CHAPTER 6 ALTERNATIVES AND TECHNICAL STUDY

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6.1 General

The objective of the study is to propose the most feasible crossing between Mombasa Island the South Mainland in both economic and technical and engineering terms.

In the Phase I study a total of 7 alternative routes and 12 alternative structures were studies by considering future regional development plans, road network, land use and technical and engineering aspects. As a result, two alternatives (bridge alternative B_1 and conventional tunnel alternative T_2) were selected for further study.

In the Progress Report II these two alternatives were studied using the trunk route network on Mombasa Island, which was studied in the same Report. As a result the tunnel alternative was abandoned because of many disadvantages in operation, traffic service to the island, construction difficulty and cost.

An alternative route study for the selected bridge alternative B_1 was made for the corridor between the existing ferry and the mouth of the harbour, considering alternative navigation clearances and accessibility to Nyerere Avenue. A crossing route, which passes over the narrowest channel portion in the harbour, was based on the existing land use of the resort and high class residential area and the length of the deturning route.

An alternative access study on Mombasa Island was made based on the determined bridge crossing alignment, traffic analysis and existing road improvement plans. Considering staged construction Alt. -D for the navigation clearance of 73.2 (one loop access with four lanes main bridge at the initial stage) and Alt. -A for 55 and 45 m (separate two lane construction at initial stage) were recommended for the preliminary design.

Structural studies for tunnels and bridges were incorporated into the alternative study based on the results of the topographic and hydrographical survey and soils survey.

In order to examine the possible practical alternatives, field reconnaisance, data collection and analysis were made. Field reconnaisance was carried out using aerial photos, topographic maps, a scale of 1:2,500 ~ 1:10,000, provided by MOTC and the Survey of Kenya.

In this report the terms "Main bridge" and "Approach bridge" are defined as follow:

"Main bridge" is defined as that part of the bridge consisting of one continuous structure including the span across the Kilindini creek.

"Approach bridge" is defined as that part of the bridge between the crossing bridge and the existing/planned road.

6.2 Design Standards

The project road design standard made reference to the Manuals by the Ministry of Works, and Ministry of Transport and Communications (ROAD DESIGN MANUAL, PART-I Geometric Design of Rural Roads January 1979, PAAR-III Materials and Pavement Design for New Roads, May 1981) and Japanese standards.

For the purpose of feasibility study, unlike the case of detailed design principles, use of the Japanese bridge standard will not lead to any practical difference of significance from the use of the British Standard. In this Study therefore, the Japanese bridge standard is generally followed for the sake of efficiency of work, since it is better acquainted by the expatriate specialists of the study team.

6.2.1 Traffic Capacity

A traffic capacity analysis was conducted using the traffic volumes predicted for various target years described in Chapter 5. The concepts and methodology used for the road capacity analysis are based on the "Road Design Standard" of Japan as similar operating conditions, type and size of vehicles, are found in Kenya and Japan.

The throughway capacity for the project road and existing roads are calculated by case using adjustment factors (lane width, lateral clearance, heavy vehicles, and disturbance factors to through traffic, etc.) against the basic capacity.

Table 6.2.1 TRAFFIC CAPACITY

	Traffic Capacity (PCU/day)
Project Road	
4 lane, 2 way	62,800
2 lane, 2 way	18,400
Existing Road	
4 lane, 2 way	31,400
2 lane, 2 way	12,500

Note: The capacity of existing roads is estimated by including the influence of existing intersections.

6.2.2 Geometric Design Standard

1) Function and Class of Road

The Mombasa Transportation Study was conducted by the Road Department, the Ministry of Works in 1972. This study provided the planning basis for the Likoni Crossing Construction Project and it recommended that the Project road be a primary distributor as the part of the link connecting West and South Mainlands as shown in Fig. 4.1.4. The road is expected not only to serve the local traffic such as in Mombasa Island but also to serve national and international traffic. From planning aspects the proposed crossing is therefore classified as class A, based on the "Road Design Manual" (part I: Geometric Design of Rural Roads), January 1979, published by the Ministry of Works (MOW).

The function of a national and international road will be fulfilled in the final stages upon completion of the whole link connecting the two Mainlands with a road to a high class standard. In the initial stages of the project road, full control of access is not necessary because of traffic demand and from an economic aspects.

2) Design Speed

The design speed of 80 km/hour is usually applied as the maximum speed for urban roads. However the area through which the project road will run is already urbanized and highly utilized for many pruposes.

In addition the trip length of most of the traffic is short and this pattern is estimated to continue past the initial stage, therefore the design speed of the project crossing is recommended to be 60 km/hour considering the developed situation of the adjacent areas.

3) Alignments

(1) Horizontal Alignment

Large radii curves are preferable for road users. The minimum radius for general road section (embankment and small bridge) is 200 m based on the design speed of 60 km/hr and the following standards:

	Kenya Standards	Japanese Standards
Desirable Min. Radius (m)	-	200
Min. Radius (m)	160	150

The horizontal alignment for the crossing bridge section should be tangent or in a flat horizontal curve for traffic safety and ease of construction. The minimum radius of 500 m is adopted for the bridge section. The absolute minimum radius of 120 m is however adopted for a loop access road at the Phase I construction.

(2) Vertical Alignment

For general road sections a maximum gradient of 5% is acceptable to maintain traffic capacity, safety, and users' comfort. For long upgrade sections such as the approach bridges of the project crossing examination for the necessity to provide slow moving vehicle lanes must be made. This influences the composition of the road section and hence the total construction cost.

Two cases: 5% gradient with a special slow lane and 4.3% gradient without a special slow lane were examined based on the Road Design Manual of MOW. As described in Progress Report II the 4.3% grade is proposed for the approach bridge for economic and technical reasons.

4) Cross-section

(1) Lane Width

A lane width of 3.25 m is adopted based on the relation between running speeds and carriageway widths, maximum design vehicle width (2.5 m), future traffic volume and economy of construction cost.

(2) Shoulder

The outer shoulder width is related to the parking space for disabled vehicles and the side clearance of running vehicles. In an urban area the shoulder width is usually limited due to the existence of adjacent built-up facilities and difficulty of land acquisiton. Therefore a half-shoulder width of 1.25 m is adopted. The inner shoulder width of 0.75 m is adopted for 2-way, 4 lane carriageway and 1.0 m for 2-way, 2-lane carriageway at Phase-I construction.

(3) Median

It is desirable to provide a median on the road having dual carriage ways for effective, smooth and safe traffic flow. However, the width of urban motorways is reduced for the same reason as in the shoulder. A median width of 0.5 ~ 1.5 m is proposed according to the stage construction, developed situation and traffic safety.

(4) Footpath

Currently 36,925 passengers use the Likoni Ferry free of charge. Among those only 7,641 persons are pedestrian without any vehicle mode. After 1990 a maximum 570 persons in a peak hour are estimated to walk over the crossing.

Three alternatives: full length of footpath, bus transport and partial footpath with elevators, were studied as described in the Appendix of Progress Report II.

As a result, the team recommended the provision of an effective 2 meter wide footpath for full length of the project road considering pedestrian's vested right, pedestrians behavior on a footpath and the expectation of pedestrians, cyclists and tourists.

5) Vertical Clearance

Based on the MOW's standard a vertical clearance of 5.2 m is adopted.

6) Frontage Road

For local traffic and traffic from the project road, the 7 m-wide carriageway (6 m + 2 x 0.5 m) is proposed for the frontage road. A footpath width of 4 m will be provided to such frontage road considering pedestrian needs and existing footpath width.

7) Criteria for Roadway Junction

The horizontal alignment around a roadway terminal (merging and diverging) should be more than 200 m radius based on Japanese Standards.

8) Width of Right-of-way

A minimum right-of-way width of 40 m, except the access to Lunga Lunga Road at the Phase-I, is recommended based on the MOW ROAD DESIGN MANUAL (PART-I) for Urban areas. The right-of-way of 30 m is recommended for the initial access to Lunga Lunga Road.

9) Other Items

The other main items of the recommended design standards are presented in Table 6.2.2.

Typical cross sections for the project road are shown in Fig. 6.2.1 and 6.2.2.

Table 6.2.2 GEOMETRIC DESIGN STANDARD

Item	Unit	Recommended Value
Terrain	-	Flat
Design Speed	km/h	60
Min. R.O.W Width	m	40 (30)
Lane Width	m	3.25
Shoulder Width: Outer	m	1.25
: Inner	m	0.75 or 1.00
Median Width	m	1.50 ∿ 2.50
Crossfall of Carriageway	%	2.5
Maximum Superelevation	%	10
Minimum Radii	m	200 (120)
Maximum Gradient:		
General Section	%	5
Crossing bridge section	%	4.3
Stopping Sight Distance	m	75
Minimum Vertical Curve Length	m	50
Minimum Horizontal Curve Length	m	100 or 700/θ
Vertical Clearance of Roadway	m	5.2

Note: 1) θ shows intersection angle for horizontal curve.
2) R=120 m is used for a loop access at Phase–I of the project as the absolute minimum value of 60 km/hr design speed.

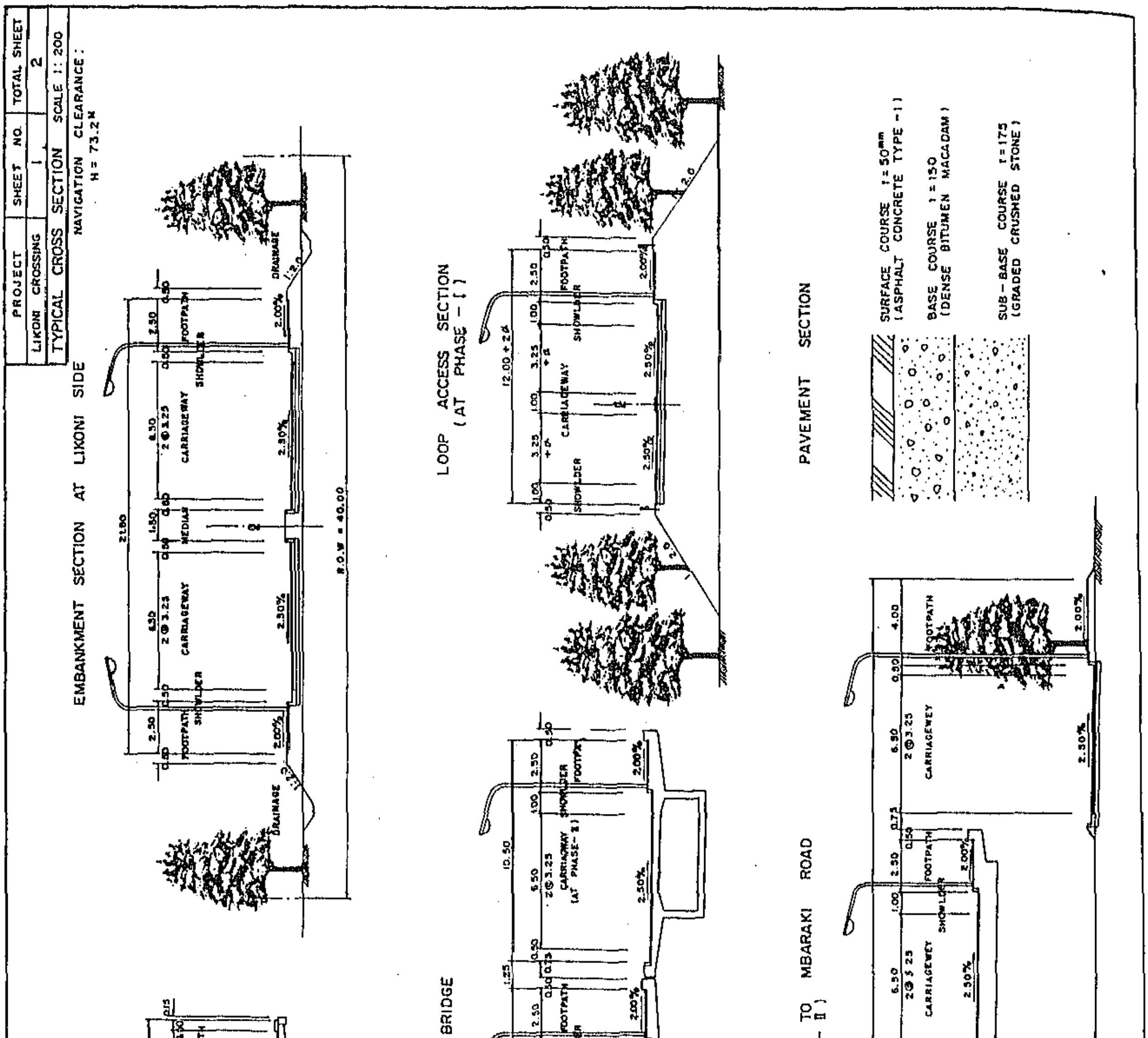


Fig. 6.2.1 TYPICAL CROSS SECTION (H = 73.2 M)

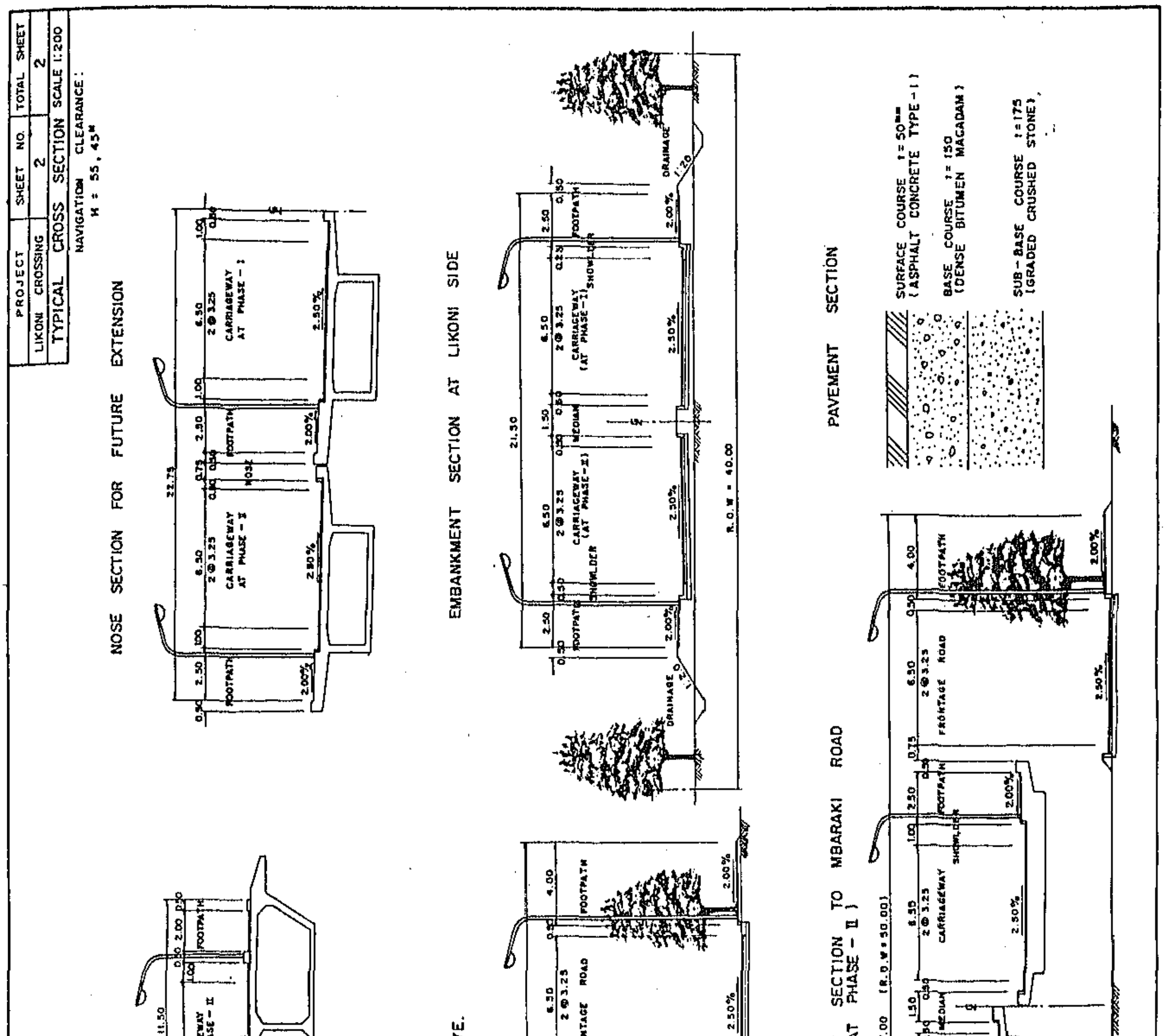


Fig. 6.2.2 TYPICAL CROSS SECTION (H = 55 AND 45 M)

6.2.3 Bridge Design Standard

The Bridge Design Standard of Japan Road Association is generally followed in this study, except some modifications made as required to adjust to local requirements from natural conditions (such as wind load, earthquake and temperature), project execution and material availability.

