27	66	36	79	48	92
3	18	49	25.5	58	34	70	45	82
3.5	17.5	45	24.5	54	32.5	65	43.5	76
4	17	41	23.5	50	31	61	42	70
5	15.5	35	22	43	29.5	54	39	60
6	14.5	30	21	38	28	48	36	53
8	13.5	27	19	33	26	40	33	47
10 and over	12.5	23	17.5	29	24	34	31	40

Design CBR = 9

Traffic volume classification: C

T_A = 25 (cm)

Total thickness of minimum pavement 37 cm

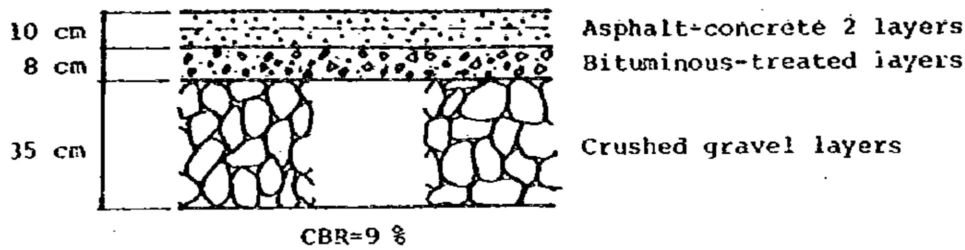
Paved thickness equivalent coefficient is shown in the following Table:

Table Equivalent Coefficient for Calculation of T_A

Layer	Method & material	Condition	Equivalent coefficient
Surface & base courses	Heating asphalt compound for surface and base courses	-	1.00
Base course	Bituminous treatment	Stability 350 kg -	0.80
		Stability 250 to 350 kg	0.65
	Cement treatment	Unconfined compression Strength 30 kg/cm^2	0.55
	Grain adjustment of soil	Corrected CBR 80 and over	0.35
	Infiltration type	-	0.55
	Macadam		0.35
Subbase course	Crushed gravel, gravel and sand	Corrected CBR 30 and over	0.25
		Corrected CBR 20 to 30	0.20

Structural layer	Thickness	Equivalent coefficient	T_A
Surface course (Asphalt-concrete 2 layers)	10 cm	1.0	10 cm
Base course (Bituminous-treated)	8 cm	0.8	6.4 cm
Subbase course (Crushed gravel)	35 cm	0.25	8.7 cm
	53 cm		25.1 cm > 25 cm

The paved thickness was determined from the result of the two designing methods as mentioned above.



1.5 Design of Drainage

(1) Drainage ditch

Drainage ditches are of three different kinds as follows:

a) Side ditch without timbering

To be used for the section where the grade is in no danger of being scoured by water accounts for less than 3 % of the total.

b) Side ditch with concrete timbering

To be used for the section where the grade is in danger of being scoured, accounting for 3 % or more.

c) Median strip ditch

To provide the U-type concrete-timbered side ditch at median strip for the superelevation section.

(2) Culvert

Culverts to be used for the crossing of irrigation channels and drains are of two different types as follows:

a) Pipe culvert: Concrete pipe with a diameter of 1.0 m at minimum for convenience of cleaning.

b) Box culvert: Concrete box culvert to be used as the crossing culvert for irrigation channels.

1.6 Intersection Design

This Section refers to the Intersection where the proposed road will cross the main road. The content for review includes the location, configulation and traffic capacity of the Intersection. The geometric design plans prepared therefrom are shown in the Appendix compliation of drawings.