1) Design Loads

(1) Live Loads

The live loads to be adopted in the design are illustrated in Fig. 6.2.3 and Fig. 6.2.4.

(1) T-LOADING (LIVE LOAD FOR SLABS AND FLOOR SYSTEMS)

Class of Bridge	Loading	Gross Weight W(Ton)	Weight of a Front Wheel 0.1 W(kg)	Weight of a Rear Wheel 0.4 W(kg)	Width of a front Wheel b_1(cm)	Width of a Rear Wheel b_2(cm)	Length of Contact Area of a Wheel on the Road Surface a(cm)
1st	T-20	20	2,000	8,000	12.5	50	20
2nd	T-14	14	1,400	5,600	12.5	50	20

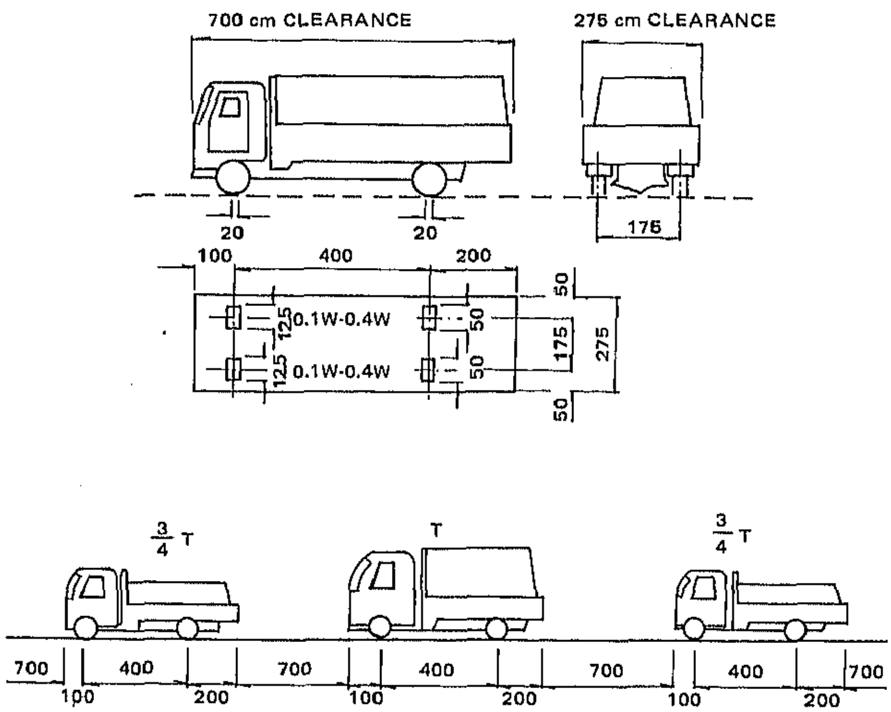


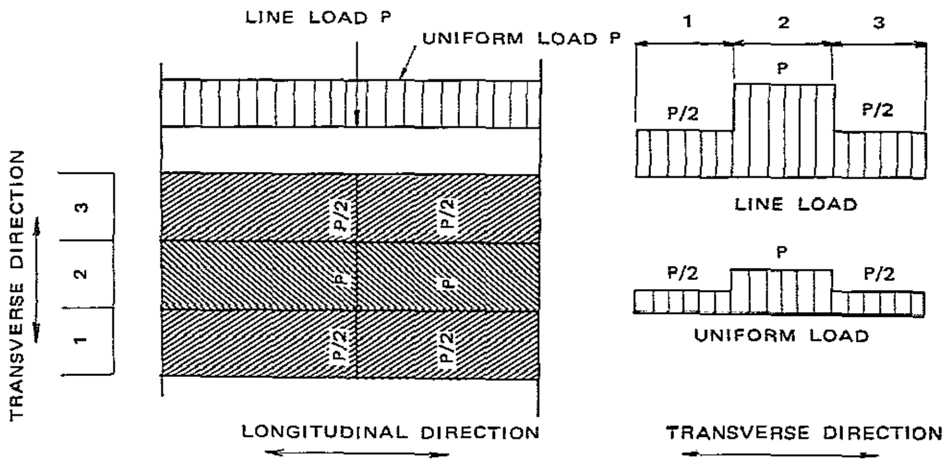
Fig. 6.2.3 T-LOADINGS

(2) L-LOADING (LIVE LOAD FOR MAIN GIRDERS)

Class of Bridge	Loading	Main Loads (up to 5.5 Meters in Width)			Sub-Loads
		Line Load P (kg/m)	Uniform Load, P (kg/m²)		
			L ≦ 80	L > 80	
1st	L-20	5,000	350	430-L But not less than 300	50% of Main Loads
2nd	L-14	70% of those of L-20			

WHERE,

L = SPAN LENGTH IN METERS



(3) TT-43 LOADING

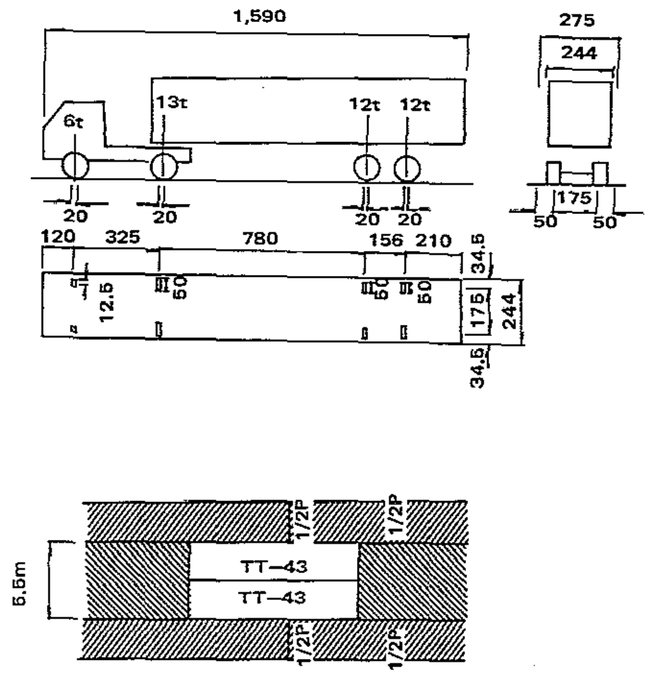


Fig. 6.2.4 L-LOADING & TT-43 LOADING

(2) Wind Load

Wind load for structural design in this project is assumed to be that of 30 m/sec velocity based on the past records as described in Appendix C.

(3) Seismic Load

Seismic characteristics in the project area are described in Appendix B and outlined as follows:

- Mombasa locates in Zone-VI of the seismic zone map, and has experienced only very slight earthquakes in the past.
- For high rise structures seismic design is required.
- The maximum acceleration occurred in the past was 44 gal. Hence, Horizontal Seismic Coefficient to the earthquake load is: Kh = 0.044.
- Earthquake of Zone VII (Horizontal Seismic Coefficient to the earthquake load: Kh = 0.094) occurs about once in ten to 20 years but only in western part of Kenya.

In consideration of the importance and economic scale of this project, however, a horizontal seismic coefficient Kh = 0.10 is adopted in the structural design so that earthquakes of zone VII in modified Mercallic Scale, or 94 gal. in maximum acceleration, which so far occurs only in other parts of Kenya, can be withstood should it occur at this project site.

(4) Temperature effects

The basis for temperature load computation for structural design are taken as follows:

- Range of temperature for steel bridges: from +10°C to +50°C
- Difference of temperature between sunny and shady parts of steel bridges: 15°C.
- Range of temperature for concrete bridges: from +10°C to +40°C.
- Difference of temperature between sunny and shady parts of concrete bridges: 5°C.

(5) Materials

Generally materials for construction and their use shall be based on the Japanese specifications or their equivalents. However, the materials which can be procured in Kenya should be in compliance with the respective specifications in Kenya.

Cement

KS 02-21 (1976):	for ordinary Portland cement
BS No. 12 (1982):	for ordinary Portland cement
BS 4027 (1980):	for sulphate-resisting Portland cement

Steel:

KS 02-22 (1976)	for steel bar for concrete reinforcement
BS-4461	for steel bar for concrete reinforcement
JIS G3112	for steel bar for concrete reinforcement
JIS G3101	for structural steel
JIS G3106	for structural steel
JIS G3114	for structural steel
JIS G3444	for steel pile
JIS G3536	for steel wire and strand for prestressed concrete
JIS G3109	for steel bar for prestressed concrete

Concrete

$\delta k = 350$ kg/cm^2	for prestressed concrete
$\delta^k = 240$ kg/cm^2	for reinforcement concrete

where: δk is the nominal design strength of concrete on 28 days.

Note: Compression stress of 350 kg/cm^2 was selected refering to the concrete test result of New Nyali Bridge, which the results were varied in the range of 340 kg/cm^2 – 610 kg/cm^2. The value was selected on the safety side and this is the subject the material testing in the detailed design stage.

6.3 Alternative Study

6.3.1 Alternative Crossing Study

1) Transportation Master Plan

The Mombasa Transportation Study, which was conducted by the Road Department of the Ministry of Works in 1972, has provided the basic instruction to the Project.

In the Master Plan the 1966 Road Network proposed primary and district distributors on Momasa Island and three mainlands including water crossings based on the urban, structure and regionsl development plans.

2) Alternative Corridor

The Likoni crossing has a direct meaning as an alternative to the existing ferry, but in the broader meaning it should form a part of the trunk road network in the Monbasa area. From this sense the alternative corridor was determined to extend the full length of Kilindini Harbour.

3) Selection of Alternative Crossings

Alternative crossings were selected on the basis of the road network (including access road), land use, harbour creek, existing facilities (including port facilities), soils condition and technical considerations as described in Progress Report I.

For the structural alternatives three types: bridge, immersed tube tunnel (sunken-tube tunnel) and conventional tunnel were studied, using the channel conditions of Kilindini Harbour.

In all 12 alternatives were selected including 7 routes from structural aspects. These are alternatives B1 to B6 for bridge structures and T1 to T4 for tunnel structures as shown in Fig. 6.3.1, and described in Table 6.3.1.

Alternative routes are evaluated by factors, such as accessibility to port and industrial area and town centre, connection with primary distributors (trunk roads) and relation to the development of South Mainland. Table 6.3.2 shows the evaluation for bridge alternatives. Table 6.3.3 indicates that the tunnel routes have longer access routes to the existing and planned road system in terms of accessibility. Table 6.3.4 shows the comparison of all alternatives. All bridge alternatives were compared using the vertical navigation clearance of 73.2 m (240 feet).

Bridge alternative B1 – 1 is the case where the navigation clearance is reduced to 55 m.

All the alternatives were studied in Progress Report-I. As a result the bridge scheme B1 and tunnerl scheme T2 were selected as the most viable alternative structures for further study.

Table 6.3.1 DESCRIPTION OF ALTERNATIVE CROSSINGS

Alternatives	Type of structure	Description
B1	Bridge	- Closest alignment to harbour mouth connecting Mzimle with Likoni at the south of the existing ferry
T1	Conventional Type Tunnel	- Narrow channel width (420 m) - Channel depth (-47 m)
B2	Bridge	- Alignment crossing at the edge of Mbaraki quay
T2	Immersed Tube Tunnel & Conventional Type Tunnel	- Channel with (700 m) - Shallowest channel crossing (-27 m)
B3	Bridge	- Alignment crossing at Ras Liwatoni and Ras Bofu - Channel width (760 m)
B4	Bridge	- Alignment crossing the west headland of Liwatoni creek and Muesa creek
T3	Immersed Tube Tunnel & Conventional Type Tunnel	- Channel width (1,160 m) - Channel depth (-42 m)
B5	Bridge	- Alignment crossing at Ras Kilindini
T4	Conventional Type Tunnel	- Channel width (625 m) - Channel depth (-44 m)
B6	Bridge	- Alignment crossing at Ras Kikangoni - Channel width (880 m)

Note: B1-B6: Bridge Alternatives
T1-T4: Tunnel Alternatives

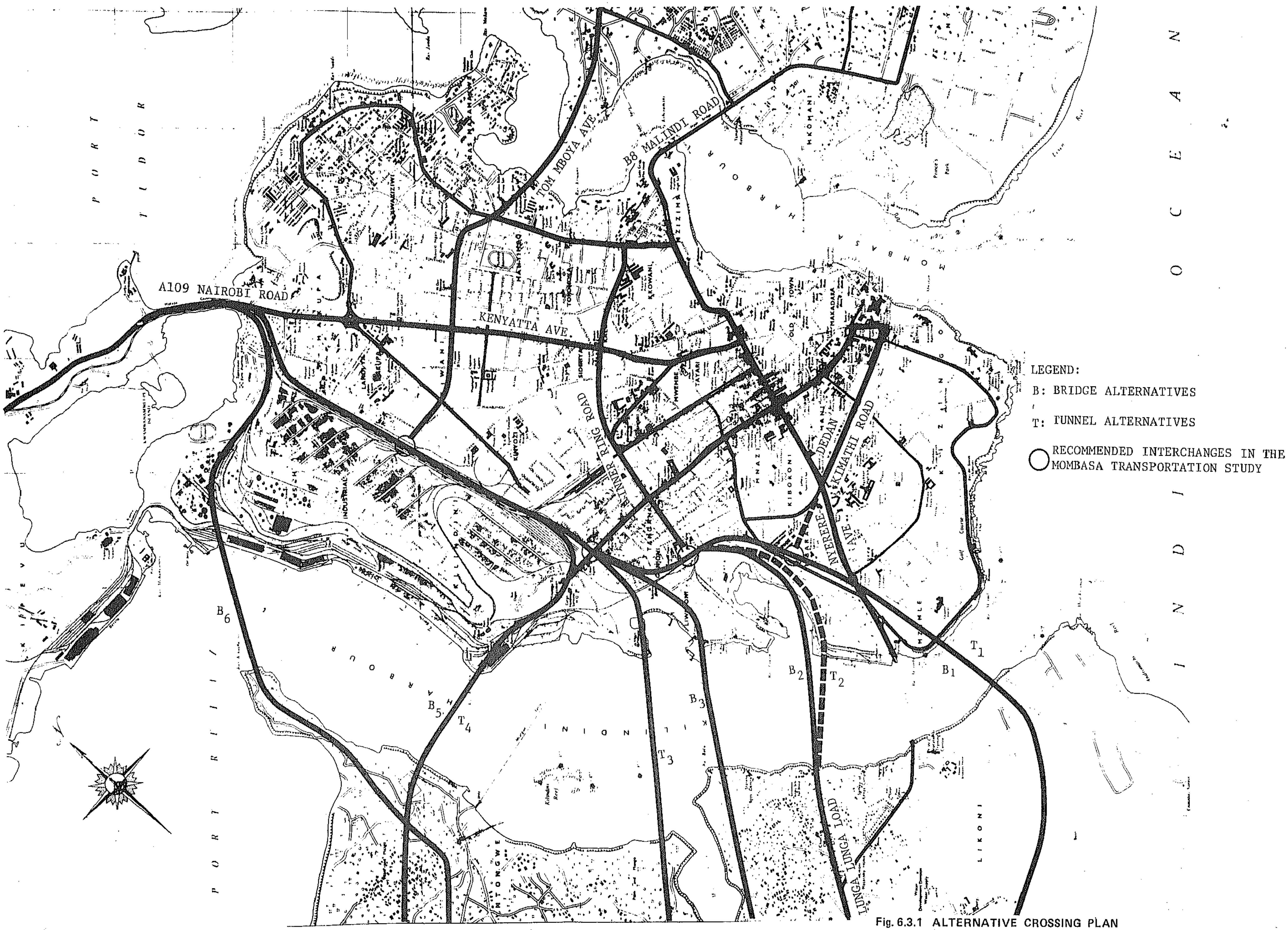


Fig. 6.3.1 ALTERNATIVE CROSSING PLAN

Table 6.3.2 GENERAL EVALUATION OF ALTERNATIVES

Item \ Alternative Routes	B1	B2	B3	B4	B5	B6
1. Access to Port/ Industrial Area	Indirect access		Semi-direct access		Direct access	Direct access
2. Access to town centre	Good access to Inner Ring Road		Inconvenient access to island south			Bad
3. Connection to distributor - North (New Nyali)	Good Access to the North					Possible but indirect access
- West (Makupa-causeway)	Good Access to the West					
- South	Good		OK		Bad	
4. Relation to Development of South Mainland	Good for housing development	No special meaning due to connection to existing road	Good for new Port (Dongo Kundu) and industrial area			