1.6.1 Location and type of Intersection

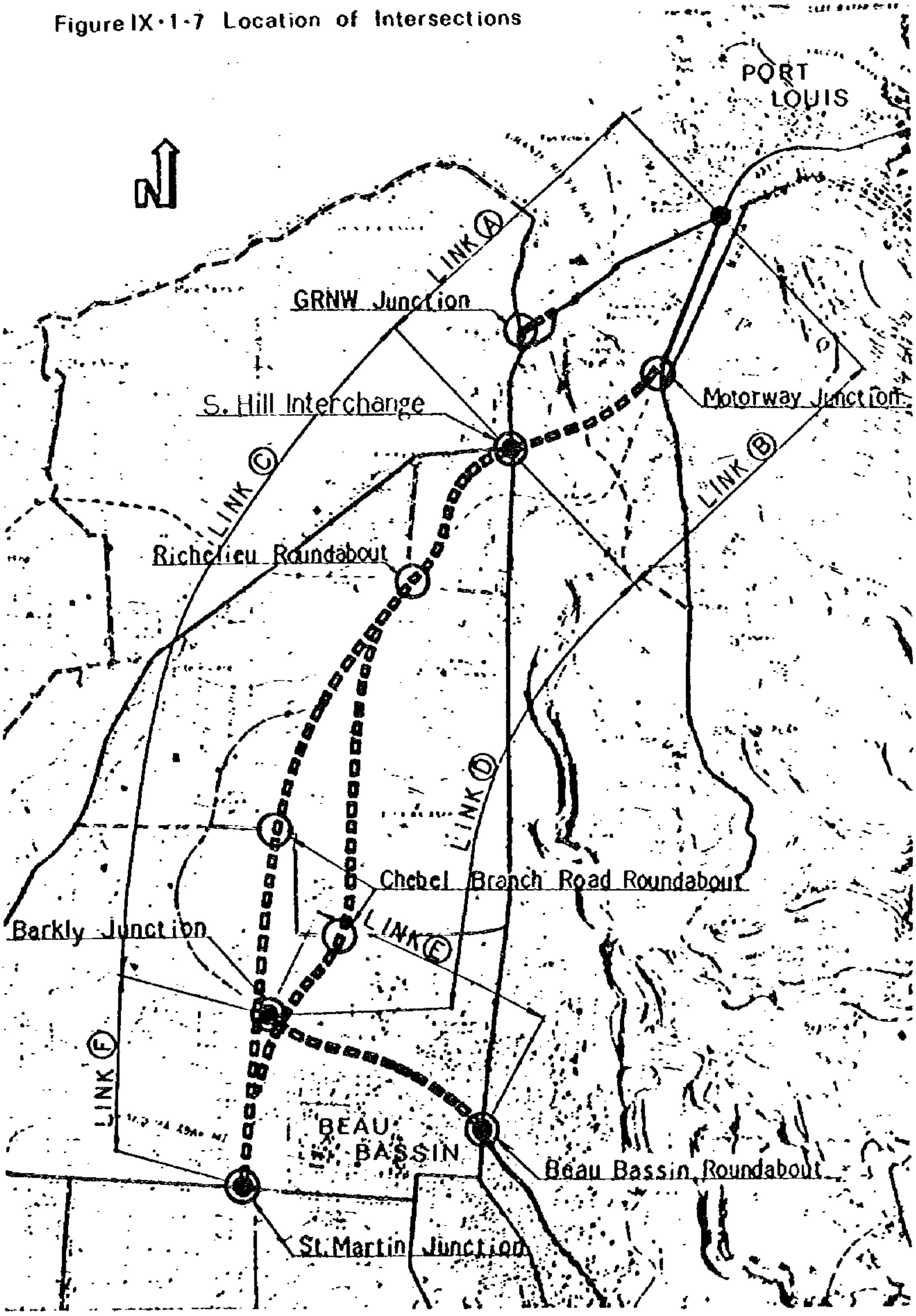
Main Intersections and their locations are as shown in Fig. IX-1-7. Each of them is named from their location or the name of the road they will cross, as shown in Fig. IX-1-7. They are as listed hereunder.

G.R.N.W. Junction:	Pointe Aux Sables & Petite Verger Road
Motorway Junction:	Motorway
S. Hill Junction:	Route A_1 (Royal Road)
Richeliu Roundabout:	Richelieu Approach Road
Chebel Branch Road Roundabout:	Petite Riviere Approach Road or Chebel Branch Road
Barkly Junction:	Access Road to Beau Bassin Roundabout
St. Martin Junction:	Lower Plaines Wilhems Cemetery Road
Beau Bassin Roundabout:	Route A_1 (Royal Road)

As a rule, each Intersection will be At grade Intersection, and roundabout type system. To meet traffic demand or topographic conditions, grade separated Intersection may be adopted. In case of the three-leg Intersection, it will be a grade Intersection provided with channelization. Type and traffic control policy of each Intersection is explained hereunder:

Name of Intersection	Traffic Control Policy
G.R.N.W. Intersection	3-leg Intersection with channelization
Motorway Intersection	Designed for the present roundabout system but as a very large traffic volume is expected because it crosses the Motorway as important as the proposed highway, there remains the possibility for future improvement into a grade separated crossing in anticipation of its possible connection with Port Louis Ring Road.

Figure IX·1·7 Location of Intersections



S. Hill Interchange:	The proposed road will use the disused railway track, which originally crossed A1 Road with grade separated crossing. This will be utilized for the crossing of the new road. Provisions for right and left running traffic will be provided only in the direction of the main traffic flow to the Motorway Junction - S. Hill Interchange - Beau Bassin. This service system will be provided by rampway.
Richelieu Roundabout:	Grade crossing by roundabout
Chebel Branch Road Roundabout:	At grade crossing by roundabout
Barkly Junction:	3-leg Intersection with channelization
St. Martin Junction:	3-leg Intersection with channelization for the time being. However, conversion into a 4-lane crossing roundabout is anticipated in the future when the ring road is be extended to the south of Beau Bassin. The present size of the roundabout should, therefore, be such that it can be included in the future expansion.
Beau Bassin Roundabout:	At grade Intersection with the roundabout. When this is completed, it will take such a form that the new access road will have its entrance at the existing roundabout. This roundabout is situated in the city area with banks and other commercial buildings in the neighbourhood grouped close together. It is, therefore, very difficult to improve the present situation significantly. However, at the proposed junction of the Beau Bassin access road and the small street running parallel on the south side of the Beau Bassin access road, there will be no increase or decrease in the total number of the existing links. As a result, the roundabout will need to be extended in length in a north-south direction. On the other hand, each traffic weaving section will be improved to provide sufficient space for two lanes by utilizing some of the space in the front yard of the post office and the Sacre-Coeur Church.

1.6.2 Traffic volume forecast at the Intersections

Traffic volume in various cases is forecasted in Chapter 8 dealing with traffic volume forecast. The estimated traffic volume varies depending upon the planned year of road construction, road length available for service and the number of serviceable lanes. Here, the most probable two cases of typical pattern are selected for studies. In each case, the traffic volume (P.C.U.) on the single way near the intersection and the traffic volume (P.C.U.) classified by different intersections and directions are estimated. The traffic volume is estimated for hourly traffic volume during the morning peak. The two typical alternatives to be considered are summarized below.

. Alternative P_2 · West · 1987

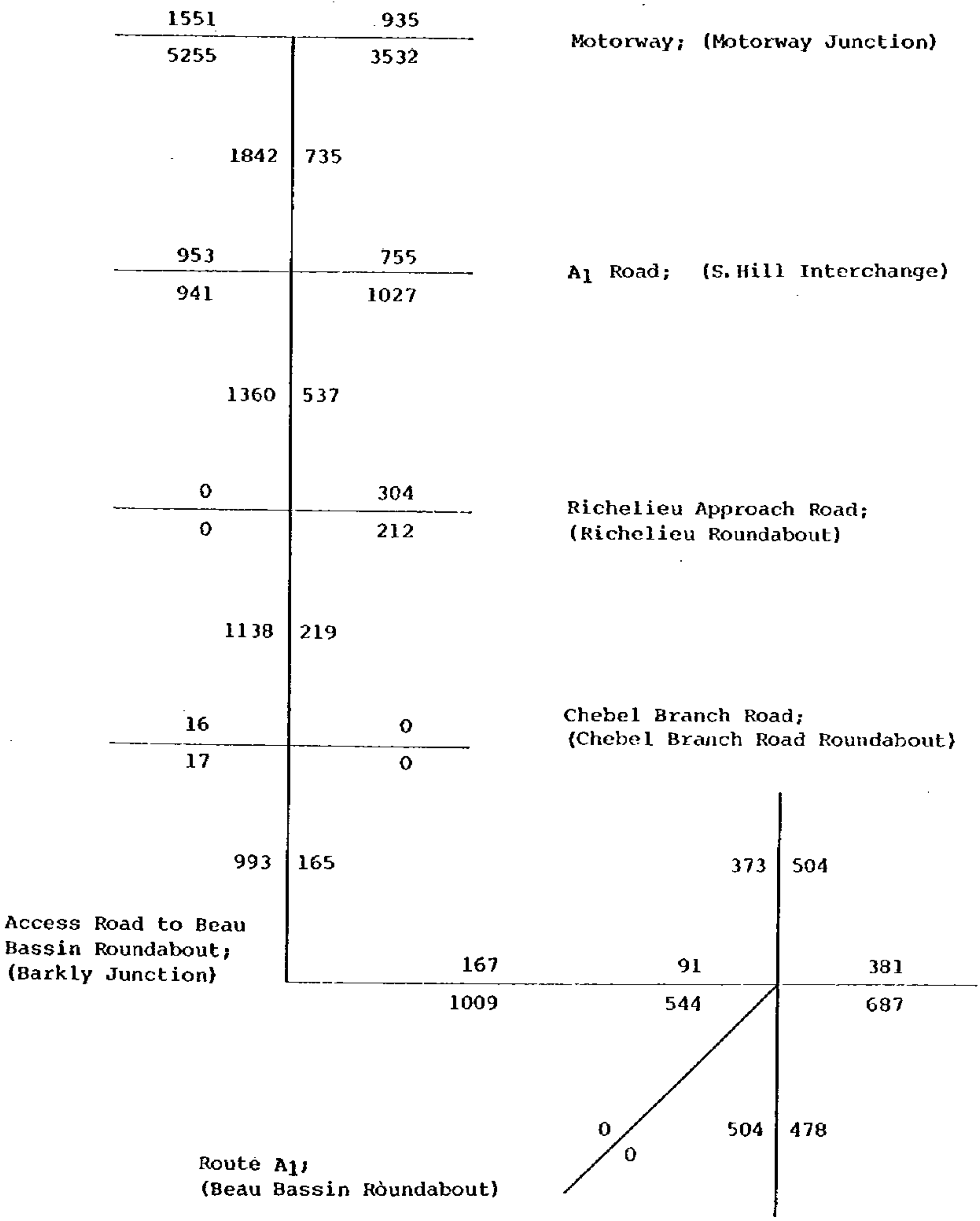
Traffic volume at morning peak time in the case where the West-side plan (Link B + C + E) will have been realized for use with two lanes from Motorway Junction to Beau Bassin Roundabout in five years (1987) after opening of the road to public traffic.