Table 6.3.3 ACCESSIBILITY OF ALTERNATIVES

Alternative route	Access Point (from edge of channel to existing ground)	Possible excess to existing/Planned road
Conventional Tunnel		
T1	Makande road	Makupa causeway (A109)
T2	Around Mijikenda street, just north of Mombasa station	New Nyali Bridge (B8)
T3	Makupa causeway	Makupa causeway (A109)
T4	Makupa causeway	Makupa causeway (A109)
Immersed Tube Tunnel		
T2	Moi Avenue	Moi Avenue, New Nyali Bridge (B8)
T3	Around Mijikenda street, just north of Mombasa station	New Nyali Bridge (B8)
Bridge		
B1	Kimathi Avenue	Kimathi Ave., Inner Ring Road, Moi Avenue, New Nyali Bridge
B2	Tangana Road (Inner Ring Road)	Inner Ring Road, Moi Ave., New Nyali
B3	Moi Avenue	Moi Ave., New Nyali Bridge
B4	Moi Avenue	Moi Ave., New Nyali Bridge
B5	Moi Ave., (Necessary arrangement of port berth)	Moi Ave., New Nyali Bridge
B6	Makande Road (Necessary arrangement of port area and danger due to pass oil firm area)	Makupa causeway

Table 6.3.4 COMPARISON OF ALTERNATIVE CROSSINGS

Items \ Scheme	Bridge Alternative							Tunnel Alternative					
								Conventional tunnel				Imersed tube tunnel	
	B1	B2	B3	B4	B5	B6	B1–1	T1	T2	T3	T4	T2	T3
Physical Conditions													
1. Channel width (m)	420	700	700	1,210	640	880	420	420	700	1,160	625	700	1,160
2. Channel depth (m)	48	26	44	48	52	50	48	47	27	42	44	27	22
Main Span													
3. Lengths (m)	790	900	910	1,450	850	1,140	790	550	830	1,350	850	960	1,375
Main Span	430	600	610	970	570	760	430						
Side Span	180x2=360	180x2=300	180x2=300	240x2=480	140x2=280	190x2=380	180x2=360						
4. Type of Structures	Cable-stayed	Suspention	Suspension	Suspension	Suspension	Suspension	Cable-syated	–	–	–	–	–	–
5. Formation level in centre of Bridge (m)	81	81	84	89	80	80	62	–100	–60	–90	–94	–31	–35
Approach													
6. Lengths (m)	2,330	2,370	2,690	2,010	2,190	2,020	1,500	7,830	4,840	6,820	7,340	3,020	2,945
7. Type of structures	Prestress Concrete	Prestress Concrete	Prestress Concrete	Prestress Concrete	Prestress Concrete	Prestress Concrete	Prestress Concrete	Convnl. T Open Sn.	Convnl. T Open Sn.	Convnl. T Open Sn.	Convnl. T Open Sn.	Shild T. Open Sn.	Shild T. Open Sn.
8. Gredient (%)	5	5	5	5	5	5	5	3	3	3	3	3	3
Access road													
9. Connection point of bridge in Mombasa Is.	Mbaraki Road	Tangana Road	Mwakilingo Road	Mwakilingo Road	Mwakilingo Road	Shimanzi Road	Mbaraki Road	–	–	–	–	–	–
10. Portal of tunnel in Mombasa Island	–	–	–	–	–	–	–	A–109 Nairobi Road	Marshallin yard of K.R	A–109 on Causeway	Makupa Causeway	Marshalling yard of K.R.	Tangana [illegible]
11. Evaluation for connecting condition	Good	Good	Average	Poor	Poor	Poor	Good	Poor	Poor	Poor	Poor	Average	Poor
Costs													
12. Construction costs (M.S.)	1,344	1,807	1,873	3,571	1,635	2,196	1,117	4,202	3,223	4,775	4,372	3,828	4,220
Main span (M.S.)	739	1,139	1,151	3,031	1,029	1,756	739	679	1,025	1,667	1,050	1,440	1,063
Approach structure (M.S.)	566	629	682	501	507	401	340	2,603	1,548	2,208	2,428	1,896	1,662
Facilities (M.S.)	39	39	40	40	39	39	38	920	650	900	894	492	495
13. Land acquisition costs (M.S.)	65	38	72	44	46	46	45	149	129	114	156	73	74
14. Total costs (M.S.)	1,409	1,845	1,945	3,616	1,682	2,242	1,162	4,351	3,352	4,889	4,528	3,901	4,294
15. Maintenance costs (M.S. Year)	10	14	15	40	13	22	9	25	14	24	24	11	11

6.3.2 Future Trunk Road Route on Mombasa Island

1) General

Mombasa Island has the biggest traffic generation and attraction in East Kenya. The future trunk road from Nairobi towards Tanzania was proposed in the Mombasa Transportation Study to pass through Mombasa Island.

For the planning of the project road, it is important to study the route location of the future trunk road on the island. It can be judged that the location of the route of the required trunk road should be in the western part of the Island to meet the following requirements:

- The two mainlands, west and south should be connected with a shortest route without making any detour.
- The location of the route should have such functions as:
  - accessibility with Malindi Road from north and district major roads;
  - possibility in establishing a more effective road network; and
  - possibility in easier traffic distribution.
- The trunk road can be expected to become a high class road serving a limited number of district roads. Its location should therefore be in a position to minimize its adverse effect to the adjacent residential area, CBD and other urbanized areas.

2) Alternative Routes

Considering the existing land use and roads four alternative routes were selected as shown in Fig. 6.3.2. These are described in Progress Report II and as follow:

Alternative A:	along Shimanzi Road (Mbaraki Road – Siding for Mbaraki Berth down to the south – Shimanzi Road – Makupa Creek)
Alternative B:	along the railway right-of-way (Mbaraki Road – Archbishop Makarios Road – Mwakilingo Street – Railway siding – Makupa Causeway)
Alternative C:	along Mwangeka Road (Mbaraki Road – Archbishop Makarios Road – Mwakilingo Street – Mwangeka Road – Makupa Causeway)
Alternative D:	along Lumumba Road (Mbaraki Road – Archbishop Road – Mwakilingo Street – Lumuba Road – Kenyatta Avenue).

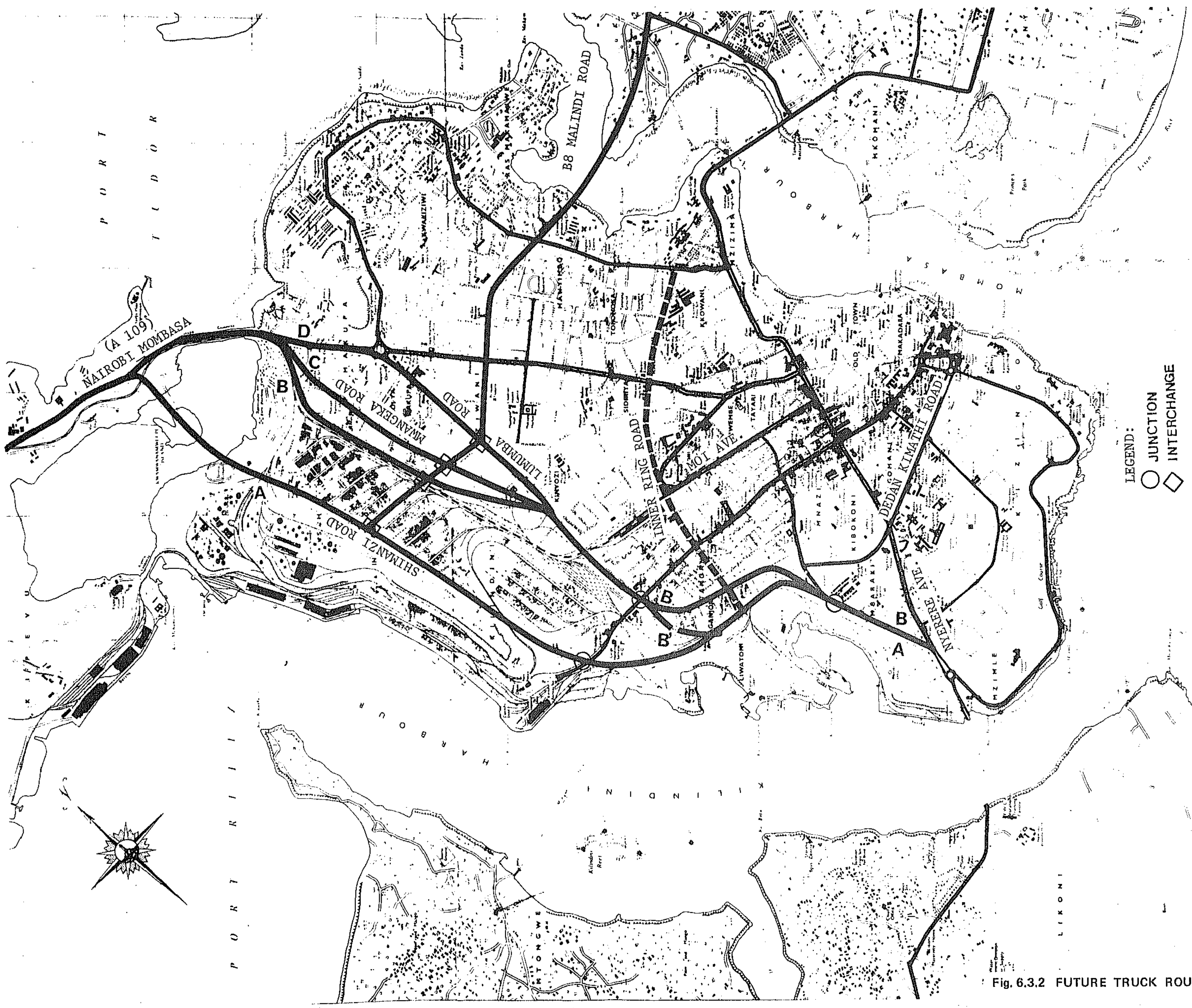


Fig. 6.3.2 FUTURE TRUCK ROUTE ON MONBASA ISLAND

3) Comparison of Alternative Routes

Each alternative route is evaluated through four check points: traffic; environs; construction difficulty; and construction cost; and is tabulated in Table 6.3.5.

As a result Alternative B Route which is along side the Nairobi – Mombasa railway line is recommended for the best location of the proposed trunk road on Mombasa Island. The major favourable aspects of Alternative B Route are summarized as follows:

- No problems in traffic access and control to the connection with Inner Ring Road (served by frontage roads) and Moi Ave.
- Favourable use of boundary areas between different land uses such as port and industrial area and commercial and residential area.
- Traffic to and from Mombasa Island: on this route traffic service can be provided not only to the port and industrial area but also to the CBD and residential area and the whole Island.
- Relocation of rail freight sidings at the junction of Malindi Road are required but an improved rail service to south can be made.
- Replacement of the housing complex for the railway workers in the north: space available for its transfer into the right-of-way of the railway facilities at the back of Mombasa Station.

Table 6.3.5 COMPARISON OF ALTERNATIVE ROUTES OF THE PROPOSED TRUNK ROAD ON MOMBASA ISLAND

Check Item / Alternative Route	A	B	C	D
1. Traffic				
1) Formation of Trunk Road network (connection with Malindi extension)	Difficult (due to the existence of railway facilities, warehouses and factories in the industrial area).	Possible, but required rearrangement of rail sidings for rail freight.	Possible	Possible, but problem in control of traffic at a junction of existing road with ramps (5-leg at grade intersection).
2) Interchange				
(1) Spacing	0.7,1.02,1.57 km	0.7,0.76,1.42 km	0.7,0.76,1.37 km	0.7,0.76,1.34 km
(2) Installation of IC ramps	Impossible to provide ramps of interchange with Inner Road, but two IC's can be provided			
(3) Control of traffic to/from frontage roads	Difficult on ramps of Inner Ring Road IC with sidings of rail freight	Favourable traffic service for Inner Ring Road with provision of frontage road	Same as B	Same as B
(4) Improvement to existing traffic flow	Better traffic service in the industrial area	- Better traffic service on the whole island - Division by the existence of Mombasa Station can be connected by Trunk Road	Same as B	Same as B but improvement required for the intersection of Kenyatta Ave. with Makande Road
2. Environment Length of affected residential area (total of both sides)	2,900 m	3,410 m	4,760 m	4,530 m
3. Construction problem (in urbanized/industrial area)	Not much difference among the four alternatives			
4. Construction Quantity and cost.				
(1) Length of Roads	5.65 km	4.89 km	4.76 km	4.30 km
(2) Bridge and viaduct	3.38 km 939.0	2.9 km 807.7	3.9 km 1,083.5	3.7 km 1,027.9
(3) Frontage road and Access road	4.9 km 54.9	4.8 km 53.7	3.2 km 35.8	7.6 km 85.2
(4) Land Acquisition compensation cost	- 83.3	- 101.0	- 201.6	- 191.6
(5) Total (Mill Shs)	1,077.1	960.5	1,321.0	1,304.7

Note: Above estimates are based on cost estimates in Progress Report I.

6.3.3 Comparative Study of Bridge (B_1) and Tunnel (T_2)

1) General

The bridge alternative B_1 and tunnerl alternative T_2 (conventional type) were reviewed and studied in the Progress Report II on the comprehensive aspects of traffis service and safety, operation, construction, cost and environment, etc. based on the future trunk road alignment studied in 6.3.2.

2) Tunnel Gradient

The tunnerl gradient is related to traffic safety, ventilation and construction efficiency.

The down speed of large vehicles into the tunnel causes traffic congestion which is against traffic safety and driving comfort. According to the statistics on accidents in tunnels in Japan, automobile fire accidents occur 1.4 times more frequently than on normal roads and highways.

The steeper the ascending gradient becomes, the greater the need becomes for tunnel ventilation against CO-gas and smoke emission. For example, while the coefficient of air-requirement in a unit length of a tunnel is 1 for a 3% gradient slope, it becomes 1.6 for a 5% gradient slope.

Excavation work for long tunnels mostly relies on trucks for the transportation of dug earth and construction materials. Recent improvements to exhaust gas purification technics, trucks can work on a 3% gradient slope.

Therefore, it is to be recommended that the slope gradient shall be as flat as possible to minimize traffic accidents and construction cost and period.

3) Traffic Service to Mombasa Island

The Mombasa Transportation Study recommended the provision of 4 interchanges on Mombasa Island to serve local traffic. It is a vital for the trunk road plan to assure these traffic services to the Island as the traffic analysis shows that the Island accommodates 75.4% of the total existing crossing traffic and that this traffic in 2010 is estimated to increase to 84.3%.

Fig. 6.3.4 shows the tunnel sections by alternative gradients to provide of the same equivalent traffic service to the Island as the bridge alternative. Table 6.3.6 shows the stituation of interchange connections and streets related to the interchanges (I.C.) on the comparison between tunnel and bridge.

It is not practical to construct interchanges and related streets under the existing ground. The tunnel alternative (T_2) has big disadvantages to traffic services to the Island compared to the bridge alternative (B_1) as shown in Fig. 6.3.3.

Table 6.3.6 PHYSICAL FEATURES OF INTERCHANGE AND RELATED STREETS BY TUNNEL AND BRIDGE

Alternative / Item	Tunnel Gradient (T2) 3%	Tunnel Gradient (T2) 5%	Bridge Alternative (B1) 4.3%
Interchange			
- Dedan Kimathi	Underground	Underground	On the ground
- Inner Ring Road	Underground	On the ground	On the ground
- Moi Avenue	Underground	On the ground	On the ground
Major street length related to underground I.C.	L = 7,810 m	L = 1,150 m	None

4) Cost Estimation

The conditions for the cost estimation are listed as follows:

Tunnel (T_1)

Horizontal alignment:	As shown in Fig. 6.3.3.
Vertical alignment:	As shown in Fig. 6.3.5.
Tunnel cross section:	As shown in Fig. 6.3.4. 2-way 4 lanes with footpath (2.0 m wide x 2.5 m high) only for channel section.

Bridge (B_1)

Total bridge length:	3,280 m
– Main bridge:	830 m
– Approach bridge:	2,450 m
Bridge cross-section:	As shown in the "Design Standard". Dual-carriageway of 4-lanes with footpath on both sides of the bridge.

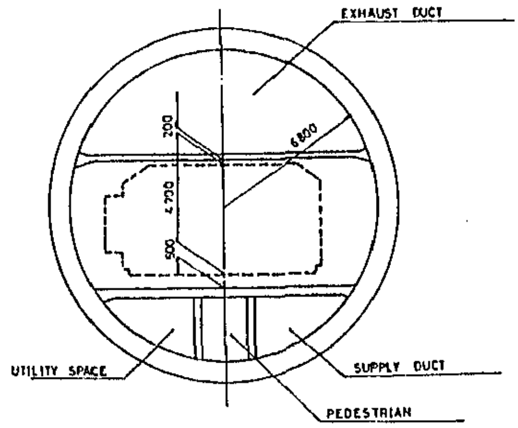


Fig. 6.3.4 CROSS-SECTION OF CONVENTIONAL TUNNEL (FOR 2-LANE CHANNEL SECTION)

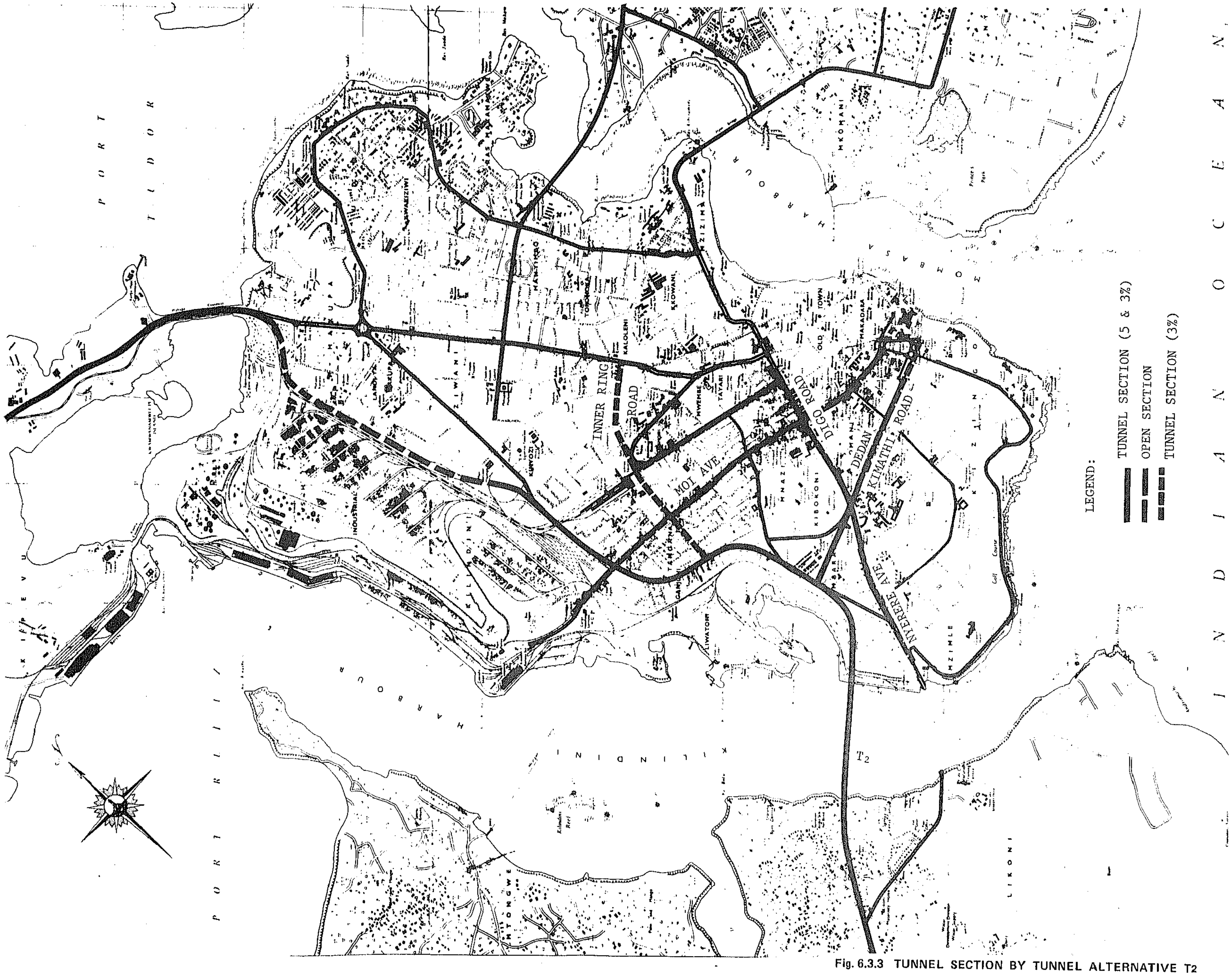
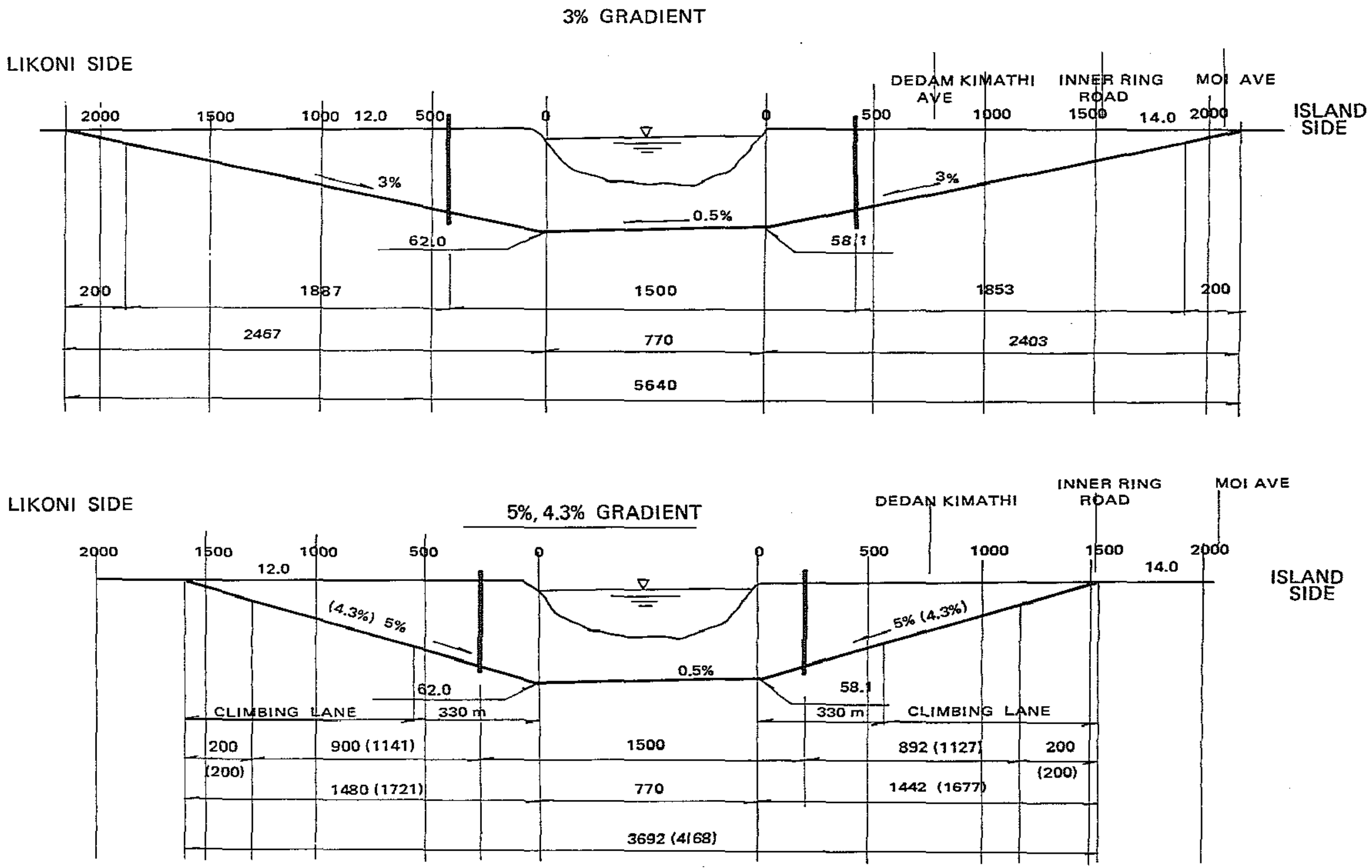


Fig. 6.3.3 TUNNEL SECTION BY TUNNEL ALTERNATIVE T2



NOTE: FIGURES IN BRACKET ARE THE LENGTHS FOR 4.3% GRADIENT

Fig. 6.3.5 TUNNEL PROFILLES (CONVENTIONAL TUNNEL T2)

The same unit costs were used as those for comparison in Phase I. Table 6.3.7 shows the costs for the tunnel (gradient of 3% and 5%) and the bridge.

Table 6.3.7 COST COMPARISON OF TUNNEL (T2) AND BRIDGE (B1)

Alternative / Item	Tunnel Alternative (T2)		Bridge Alternative (B1)
	3%	5%	4.3%
1. Construction Costs (M.S.)	3,720	3,050	1,412
- Main Span/ Crossing	1,179	1,179	776
- Approach Structure	1,801	1,188	595
- Facilities	740	683	39
2. Land Acquisition & Compensation Cost	129	78	69
3. Operation/ Maintenance Cost (M.S./Year)	37	32	5

As a result the tunnel alternatives, in both cases of gradient, have big disadvantage in the cost.

5) Sepcific Tunnel Subjects

(1) Tunnel hazard and facilities

The greatest hazard in a tunnel is the smoke caused by a fire resulting from a traffic accident.

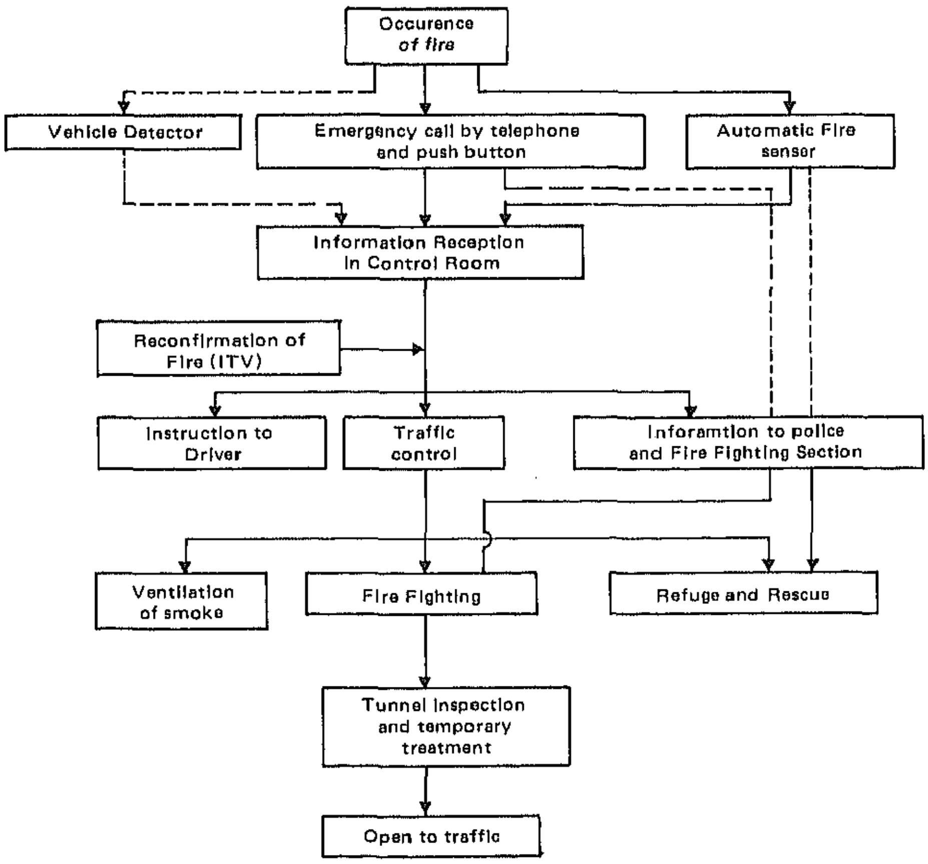
To ensure traffic safety and comfort for vehicles and pedestrians, and to minimize the losses caused by an accident, many kinds of ancillary facilities must be provided for tunnel operations as shown in Table 6.3.8.

These facilities should be maintained so as to operate properly at any moment. Fig. 6.3.6 shows the ordinary countermeasures and processes to be taken in case of a fire.

Table 6.3.8 TUNNEL ANCILLARY FACILITIES

Ancillary Facilities	Purpose
(1) Ventilation system	For health, safety and comfort of pedestrian, inspector and worker in the tunnel as well as maintaining the clear sight of drivers
(2) Lighting system	For safety drive in the tunnel and its entrance section
(3) Signal and Signs	For traffic control in case of incident and normal instruction (warning)
(4) For fighting and alarm system	To sense automatically a fire and transmit it to the control room, fire hydrants and extinguishers
(5) Television system	To minotor the traffic flow at the entrance section and in tunnel
(6) Laudspeaker system	To guide drivers in safe in case of incident
(7) Emergency telephone system	For driver's emergency call or any trouble occurred
(8) Sump drainage system	To discharge incoming rainwater, leaking water and firewater
(9) Power distribution system	To make effective use of the tunnel facilities as above
(10) Control system	On collection of data and information suitable countermeasures made to the relevant sections of tunnel as well as to nearby community authorities

Fig. 6.3.6 COUNTERMEASURES AND PROCESS IN CASE OF FIRE



(2) Back-up assistance

A tunnel shall not be operated independently of other facilities in the region, but be incorporated as part of the whole facilities of the region. In order not to make a tunnel the weak point of the region, the whole region must give full back-up assistance to the tunnel as listed below:

– Complete enforcement of traffic rules and anti-traffic violation control

– Thorough traffic patrol in the tunnel

– Stable supply of electric power

– Establishment of fire-fighting system

– Establishment of ambulance system including necessary hospitals

– To strengthen the requirements for private vehicle maintenance.

(3) Construction difficulty

Based on the soils survey conducted for the bridge alignment and Mbaraki berth data, the subsoil conditions of the tunnel alignment are sand and coral with high permiability. These soils involve high possibility to water in-flow during the tunnel excavation especially under the sea against which strenuous supplementary construction methods such as water sealing and soil stabilization are required. In the case of water in-flow the tunnel excavation faces an extremely difficult situation due to limitless seawater, even if careful countermeasures are taken before excavation. This depends greatly upon uncontrollable nature, resulting in a larger risk for the tunnel scheme, even after a complete subsoil survey and leads to a high final construction cost together with a longer construction period.

(4) Environmental Influence

Two air shafts will be required along the harbour channel route. Concentrated exhaust gases from the ventilation towers will be spread over the area. This effect cannot be ignored upon the resort and residential areas. During and after construction of the tunnel, considerable settlement at ground level will occur due to soil looseness and cavitation caused by the tunnel excavation. This must be compensated by the tunnel owner.

6) Recommendation

A tunnel is a sensitive structure and requires care in its routine operation since it is an enclosed tube unlike an ordinary road (including bridges).

As the result of the comparative study the team has recommended the bridge alternative (B1) as the Likoni Crossing Scheme over the tunnel scheme, which has many disadvantages such as, pedestrian passage, traffic safety, continuous tunnel operation, construction cost, traffic service to the island, construction difficulties (sand and silty sand) and environmental problem.