. Alternative P_4 · West · 1992

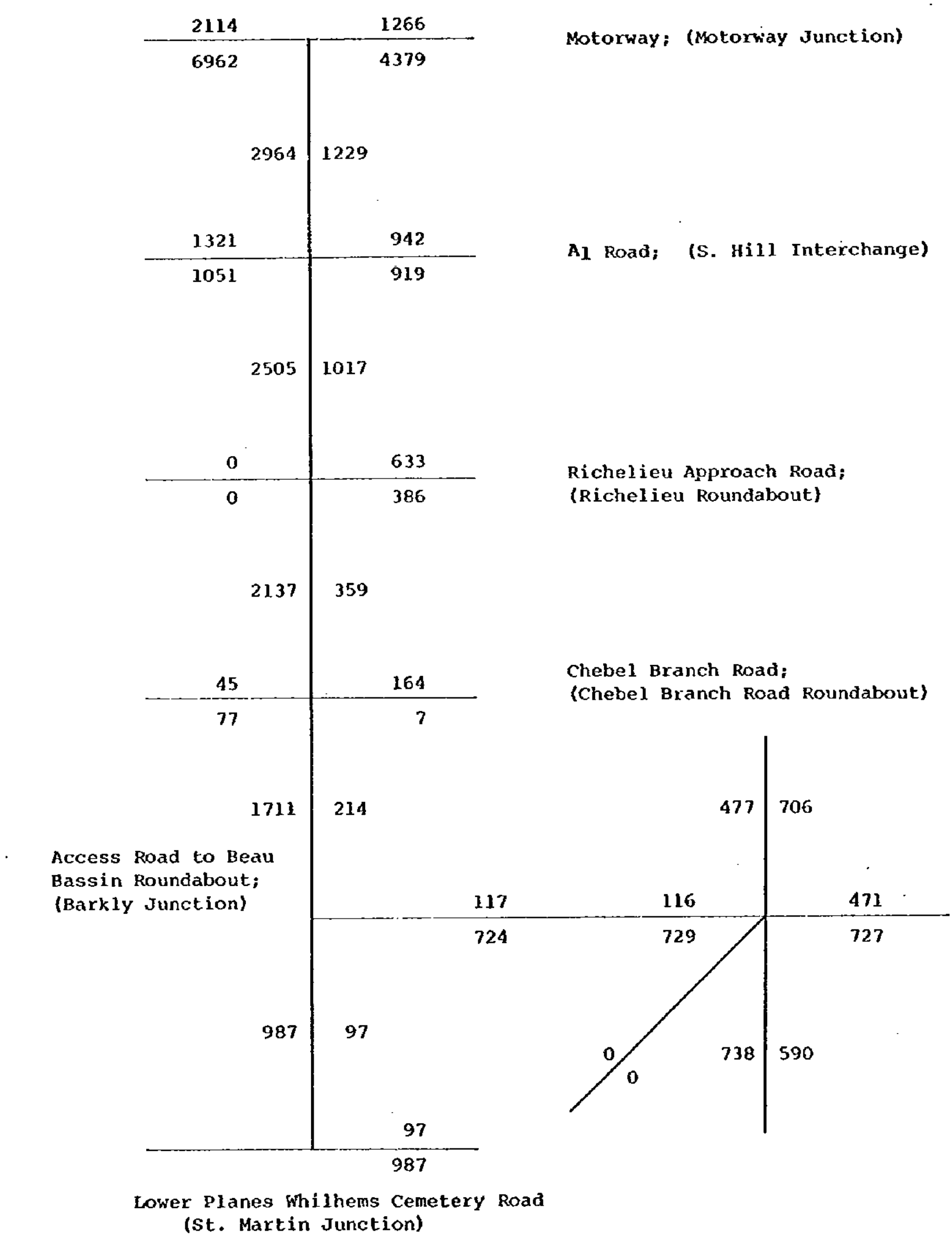
Traffic volume at morning peak time in the case where the West-side plan (Link B + C + E + F) will have been realized for use with four lanes from Motorway Junction to Beau Bassin Roundabout and St. Martin Junction in ten years (1992) after opening of the road to traffic.