6.3.4 Alternative Alignment Study for Bridge Crossing

1) Genral

The alignment of bridge crossing is studied for the Harbour mouth area considering accessibility to Nyerere Avenue and possible navigation clearance of the Kilinidini Harbour as described in the Interim Report of this project.

2) Description of the Study Area

The study corridor is mainly used for residential (high class) and resort (hotel, golf course, park, etc.) purposes on both the island and South Mainland.

The Kilinidini Channel is at the narrowest, 400 ~ 500 m wide around the mouth of the harbour and widens towards the inner harbour.

The hinterland of Mbaraki berth, between Nyerere Ave., Mbaraki Road and Mbaraki Greek, will be developed as an industrial area producing soda ash, etc. A private catholic school, located east of Nyerere – Mbaraki intersection, is the most famous primary school in Mombasa and have plans to expand to the south.

3) Comparison of Alternative Alignment

Total three alternatives were established in the eastern subcorridor of the ferry as shown in Fig. 6.3.7. The alternatives are described as follows:

Alternative – A : Alignment passing through the narrowest channel location. (Navigation clearance: 45, 55, 65 and 73.2 m)

Alternative – B : Alignment passing just east of Oceanic Hotel (Navigation clearance: 55 m)

Alternative – C : Alignment at the mouth of Kilindini Harbour and located at the east end of the harbour (Navigation clearance: 65 and 73.2 m)

Table 6.3.9 shows the comparison of seven alternatives, all to Nyerere Ave. and Mbaraki road.

The team has proposed Alternative-A without loop ramps because it has big advantages in less community disturbances and vehicular running costs.

The major findings are described as follows:

- Detour route (Alternatives B and C) have disadvantages on running costs.
- Alternatives B and C have the problem of disturbing the existing high class residential houses along the coast.
- All alternatives must have loop ramps to access to Nyerere Ave. except 45 m clearance of Alt.-C .

Fig. 6.3.7 ALTERNATIVE ALIGNMENT FOR BRIDGE CROSSING

Table 6.3.9 COMPARISON OF ALTERNATIVE CROSSING ALIGNMENT

Alternative	A				B	C	
	A - 45	A - 55	A - 65	A - 73.2	B - 55	C - 65	C - 73.2
1. Navigation clearance (m)	45	55	65	73.2	55	65	73.2
2 Detouring distance (m) Running Cost / 20 years (million shillings)	(1,997 m) - 0				(2,696 m) 699 m, 406	(2,975 m) 938 m, 545	(2,935 m) 938 m, 545
3. Influence to community	- No problem to the community due to park on the Island - Less problem to high class residential area due to shorter passing distance in Likoni area				- Big problem to high class residential area on the Island and bigger problem on Likoni area		
4. Accessibility to Nyerere Ave. Necessary distance access to Nyerere Ave. without loop ramp (m)	Possible access with loop ramp				Possible access with loop ramp		
	270	500	745	910	265	295	620
5. Cost (Million shilling)							
1) Loop Alternative							
Bridge	1,112	1,256	1,389	1,511	1,276	1,411	1,516
Land	2.8	2.9	3.0	4.1	4.4	5.7	6.0
Compensation	5	5	7	10	19	20	21
Total cost	1,119.8	1,263.9	1,399.0	1,525.1	1,299.4	1,436.7	1,543.0
2) Mbaraki R. Access Alternative							
Bridge	1,060	1,243	1,420	1,557	1,243	1,393	1,535
Land	2.5	2.6	2.8	2.9	3.9	4.6	4.7
Compensation	5	5	5	5	19	19	19
Total Cost	1,067.5	1,250.6	1,427.8	1,564.9	1,265.9	1,416.6	1,558.7

– In case of comparing alignments with the same navigation clearance the total costs are almost same, irrespective of the presence or absence of loop ramps.

6.3.5 Alternative Navigation Clearances

1) General

Upto the Progress Report II the study had been made for the highest clearance (240 feet) across the Kilindini Harbour. The clearance is eventually the principal requirement of the current harbour. It is also necessary to study the lower clearance than the current requirement for the project feasibility purpose. This study was made in the Interim Report of the project.

The Mombasa Port currently has three functions, commerce, military and tourism. On these functions the team tried to establish some alternative clearances and finally to propose the optimum scheme based on "East African Ports Development Study" conducted by the East African Harbour Corporation in 1977.

2) Port Clearance and Ship in the Future

(1) Port clearance

The navigation clearance of a port varies depending on the needs and characteristics of the port as shown in Table 6.3.10.

(2) Ships in the future

Table 6.3.11 shows the standard dimensions by vessel and tonnage and Table 6.3.12 shows the largest class ship currently operating in the world.

Some distinctive trends on building of cargo ships have appeared in recent years as listed below:

– New cargo ship trend is for specialization and for multiple usage.

– The trend of increasing ship's capacity has been developing by widening the beam rather than increasing the dimensions of their draft and length.

– The height of mast has a tendency to be lower due to the development of radar.

The size of passenger ships has reached maximum size and bigger ships are not expected to be built in future. The mast height has the same tendency as that of cargo ships.

Table 6.3.10 HIGH CLEARANCE IN THE WORLD

Bridge Name	Nation	Completion	Region	River, etc.	Clearance (m)	Waterlevel Dadum	Mainspan length (m)
George Washington	U.S.A.	1931	New York	Hudson River	65	M.H.W.L.	1067
San Francisco Oakland Bay	"	1936	San Francisco	San Francisco Bay	64	M.L.L.W.L.	704
Golden Gate	"	1937	"	"	65	M.H.W.L.	1280
Bronx Whitestone	"	1939	New York	East River	46	"	701
Tacoma Narrows	"	1949	Tacoma		57	M.L.W.L.	853
Mackinac Straits	"	1957	Mackinac	Michigan River	52	"	1158
Walt Whitman	"	"	Philadelphia	Delaware River	42	M.H.W.L.	610
Waka-do	JAPAN	1962	Kita-Kyushu	Dokai-Bay	40	H.W.O.S.T.	367
Verranzano Narross	U.S.A.	1964	New York	Hudson River	63	M.H.W.L.	1298
Forth Road	U.K.	"	Edinbrough	Forth River	46	H.W.O.S.T.	1006
Severn	"	1966	Bristol	Severn River	37	M.H.W.L.	985
Salazar	PORTUGAL	1966	Lisbon	Tagus River	70	M.W.L.	1013
Kanmon	JAPAN	1972	Kita-Kyushu	Kanmon Strait	61	M.W.O.S.T.	712

Table 6.3.11 STANDARD DIMENSIONS OF VESSELS

	Tonnage	Length (m)	Beam (m)	Depth (m)	Draft (m)
Passenger Ships	10,000	160	20.6	12.3	8.2
	15,000	181	23.1	13.9	8.8
	20,000	197	25.1	15.1	9.2
	30,000	223	28.2	17.0	10.0
	50,000	252	32.5	19.4	10.7
General Cargo Ships	10,000	144	19.4	11.2	8.2
	15,000	162	21.7	12.7	9.1
	20,000	177	23.4	13.8	10.0
	30,000	199	26.1	15.7	11.0
	40,000	217	28.3	17.2	11.9
	50,000	232	30.0	18.4	12.7
	60,000	245	31.3	19.4	13.4
	70,000	257	32.6	20.3	14.0
Oil Tankers	10,000	139	19.0	9.9	8.1
	15,000	157	21.7	11.3	9.0
	20,000	171	23.8	12.4	9.8
	30,000	194	27.2	14.1	10.9
	40,000	211	29.9	15.4	11.7
	50,000	226	32.1	16.5	12.5
	70,000	250	35.9	18.4	13.6
	100,000	270	39.0	19.2	14.6

Table 6.3.12 LARGEST CLASS SHIPS IN THE WORLD

Vessel Type	Name	Tonnage (ton)	Length (m)	Beam (m)	Depth (m)	Draft (m)	High from Draft (m)
Passenger ships	S.S. Norway	69,400	316	33.7		10.5	
	Queen Elizabeth II	67,100	294	32.0		10.0	53.8
Bulk carriers	World Gala	287,000	338	54.6		21.9	54.0
	Seiko Maru	248,000	326	52.6		20.5	66.1
RO/RO ships	Barber Toba	32,000	229	32.2	20.2	10.8	48.2
Full Containers	Liverpool Bay	47,300	290	32.3	24.6	11.0	42.7
Oil Tankers	Seawise Giant	239,000	458	60.8		24.6	
	Nissei Maru	239,000	379	62.0	36.0	28.2	
Aircraft Carriers		89,600	342	40.5	10.9		
Sailing ships	Juan Sebastian de Elcano	3,750	107	13.1			

3) Existing Situation of Mombasa Port

(1) Physical limit of the port

The Mombasa Port consists of natural deep creeks completely sheltered from the open sea. The harbour entrance is over coral formations which impose a limitation on the draft and length of ships which can enter. The draft limit was increased to 45 feet (13.7 m) by United States of America in January, 1983. The length limit is due to the sharp and narrow bend of the natural approach into Kilindidini Harbour. This limits ship lengths to 250 m. Due to the narrowness of the entrance only single way traffic is possible with compulsory pilot, although inside the harbour ships can pass each other in most areas.

Based on the description made above, the following is the summarized limits of the port entrance.

Channel width : 1,000 ft (300 m)
Channel depth : 45 ft (13.7 m)
Length of vessels : 250 m
Operation : Single way traffic with compulsory pilot

(2) Existing port berth and draft

Existing berth conditions are summarized based on the port statistics in 1981 as follows:

Deep water berths	:	Number	:	16
		Total length	:	3,044 m
		Max. length	:	184 m
		Draft	:	10 m
Bulk oil jetties (Tanker berths)	:	Number	:	2
		Draft	:	13.4 m
Container berths	:	Number	:	1
		Length	:	230 m
Bulk cement berths	:	Number	:	2
		Total length	:	315 m
		Number of silos	:	3
		Capacity per silo	:	6,000 ton

4) Future Prospect of the Port

(1) Future port traffic

The future port traffic including existing traffic is shown in Table 6.3.13.

Table 6.3.13 PORT TRAFFIC

Unit: 1,000 DW ton

Year	1980	2000
Import & Export		
Dry bulk	1,231	3,710
Liquid bulk	4,066	860
Others cargo	2,215	6,355
Total	7,512	10,925

Source: Annual Bulletin of Port Statistics 1981

In 1980 the total imports amounted to 2.6 times that of exports. In the future the total cargo handling in the port is expected to increase. Liquid bulk is forecast to be drastically reduced, but dry bulk and other cargo will increase upto 3 times as much as that of 1980.

(2) Arrival Ships

In 1980 ship (dry cargo deep sea and coasters only) monthly arrivals averaged 87 ships.

Table 6.3.14 shows some data on arriving ships and future ship sizes.

Table 6.3.14 ARRIVING SHIP SIZES

(Unit : DW. ton)

Arrival Ship / Type of ship	Past most frequent ships	Past biggest ships	Expected ships (1990)	Possible ships
General cargo ship	10,000	20,000	20,000	50,000
Container ship	20,000		40,000	50,000
RO/RO ships	15,000		40,000	40,000
Bulk carriers	15,000		40,000	40,000
Oil tankers	20,000	50,000	50,000	65,000
Passenger ships	-	67,000		

5) Height of Mast

Fig. 6.3.8 shows the relation between gross tonnage (GT) and the height of ship masts. It is possible to determine the prevailing mast height if the gross tonnage could be determined although the actual values varies greatly.

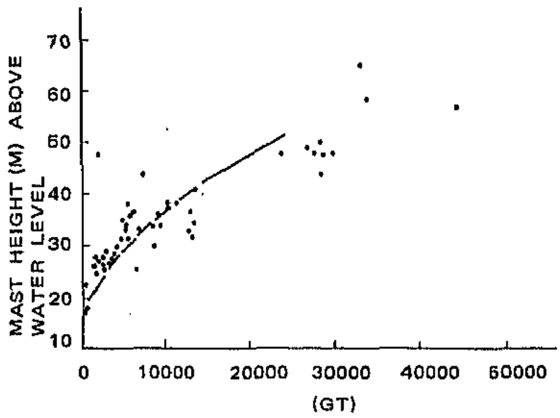


Fig. 6.3.8 RELATION BETWEEN GROSS TONNAGE AND HEIGHT OF SHIP MASTS

In general the mast height becomes lower in the order of warship, passenger ship and cargo ship as shown in Table 6.3.15.

It is very difficult to obtain the correct data on the mast heights.

Table 6.3.15 HEIGHT OF SHIP

Ship Type	Name of ship	DWT	Total Height from Draft (m)
Warships		89,600	70
Passenger ship	Queen Elizabeth II	67,000	53.8
	Cambera	45,000	57
	Caronia	38,000	59
Cargo ship	Mining ship	50,000	42
		20,000	38
RO/RO ship	Barder Toba	32,000	48
Container ship	Liverpool Bay	47,000	43
Tanker	Tokyo	95,000	47
	Sansui	62,000	46
	Borgila	52,000	40

6) Establishment of Alternative Navigation Clearance

Total three alterntive navigation clearance were established for the project road study, namely 73.2 m, 55 m and 45 m above the highest high water level based on the collected data.

The 73.2 m clearance is the basic requirement of this project and accommodates the maximum height of all conceivable operating vessels. The 55 m is the assumed clearance required to fully maintain the commercial and tourist function of the harbour. The clearance of 45 m can allow passenge of almost all the commercial vessels which have arrived in the past.

7) Practical Navigation Clearance

The navigational clearance in this Study was established as a rectangle section with 330 m lateral clearance as shown in Fig. 6.3.9. In practice the tide variation and the crown allowance at the centre of the crossing bridge should be considered.

The crown allowance is estimated to be 1.77 m above the navigation clearance. The tide varies with average monthly maximum of 3.8 m and other tide variations are shown in Table 6.3.16.

Loading conditions of ship, ship's trim by wave and longitudinal movement of bridge by wind, are also other variation factors for the practical clearance. It is very difficult to estimate accurately since these occur independently. If the value due to these factor is one meter, the available clearance for passing ship can be estimated in Table 6.3.16.

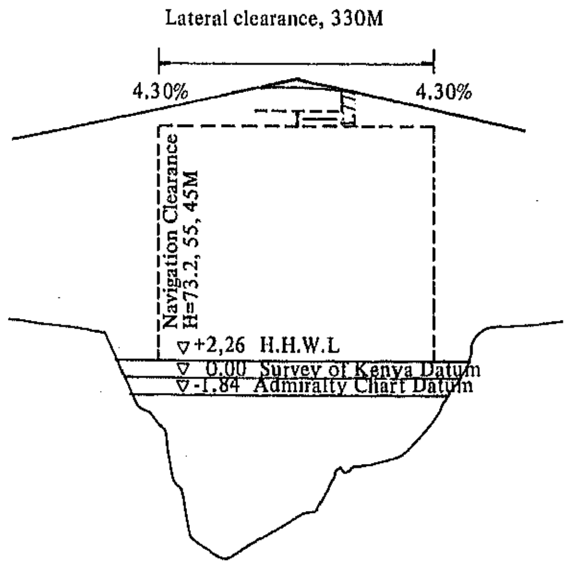


Fig. 6.3.9 PRACTICAL CLEARANCE FOR SHIP'S PASSAGE

Table 6.3.16 TIDE IN KILINDINI HARBOUR

	Tide on Admiral Chart Datum		Sea Level by Survey of Kenya Datum	Practical clearance	
	Variation			H = 55	H = 45
Highest Tide	3.7 ∿ 4.0	+ 4.10	+ 2.26	55.77	45.77
Mean High Tide	2.0 ∿ 4.0	+ 3.47	+ 1.63	56.37	46.37
Mean Sea Level (Survey of Kenya Datum)	-	+ 1.84	0	58.03	48.03
Mean Low Tide	0.0 ∿ 1.8	+ 0.27	- 1.57	59.60	49.60
Admiralty Chart Datum	-	0	- 1.84	59.87	49.87
Lowest Tide	-0.1 ∿ +0.2	- 0.03	- 1.87	59.90	49.90

Source: TIDE TABLES FOR EAST AFRICAN PORTS, KENYA PORTS AUTHORITY

The minimum sea depth, 45 Feet (13.7 m) is available for full width of lateral clearance (330 m) at any tide. The available clearance at the centre of the bridge and mean-low-water level is assumed as 59.6 meter for 55 navigation clearance and 49.6 m for 45 m clearance.