For reference only, the estimated traffic volume after renewal of G.R.N.W. Bridge in Route A_1 in addition to the proposed service section by 1992, is shown in Appendix Figure IX-2.

Traffic Flow at Morning-Peak Hour in 1987
(Alternative P_2 West) (P.C.U.)

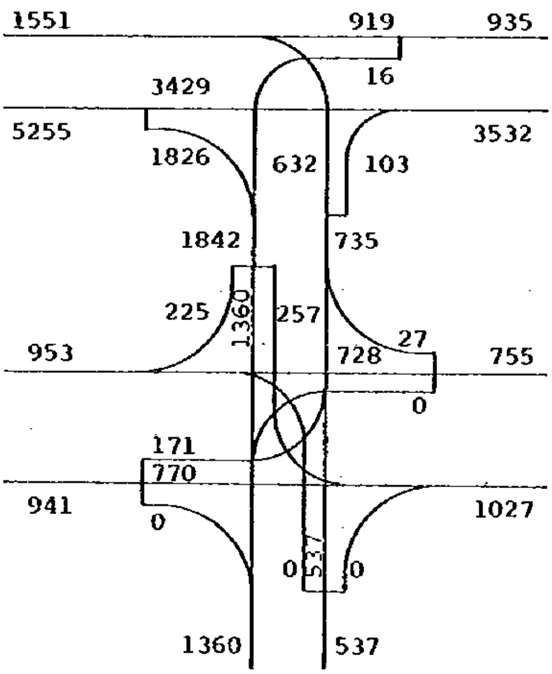


Traffic Flow at Morning-Peak Hour in 1992
(Alternative P_3 West) (P.C.U.)



Junction Traffic Flow at Morning-Peak Hour
(P.C.U.)

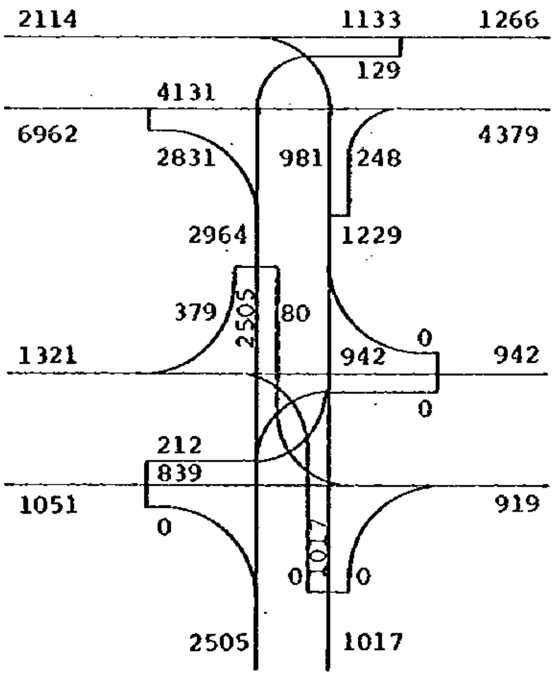
<1987- P_2 West>



Motorway; (Motorway Junction)

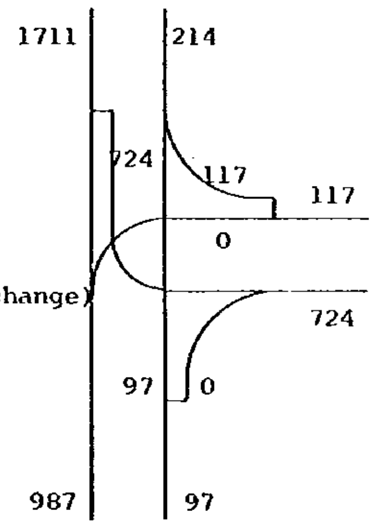
A_1 Road; (S. Hill Interchange)

<1992- P_3 West>



Motorway; (Motorway Junction)

A_1 Road;
(S. Hill Interchange)



(Barkly Junction)

1.6.3 Review of Traffic Capacity at the Junctions

Traffic capacity for such crossing systems as stated in the above can be estimated by various applicable methods. For our purpose, the British method of calculation has been used by reference to the following data:

Data 1. Technical Memorandum H2/75
Roundabout Design

Data 2. Roads in Urban Areas
Ministry of Transport

a) Traffic capacity for give-way intersection

The data provide the possible inflow of traffic volume from the non-priority road into the priority road depending upon the trafficable volume on the first-priority road as well as the specific traffic conditions such as whether the road can offer a wide range of vision or not and whether it is of a 4-lane or a 2-lane system. The estimated inflow of traffic volume into the 4-lane road with a wide range of perspective vision will be as shown below:

Traffic volume on first-priority road	Possible inflow of traffic volume into non-priority road
1,500 vehicles per hour	160 vehicles per hour
1,000 " "	330 " "
600 " "	540 " "

b) Small-sized roundabout

The roundabout with an outside diameter of 20 to 50 m is defined as a 'small roundabout' and is given the capacity as specified below:

Arms	Design flow range total all arms veh/hr (P.C.U.)
3-way	up to 5,500
4-way	" 4,000
5-way	" 3,500

c) Conventional roundabout

The conventionally used roundabout is considered basically to be designed for capacity as specified below. However, it is further necessary to make review of capacity at the weaving section.

Design flow range	Total all arms veh./hr. (P.C.U.)
3-way	5,000 + α
4-way	3,500 + α
5-way	3,000 + α

Capacity at weaving section (Qp):

$$Qp = \frac{86w\ (1+\frac{e}{w})\ (1-\frac{P}{3})}{1+\frac{W}{\ell}}$$

Where, W: Width (feet) of weaving section

e: Mean value of lane width (feet) at inflow part of weaving section

ℓ: Length (feet) of weaving section

P: Rate of weaving traffic volume as against total traffic volume at weaving section (0.4 to 1.0)

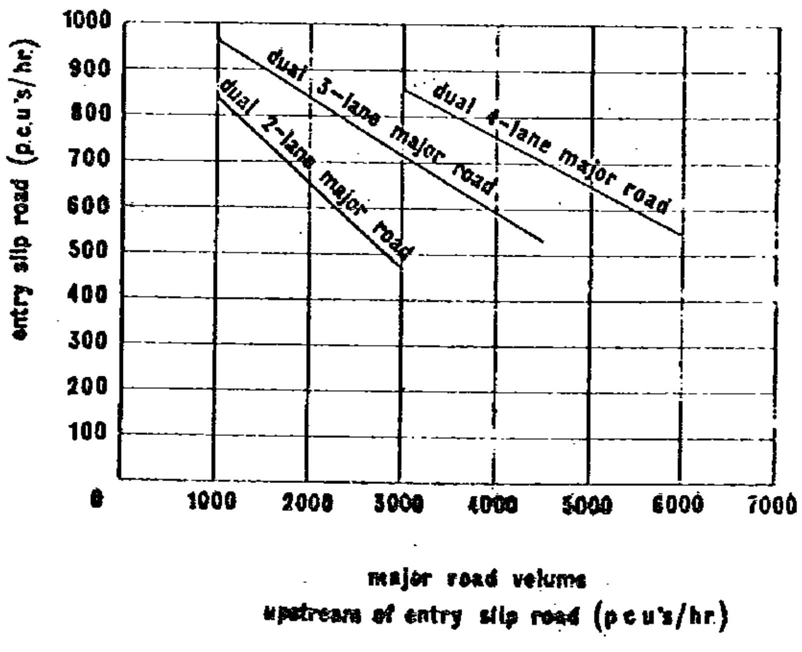
d) Rampway

The traffic capacity for the rampway may be determined from the smallest of either the traffic volume of interrupted flow on the rampway or the traffic capacity at the merging or diverging end of the rampway and the main road or the connected road. The traffic capacity at each part of the rampway is as follows:

- The design traffic capacity of the rampway: 1,200 p.c.u. per lane per hour.

. The traffic capacity of ramp terminals: The traffic capacity at the merging end with rampway and the main road is normally subject to variations depending upon correlations between the traffic volume at the corresponding part of the main road and that of the rampway, which may be obtained from the following Figure IX-1. Now, on the basis of the basic traffic capacity as provided above, the traffic condition at each junction is as follows from the traffic volume as indicated in Section IX-1-6-2.

Figure IX-1 Upstream of Entry Slip Road



Intersection	Congestion (C)	
	1987 (2-lane)	1992 (4-lane)
Motorway J.	C ≒ 1.08	C ≒ 1.37
S. Hill J.	C < 1.0	C < 1.0
Richelieu R.	C < 1.0	C < 1.0
Chebel Branch Road J.	C < 1.0	C < 1.0
Barkly J.	-	C < 1.0
St. Martin J.	-	C < 1.0
Beau Bassin R.	C < 1.0	C < 1.0

Table above shows congestion (C = Traffic flow/capacity) at each junction with reference to the traffic capacity as aforestated. For the Motorway Junction, a separate calculation has been made for its traffic capacity, the result of which is shown in Appendix IX-4.