It can be understand from Table 6.3.15 that the clearance of 55 m accommodates full commercial and tourist ship's passage in the harbour and the clearance of 45 m maintains full commercial ship's passage.

6.3.6 Alternative Access Study on Mombasa Island

1) General

The Likoni Crossing Project Spans across the Kilindini Harbour, and connects the South Mainland and the Mombasa Island. The project does not aim at a thorough solution of traffic problems on the island. Its impact on the traffic situation is however immersurable.

Traffic problems on the island are analysed by using forecasts presented in Chapter 5 and in Para 6.4 in Chapter 6.

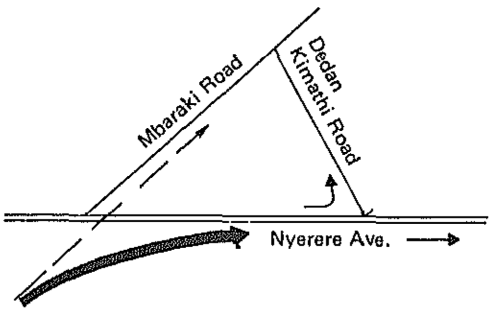
The conceivable access patterns have been studied with navigation clearances H = 73.2 m, 55 m and 45 m, based on the recommended crossing bridge alignment.

Satisfactory traffic distribution is an important requirement of the Project road. Traffic to and from the Project road requires to be properly managed throughout all stages of the project implementation.

2) Access Point Location and Traffic Distribution Characteristics

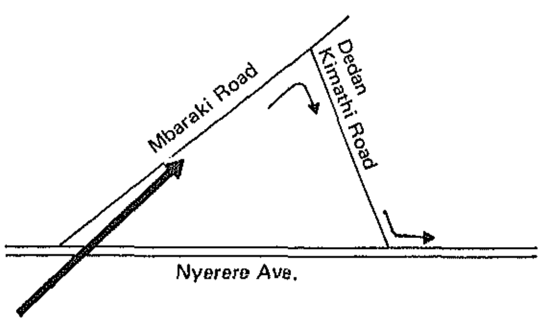
Three alternative locations of access point to the existing road on the Island are conceivable. Each of these has its characteristic functional feature in traffic distribution on the Island, as summarized below.

(1) Access point featuring strong connection with CBD



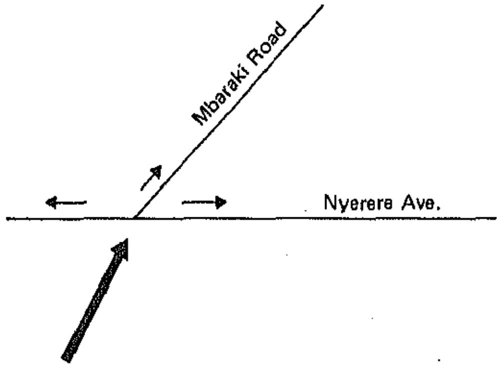
Nyerere Ave. is the direction of the major traffic and the project road terminates at Nyerere Ave. The traffic directed to Arch. Makarios Road turns at the next intersection.

(2) Access point featuring strong connection with future trunk road



Mbaraki Road is a major road at which the project road terminates. The traffic to CBD turns at the next intersection.

(3) Access point featuring good traffic distribution in both directions of Nyerere Ave and Mbaraki Road



The project road terminates at the intersection of Nyerere Ave. – Mbaraki Road.

3) Basic Considerations in the Access Study

(1) Traffic Distribution Performance of the Project Road

Table 6.3.17 shows traffic distribution patterns of the Project Road "with the without improvement" of existing road network, based on traffic forecast presented in Chapter 5.

Pattern I represents the case without improvement of existing roads, and pattern II represents the case with improvement, including 4 lane expansion of Archibishop Makariios Road, a flyover construction behind the Mombasa railway station and the construction of Inner Ring Road (4-lanes).

In the case of pattern II, a considerable portion of the crossing traffic in 2010 will divert to the Mbaraki Road (the direction of future trunk road) thus reducing the traffic load on Nyerere Ave. toward CBD.

Table 6.3.17 FUTURE DISTRIBUTION PATTERN OF PROJECT ROAD TRAFFIC

(Unit: Percentage)

Pattern \ Year	1983	1990	2000	2010
I. Without Improvement of Existing Road Network	Mbaraki Road 75	50	33	25
	Nyerere Ave. 25	50	67	75
II. With Improvement of Arch. Makarios Road (4-lanes), Fly-over behind Mombasa Station and Inner Ring Road (4-lanes)	Mbaraki Road -	-	-	85
	Nyerere Ave. -	-	-	15

(2) Future Traffic Situation on the existing arterial roads without Improvement

As studied in Para 6.4 the number of congested sections in 1990 is forecast to be limited. But these congested sections will increase continuously in number and total length, so that most of the major sections of existing arterial roads will be heavily congested by 2010.

In 2000 Nyerere Ave. and Mbaraki Road (except two intersections of Dedan Kimathi and Mnazi Moja St.) will still have some allowance to receive additional traffic to and from the project road.

(3) Traffic Analysis for Throughway and Adjacent Intersections

As analized in 6.2.1 (Traffic Capacity) and 7.3.2 (Intersection) the number of throughway lanes and the adjacent intersections (Nyerere Ave. – Mbaraki Road and Nyerere Ave. – Dedan Kimathi Ave.) are proposed as follows:

- The project road can satisfy the traffic demand in 2001 with two lanes, and expansion to 4-lanes will be required after the year 2001.
- Nyerere Ave./Dedan Kimathi Road Intersection
  Traffic here can be satisfactorily controlled by means of signals up to the year 2001 and a fly-over will be required for the road leading to CBD in 2010.
- Nyerere Ave./Mbaraki Road Intersection

  Traffic here can be satisfactorily managed by signals up to the year 2001, after which an extension to Mbaraki Road will be required.

(4) Control points and land use plan in the area

As described in 6.3.4 (Alternative Alignment Study for Bridge Crossing) private catholic school, located east of the Nyerere - Mbaraki intersection, being of historical value is an object of conservation, and therefore to be shown in planning the layout of the project.

The hinterland of Mbaraki berth, the area surrounded by Mbaraki Road, Nyerere Ave. and the berth, will be developed as an industrial area producing soda ash, etc. Raod extension into this area will not creat any significant environmental problem.

4) Planning Policy

(1) Basic Principles

During the Study, the following principles are adopted in planning the layout of the access.

- Traffic using the project road should be accommodated by the existing road network until the future trunk road is improved.

- The access road should be extended up to the future trunk road, as proposed in the Progress Report II and in para 6.3.2.
- Traffic associated with CBD will remain to be the major component of traffic accommodated by the project and should therefore be treated accordingly.
- To minimize the investment cost, construction should be staged.

Furthermore some minor points on the access alignment are found as listed below:

- The access road at the initial stage should be effectively used in all later stages.
- Even though the bridge crossing is of two separated structures, each carrying unidirectional, traffic lanes its end section of the Likoni side should merge to form a simple 2-way 4-lane roadway of an integral cross section in the final stage.
- Longitudinal slopes of the bridge access ramp should be 4.3% at maximum.
- The alignment should be as simple as possible for the convenience of users and the ease of staged construction.
- The cross section of the main bridge should desirably be symmetrical.
- Extra construction of access roads should be minimized throughout the stages of construction.
- The minimu radius for a loop access road should be 120 m (absolute minimum value of 60 Km/hr) in the initial stage.

(2) Staged construction of the access

Future traffic demand on major arterial roads varies to a large extent depending on improvements to the trunk road. In the case of without improvement, trips connected with the CBD constitutes the major portion of traffic flows. Nyerere Ave., which is expected to carry such traffic to and from the Project road, will still have some allowance to receive additional traffic in 2000. Among the three previously stated access patterns (6.3.6-2)) therefore, the pattern which involves the access to Nyerere Ave. and the intersection of Nyerere Ave. and Mbaraki Road is most preferable from traffic service and distribution points of view.

For the final stage the extension to Mbaraki Road is justified from the traffic planning point of view for the following:

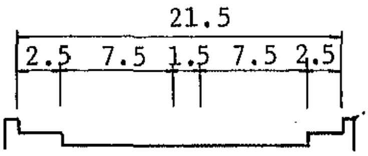
- The Project road will become a part of the trunk road connecting Nairobi and Tanzania.
- In 2010, traffic jams will prevail on major arterial roads centering on the CBD. In order to enhance the urban function the introduction of a new trunk road will be required.

(3) Alternative Cross Section of Main Bridge

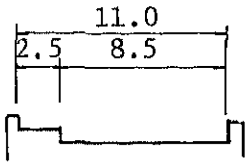
The construction and usage of the main road cross section at the initial stage vary in accordance with the access patterns and the staging process.

Three alternative cross sections can be considered for the main bridge in the initial stage. They are: A, cross sections with full width including footpath on both sides; B, 2-lanes carriageway with one side footpath and C, 4-lanes carriageway without footpath.

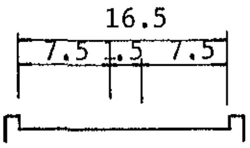
A. 4 lanes + Footpath



B. 2-lanes + One side footpath



C. 4 lanes without Foot path



In Alt.-A the full width is constructed during the initial stage. In Alt.-B two lanes are constructed during each construction stage. A distance of several metres is required between the two bridges to allow room for future tower construction.

Alt.-C is a variation of the above two alternatives with greater structural safety against wind load. Pedestrians are to use the outer lane of the carriageway at the initial stage. To provide continuous footpath throughout the bridge approach, provision of some extra footpath is required at the connection between the main bridge and approach bridge during the final stage construction. Additional footpaths are also required in the final stage project roadway.

The total cost of Alt.-A (main bridge) is estimated to be 70 ~ 75% cheaper than that of Alt.-B.

Therefore Alt.-C has not been raised for comparison.

5) Access Patterns by Alternative Navigation Clearance

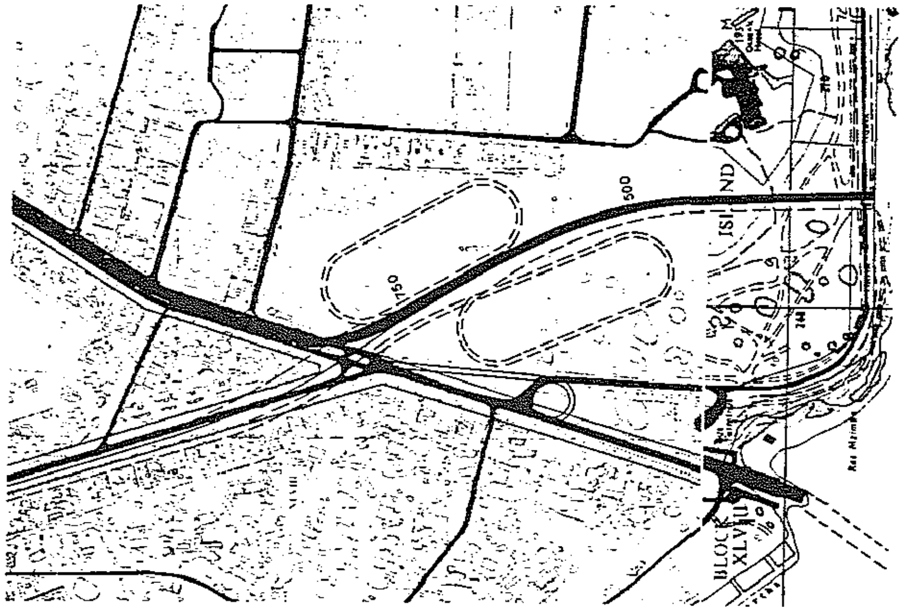
A number of alternative access patterns was examined during Phase II Study. As described in 6.3.5 with Alternative Navigation Clearances considered. Following is a summary of major points analysed.

In order to obtain a general idea four alternatives have been studied for the case of 55 m navigation clearance. These are as shown in Fig. 6.3.10 and Table 6.3.18.

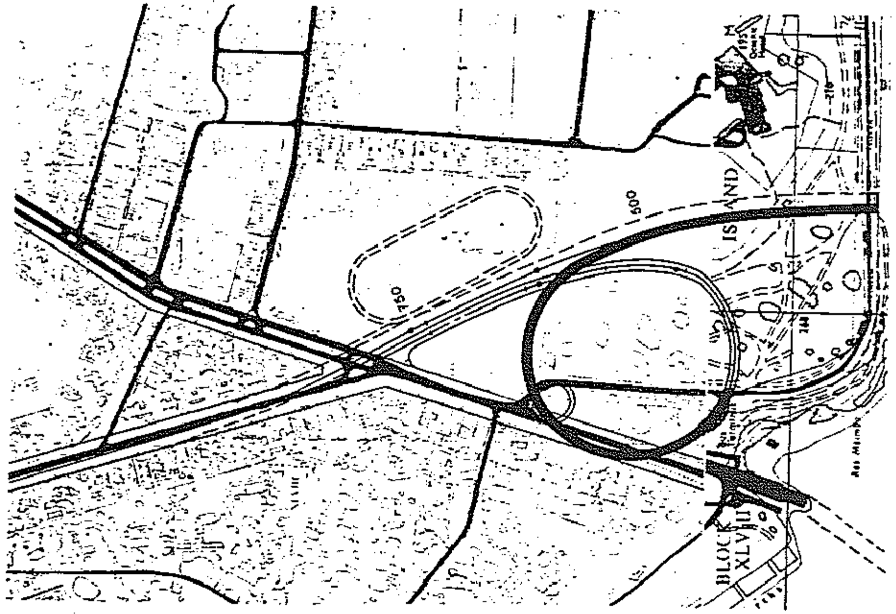
Considering the environment of the area and the requirement to maintain the private catholic school, the alternatives involving a loop ramp at the eastern side of the Project road are not acceptable. As a result two alternatives, A and D remain and modified plans of these are shown in Fig. 6.3.11 and 6.3.12.