As noted from the Table, it has been observed that the traffic volume at the Motorway Junction would reach nearly its full capacity by 1987 and 1.37 times as much as its capacity by 1992, which indicates the ceiling limit of traffic control by grade crossing system.

At the Motorway Junction, there is a big problem in the weaving by the traffic flow from Port Louis to S.Hill and the traffic flow from Pailles to Port Louis. By 1992 it will become necessary to convert the present into a grade separate intersection for more effective traffic control.

To further explain what the problem is, it is pointed out that there will still be only one motorway (4-lane) in traffic services,to Port Louis while the future traffic volume may increase largely with the two 4-lane roads leading from the south.

At this site in question, there is a plan emphasising construction of the Port Louis Ring Road to run through the mountain side as the by-pass for the city area of Port Louis. Therefore, the proposed conversion into a grade separate crossing at this junction should be fully reflected by that plan. Basides this, the Ring Road plan must be reviewed at the earliest possible data in anticipation of the future increasing trend of traffic volume, so that the most desirable pattern of traffic control by grade separate crossing can finally be determined. It is for this reason that the geometric design drawings, as a plan for the potential conversion into grade separate crossing, have been attached to the appendix B (drawings), although the plan proposed in this Report does not go far beyond the scope of an intersection at grade (roundabout). As concerns the rest of the intersection, there is little problem about the traffic capacity.

2. Preliminary Design of Bridges

2.1 Introduction

This Section tries to make the final selection of the most suitable plan, through review and evaluation of the comparative preliminary designs of the structures on the routes selected in 'VI Comparison and Selection of Alternatives'.

It first proposes the alternative plans for comparative study, then makes technical analysis on those plans, and finally selects the most economical plan by comparing their construction costs.

2.2 Alternatives by Classification of Structures

The proposed route finally selected in Chapter VI is the disused railway track route, which involves such problems as re-utilization by improvement of the G.R.N.W. old railway bridge and the St. Louis old railway bridge or new installation and the improvement of overbridge crossings of the railway track. For each of them the alternative is assumed. G.R.N.W. bridge is taken up as the chief item for studies on old railway bridges as St. Louis bridge has its own problems to be studied independently.

Table IX-2-1

Bridges	Assumptions	Alternatives
1. Old railway bridge (G.R.N.W bridge)	a. To utilize both superstructure and substructure of the existing bridge Adjustment to vertical alignment profile	Superstructure: Metal (Non-composite girder) Substructure: Rubble concrete Span: 27 m Width: 7.2+0.6 m
	b. New installation for superstructure but the existing structure to be used for substructure Adjustment to vertical alignment profile Comparative study to be made for metal and PC structures of superstructure	Superstructure: Metal (Composite girder) PC: Post-tensioning T girder Substructure: Rubble concrete Span: 27 m Width: 7.2+1.35 m
	c. New installation for both superstructure and substructure Examination of basic span composition	Superstructure: 7-span simply-supported girder PC: Post-tensioning T girder Span: 27 m 5-span simply-supported girder PC: Post-Tensioning T girder Span: 38 m 3-span continuous girder PC: Dywidage box girder Span: 59+73.8+59 m Substructure: Reinforced concrete Width: 7.2+1.35 m
	d. Additional installation separately for 2-lane bridge	By use of compared results between above alternatives, b and c

2. St. Louis Bridge	Additional installation separately for 2 lanes corresponding to the preceding 1. d.	Type: Single span with high abutments 3 spans with low abutments
3. Overbridges pailles S. Hill Coromandell	Preparation of one common plan for standard type	Type: Simply-supported pre-tensioning slab girder

2.3 Major Items for Comparative Studies

The fundamental flow of procedures for comparative studies on the alternatives begins with an establishment of the basic conditions, then the main component structure and their basic dimensions will be determined, and ends up with the cost calculation after examining the bill of quantities and the construction method. This flow is illustrated in Fig. IX-2-1.

The main problems involved in comparative studies of the alternatives are itemized as follows:

(1) In the case of the proceding 1. a, there are problems of how to deal with corrosion damages for stress checking and how to repair to a healthy state in view of the property of the component material.

(2) In the same case as mentioned above, what measures should be proposed for prevention of future scouring?

(3) In the same case as mentioned above, how to judge the essential life span of the existing bridge?

(4) In the case of the preceding 1. b, which is more economical, metal or PC structure, as far as the superstructure is concerned?

(5) In the case of the preceding 1. c, what is the most economical composition of span after all?