Fig. 6.3.10 ALTERNATIVE ACCESS LAYOUT

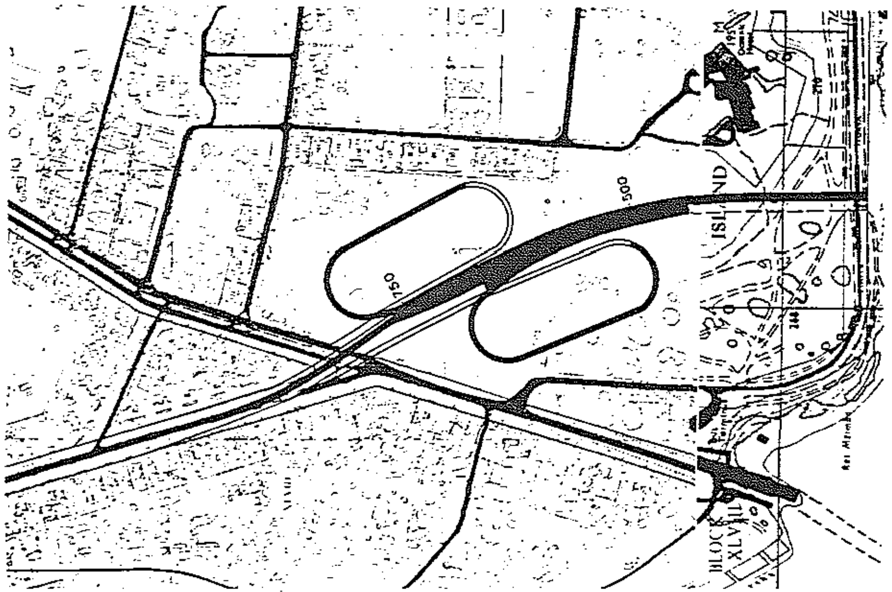
ALTERNATIVE-A



ALTERNATIVE-C



ALTERNATIVE-B



ALTFRNATIVE-D

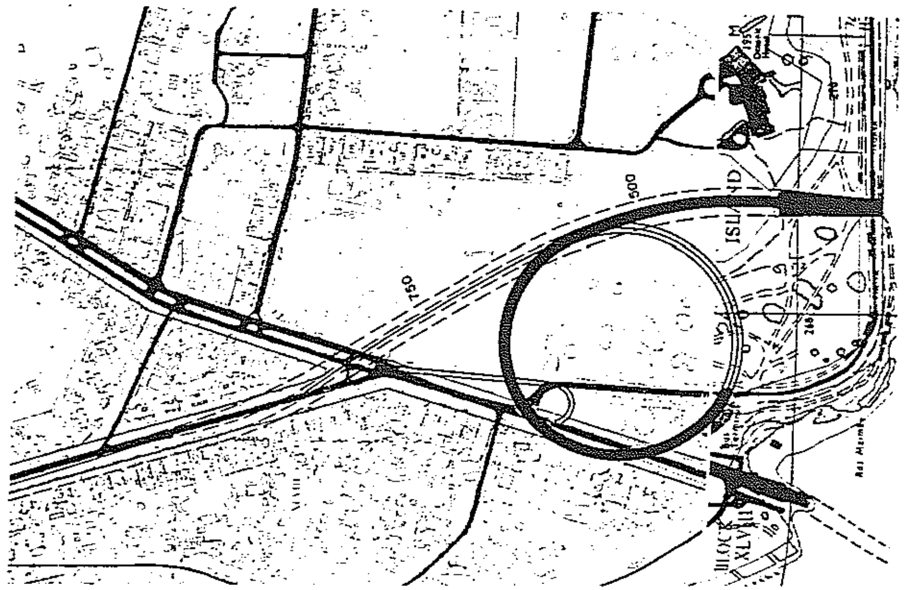


Table 6.3.18 ACCESS ALTERNATIVES

Item \ Alternative	ALT - A		ALT - B		ALT - C		ALT - D	
	Initial Stage	Final Stage	Initial Stage	Final Stage	Initial Stage	Final Stage	Initial Stage	Final Stage
1) Plan	Acess to Nyerere Ave.	- Two loops added - 4 lanes expansion	Access to Mbaraki Road and intersection	- 4 lanes expansion	Access to intersection	- 4 lanes expansion - One loop road added	Access to intersection (Centre ramp system)	4 lanes expansion
2) Quantity								
- Bridge Crossing	415^M	415^M	415^M	415^M	415^M	415^M	415^M x 2	-
Main Approach W = 10.5M	605	805	840	370	625	1,045	673	1,670
Ramp W = 5.5M	-	946.5	946.5	-	-	460.5	-	-
- Intersection	Nyerere/ Dedan Kimathi Intersection	Nyerere/ Mbaraki Intersection	Nyerere/ Mbaraki Intersection	-	Nyerere/ Mbaraki Intersection	Further improvement	Nyerere/ Mbaraki Intersection	Further improvement
- Street W = 10^M	710^M	620^M	$1,260^M$	-	-	620^M	-	$1,500^M$
- Land Aquisition	2,130 M^2	11,000 M^2	11,000 M^2	-	-	11,000 M^2	-	5,700 M^2
3) Traffic Service	Long way to Mama Ngina Drive traf-fic	Full service	Full service	Full service	Full service	Full service	Full service	Full service
4) Land use/ Environment	Residents school	Problem to primary school	- Residence - Problem to primary school	-	Park	Problem to primary school	-	Residence
5) Bridge Cost (Million Shs.)	392.5	510.7	550.9	331.3	406.6	505.4	539.2	348.5
Total	903.2		882.2		912		887.7	

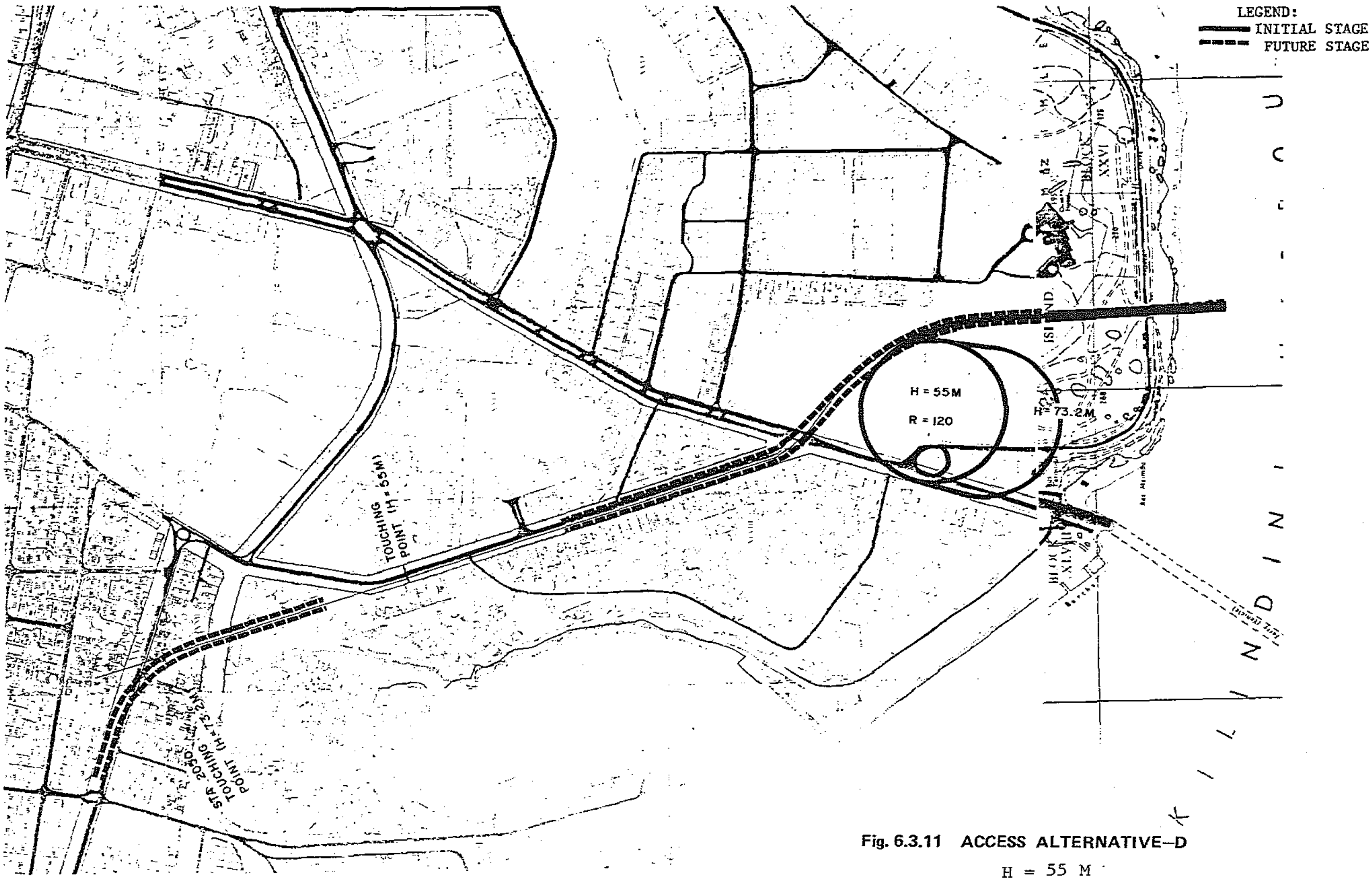


Fig. 6.3.11 ACCESS ALTERNATIVE–D

H = 55 M
H = 73.2 M

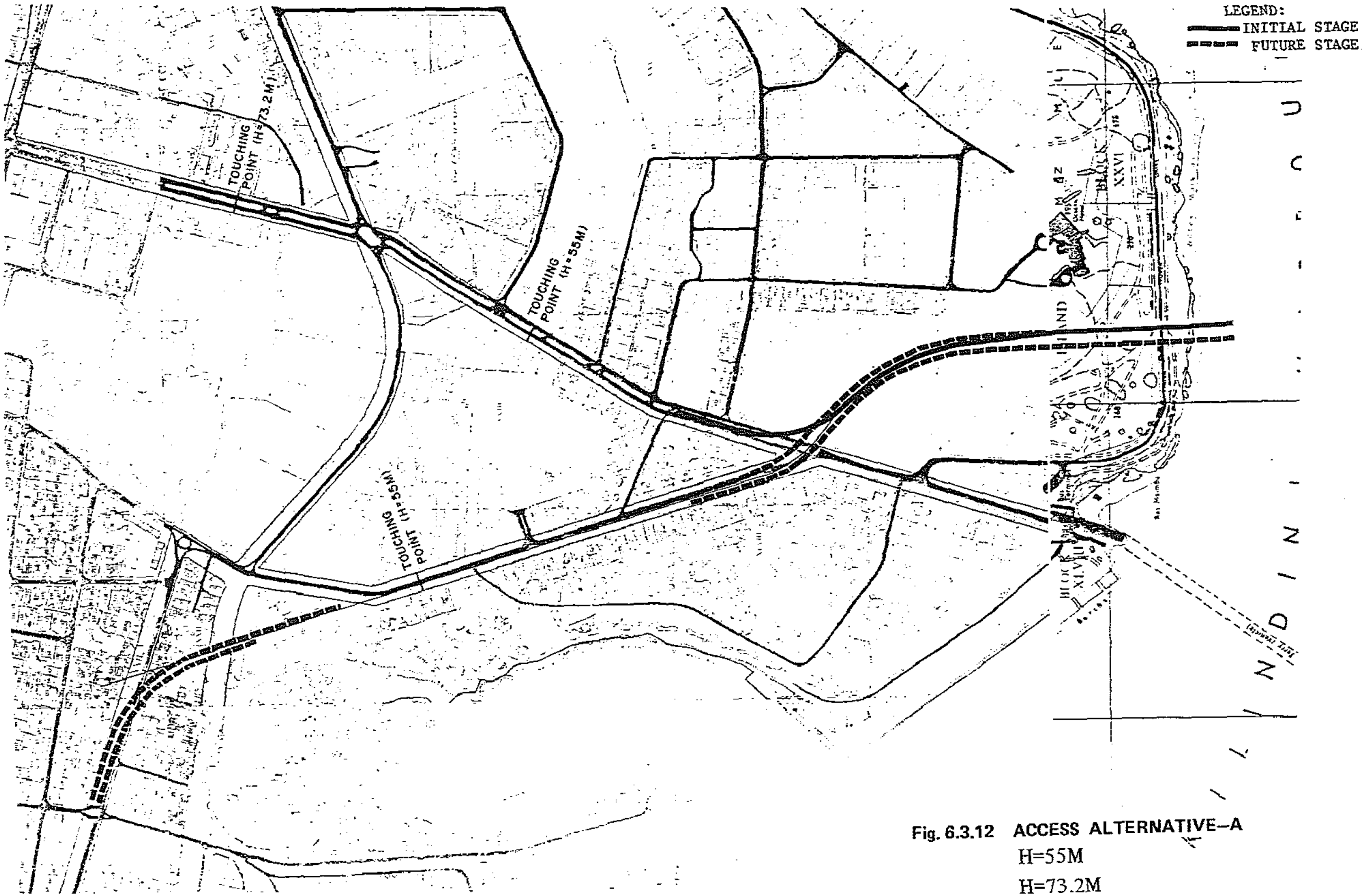


Fig. 6.3.12 ACCESS ALTERNATIVE–A
H=55M
H=73.2M

The characteristics of the two viable alternatives are described below:

Alt.-A: – Access to the major traffic direction (Nyerere Ave.) at the initial stage.

– The traffic to the port and industrial area and the West Mainland must detour via the existing roads.

Alt.-D: – Access to the intersection of Nyerere Ave. and Mbaraki Road irrespective of road improvements.

– The traffic service for all directions can be done through the access intersection.

Comparing the major features of Alternative-A and D (Navigation clearance H = 55 m) as shown in Table 6.3.19. Alternative-A is seen to be better between the two. The advantages it has over Alternative-D are as follows:

– Better traffic service

– Both alternatives can be extended to meet the future trunk road.

– Does not involve problems of small curve radius as in the case of Alternative-D which has 120 m curves.

– Cheaper bridge cost

(1) H = 73.2 M

Generally to construct the project road with a higher clearance of 73.2 m, it requires longer access roads and has many disadvantages such as unsatisfactory traffic distribution, heavy project scale, undesirable environmental impacts etc.

In the case of a direct access provided in the Nyerere Ave. during the initial stage, the access point falls beyond the intersection of Nyerere Ave. and Dedan Kimathi Road, and traffic distribution at this end cannot function satisfactorily. Therefore a loop ramp is required to provide access at the interection of Nyerere Ave. and Mbaraki Road instead.

Sufficient open space is found available in this area to accommodate the required large loop.

Other alternative connections to the crossing bridge using a loop access have also been compared. These include case of two-lane and four-lane centre ramp access structures.

As a result these alternatives have not much difference in cost due to the long ramp way required to accommodate the traffic between the South and North bound in the final stage.

The centre ramp system also can accommodate smooth traffic flow (vehicle and pedestian) on the crossing bridge.

Finally the centre ramp system is therefore selected for the case of 73.2 m clearance.

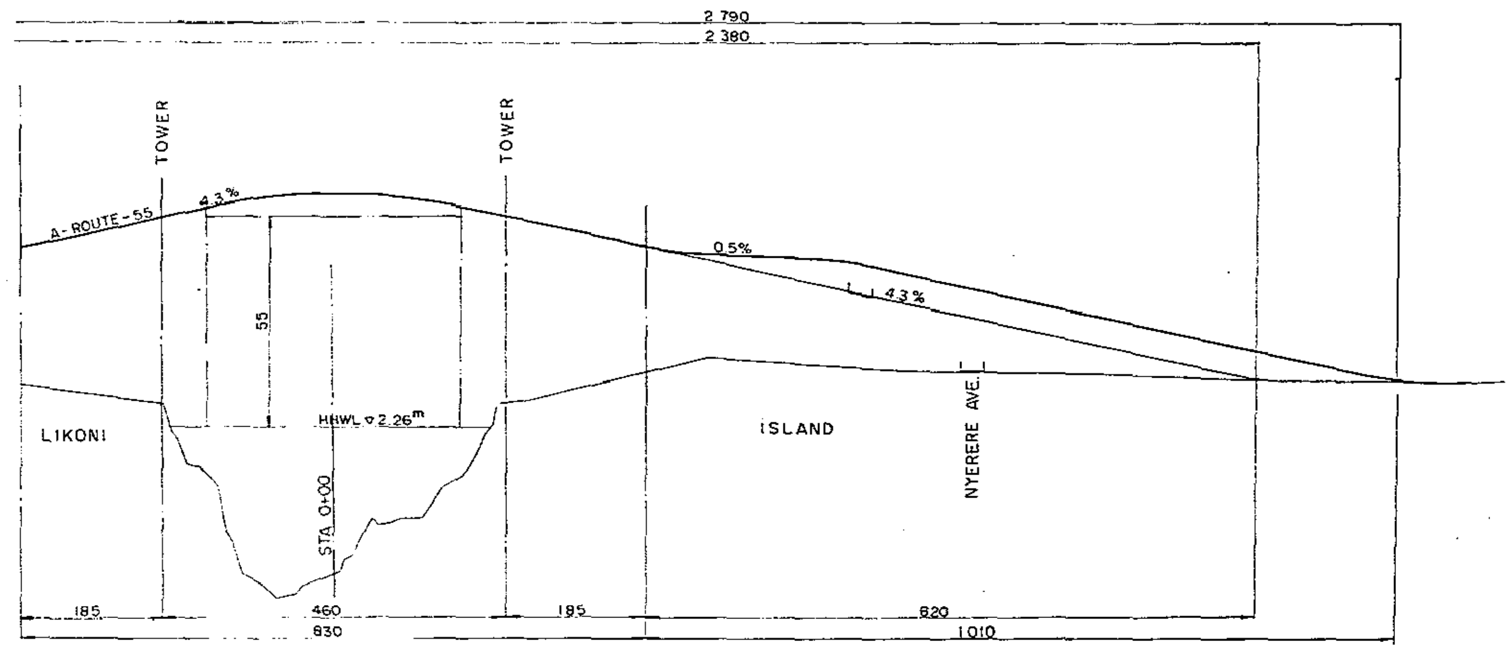


Fig. 6.3.13 ALTERNATIVE PROFILE

SCALE H = 1 : 10,000
V = 1 : 2,000

Table 6.3.19 COMPARISON OF ALTERNATIVE ACCESS (NAVIGATION CLEARANCE H = 55 m)

Item \ Alt.	Alt.-A	Alt.-D
1) Traffic Service	Partial traffic service with detour	Full traffic service
Vehicle Running Distance	Short distance in both directions, : 855 m to Mbaraki Road/Dedan Kimathi Intersection	Long distance in both directions, : 1,624 m to Mbaraki Road/Dedan Kimathi Intersection
Pedestrian Walking Distance	Short distance in both directions : 1,010 m to the above intersection	Long distance in both directions : 1,730 m to the above intersection
Traffic Desire	Access to major traffic direction	Access to both traffic directions
2) Traffic Congestion in year 2000	Congestion in Nyerere Ave.	Congestion at Mbaraki Road/Dedan Kimathi intersection
3) Environ and Landuse		
Initial stage	Residents, school	Park
Final stage	Residents, Industry (plan)	Residents, Industry (plan)
4) Adoptability of Higher Navigation clearance (H = 73.2 m)	Access point falls beyond the intersection of Nyerere Ave./-Dedan Kimathi Ave. and problems in traffic distribution and in expansion of Nyerere Ave.	Bigger loop access can be adopted
5) Construction		
Existing road		
Initial stage	Expansion of Nyerere Ave. and lane arrangement	
Final stage	Expansion of Mbaraki Road and lane arrangement	Expansion of Mbaraki Road and lane arrangement
Intersection	Nyerere Ave./Dedan Kimathi Ave.	Nyerere Ave./Mbaraki Road
6) Bridge Cost (STA 0 ∿ Access point)		
Initial Stage	430	582.7
Final Stage	573.2	406.8
Total (Mil. Shs)	1,003.2(567.0)	989.5(643.2)
	Figures in the brackets show the cost discounted by 10 percent.	