(6) In the case of the future addition of the 2-lane substructure, how far is the allowable limit for adjacent construction?

(7) What is the most economical type for maintenance and control?

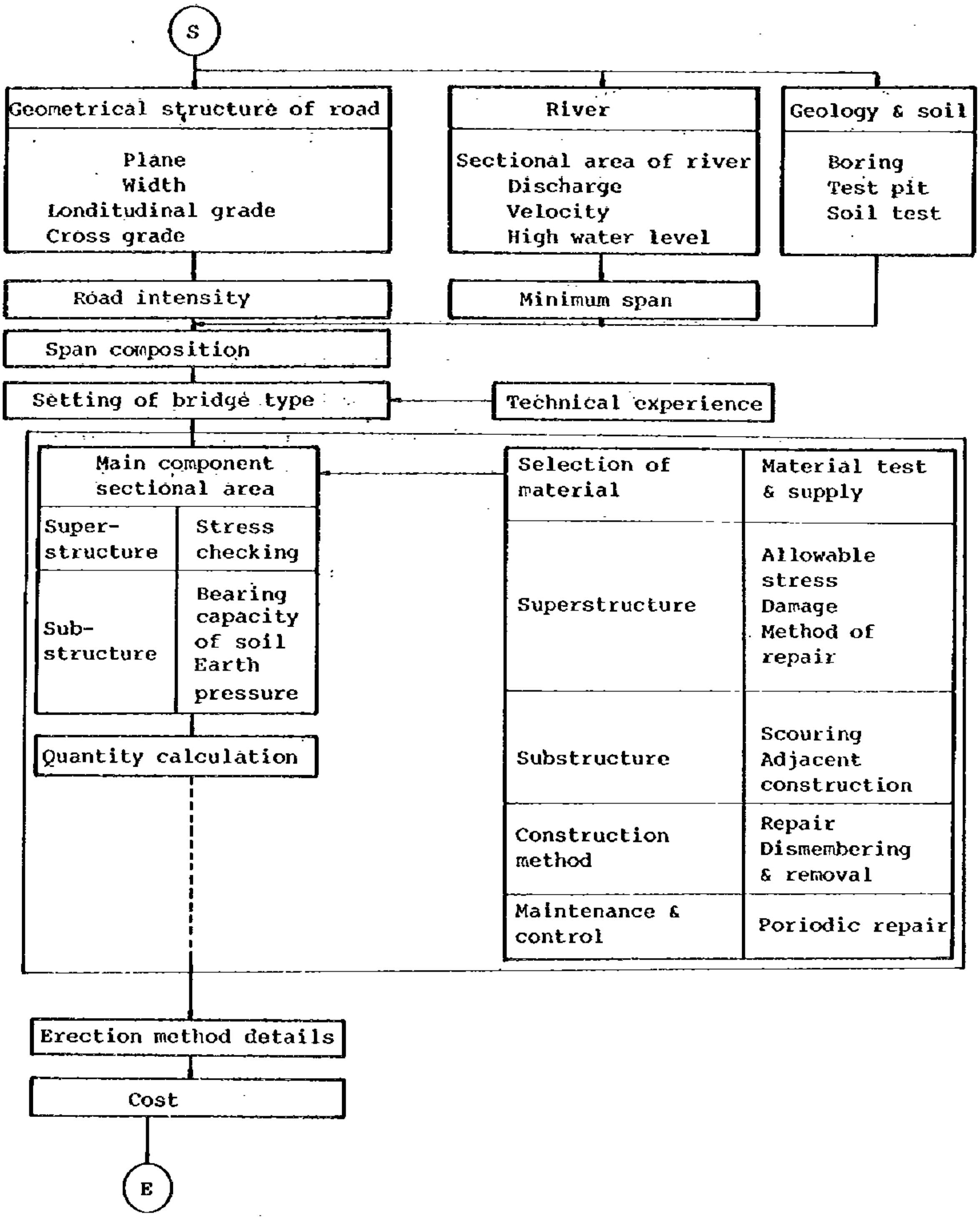


Fig. IX-2-1 Flow Chart of Comparative Study Procedures on Alternatives

2.4 Design Standards

Principally, the design standards will accord with the minutes (Appendix IV described in the Progress Report). Exemplary provisions are as follows:

(1) Load

Live load: HA loading at B.S. 153

Wind load: The estimated value for the probable mean wind speed for the duration of 10 minutes for 85 years has been obtained nearly approximate to the value provided in specification for highway bridge designs, Japan. Therefore, sepcification for Highway bridge design, Japan, applies to this case.

(2) Material: B.S. or its equivalent standard

(2) Allowable stress: BS153 or BS-CP110 shall apply in principle