(2) H = 55, 45 M

For both navigation clearance access cannot be provided at the intersection of Nyerere Ave. and Mbaraki Road, but can be made in the Nyerere Ave. During the initial stage, it is desirable from traffic distribution efficiency point of view that the access road be connected with Nyerere Ave. (before the intersection of Nyerere Ave. and Dedan Kimathi Road).

Fig. 6.3.12 shows the concepts of access plans for both clearance. Some variations are also studied. In any case one connection ramp between island bound traffic and South bound traffic is necessary at the final stage.

In case of using loop ramps for the connection to Nyerere Ave. at the final stage, extra area for the loop ramp and compensation for land acquisition along Nyerere Ave. are necessary.

This alternative has no significant advantage over the plan (Fig. 6.3.12) which provides marging and diverging point on the approach bridges to be constructed in the final stage.

The final proposal for the access layout with navigation clearance of 55 m and 45 m, consists of two lanes access to Nyerere Ave. at the initial stage and a separate four lanes extension to Mbaraki Road.

6.3.7 Alternative Main Bridge Study

1) General

This study was conducted based on the determined alignment discussed in sub-section 6.3 and the following results.

- Field reconnaissance
- Centre line profile survey
- Seabed topographical survey
- Geological investigation
- Navigation clearance (330 m wide)
- Confirmation of fairway signs and submarine electric cable locations
- Studies on probabilities of project execution problems.

Prior to the selection of the alternative types of the main bridge, the main span over the channel is studied.

2) Study for the Bridge Main Span

Generally speaking, determination of a main span length is the most important point in bridge planning, as it has a large influence over the scale and type of bridge structure, construction cost, difficulty of construction, etc. Experiences show that the shorter the span length is, the more economical is the construction cost. To the project, however, this will not always be applicable, since the shorter main span length requires the erection of main towers in the sea, which requires a huge amount of cost. The provision of the main towers near the navigation boundary (as shown in Fig. 6.3.14) will also need preventive countermeasures against collision by ships, requiring costly facilities for that purpose.

Studies are therefore made for the problems that may arise from the determination of the main span. The types of the bridge assumed are a suspension bridge and a cable-styed-bridge, because other types of bridge will not suite to the navigational requirements as discussed later in this Report.

The problems that may be encountered are as follows:

– The slopes of the sea-bed exposed on both sides of the channel are comparatively steep, and the channel is considerably deep. Therefore there would be a possibility that ships will happen to navigate near the sea shore line.

– In the case where the main towers are constructed in the sea with a minimum extra-clearance to the navigation boundary, massive foundations for the main towers are required considering collision by ships. Rough cost comparison is made between the cases that a cable styed bridge having a main span length of 380 m is constructed with its main tower footings able to resist head-on collision with a 30,000 ton ship running at the speed of 6 knots and that the same type bridge having a main span of 460 m is constructed with its main towers provided on – shore. The result is shown in Table 6.3.20. The construction of the bridge on –shore is much more economical.

Table 6.3.20 COMPARISON BETWEEN FOOTINGS ON – SHORE AND IN THE SEA (In the case of P.C. Cable-Stayed Bridge)

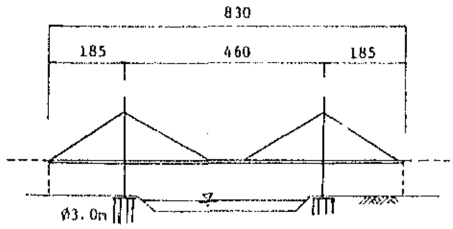
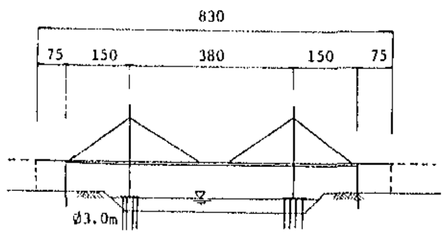
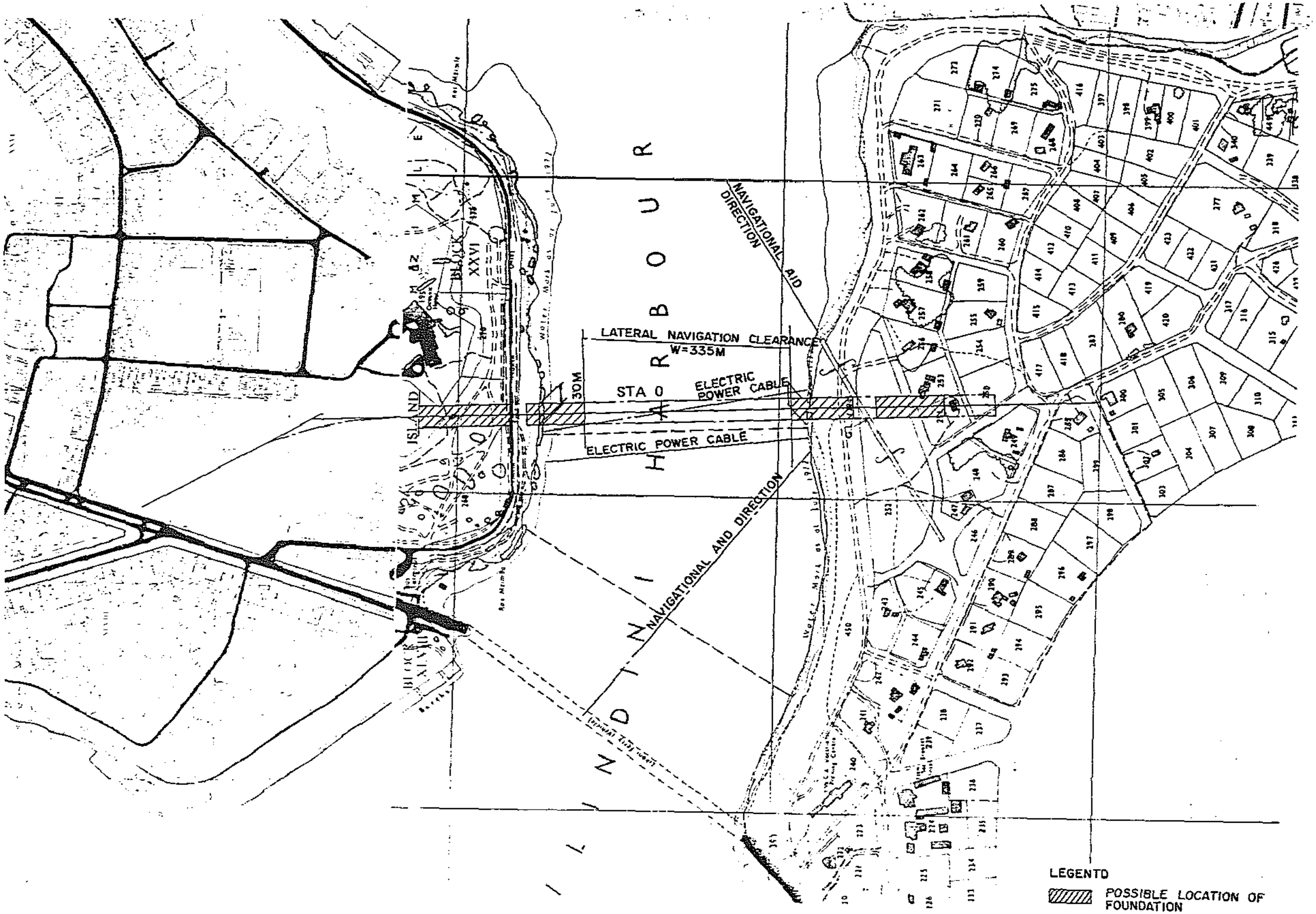
	Footing on-shore	Footing in the Sea
Side View		
Cost of Superstructure	455 million K.Shs	411 million K.Shs
Cost of Substructure	245 "	519 "
Total	700 "	930 "

Fig. 6.3.14 POSSIBLE LOCATION OF FOUNDATIONS FOR MAIN BRIDGE



– Apart from the above, there is a method to provide appropriate space between the structure and the protection. However, this method will require the fender system having the same strength as mentioned above and even a longer span length, with the result of higher costs.

– There is another method to prevent the main tower footings, from striking by ships i.e., provision of stones mounted around the substructures. However, since the sea-bed slopes are steep along the coast as mentioned above, stone filling will be very difficult.

– The pile driving and footing construction in the sea are not only very expensive but also difficult to secure the construction accuracy. Furtheremore, safety measures must be taken during the construction to secure the navigation of ships in the channel.

– The mouth of the channel is not wide. The provision of massive main towers in the channel will psychologically affect the steering of the ships sailing in the channel, and because of the large size of the foundations, influence over the tidal current cannot be neglected.

– K.P.A. has strongly requested to avoid the construction of the foundations in the channel.

Taking into account the above problems, the location of the main tower foundations is determined not to be in the sea, but in such location as the bottom of the footings may be above H.W.L., assuring a space for construction activities. The assumed PC cable-stayed bridge would require about 20 piles of 3.0 m diameter for each foundation. If these piles are provided in four rows in the axial direction of the bridge, the length of the main span is estimated as about 460 meters as shown in Fig. 6.3.15.

As a result of the above discussion, the subsequent study is made exclusively for the main span length of 460 meters of both the cable stayed bridge and the suspension bridge.

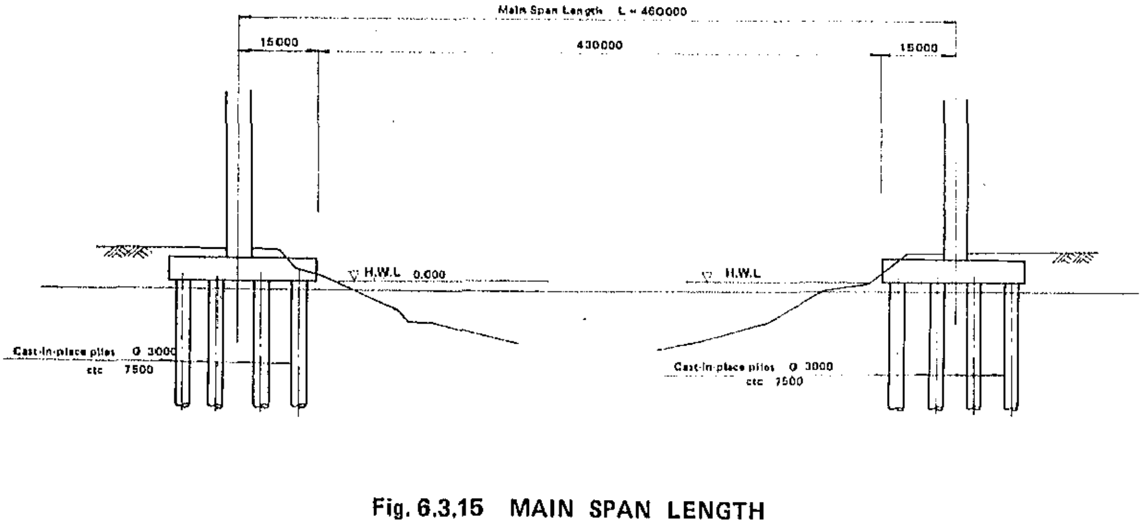


Fig. 6.3.15 MAIN SPAN LENGTH

3) Structural Type of the Main Bridge

The main bridge of this project will be, when constructed, in the top group of the worlds largest bridges. In the selection of its structural type, consideration must be given to the technical and economic aspects, and the past record of world bridge construction as well.

As stated in para. 6.3.7-2), the main span of the main bridge is determined as 460 m long. The current construction technology in the world shows the relation between applicable bridge types and span lengths as shown in Fig. 6.3.16. The long span bridges constructed in the world are plotted by year together with their structural types in Fig. 6.3.17.

As seen from the above figures, four structural systems are applicable to the bridges of about 400 to 500 m long span, namely, suspension bridge, cantilever truss bridge, arch bridge and cable-stayed bridge systems.

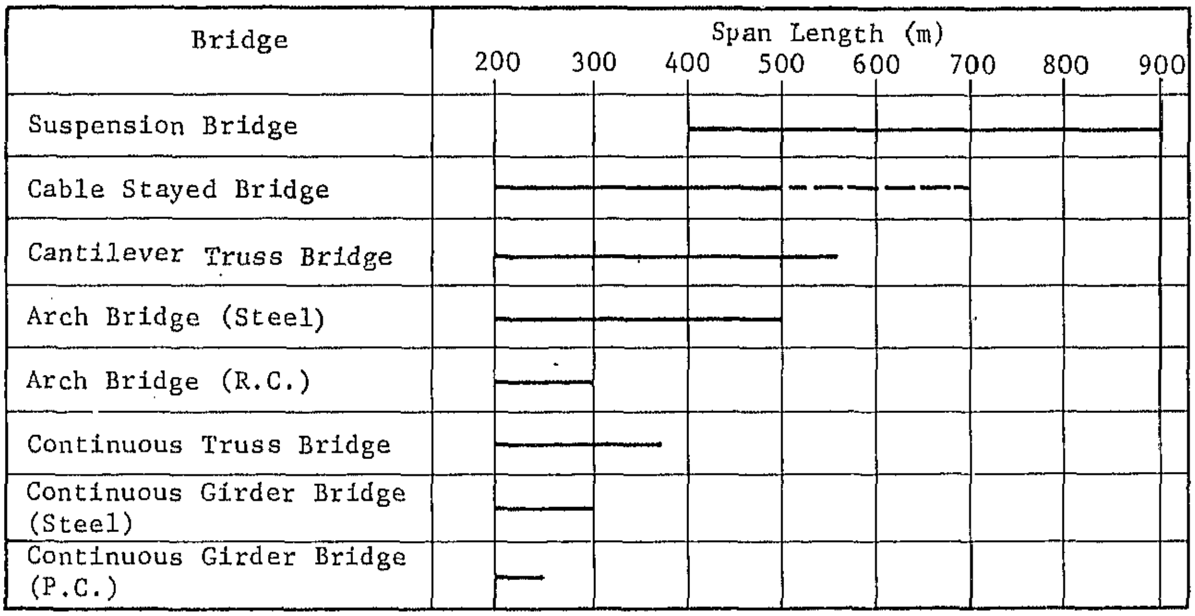


Fig. 6.3.16 APPLICABLE BRIDGE TYPE BY SPAN LENGTH

Suitability of these structural types are analysed on the basis of the comparison to the following points:

- Past records of construction in the world
- Structural characteristics
- Relative ease of construction
- Economic features

For the cable stayed bridge type which can be either steel or P.C. structure, separate comparison is also made as described in the Preliminary Design.

(1) Suspension Bridge

As shown in Table 6.3.21, this structural type has the widest range of application in terms of span length. Particularly for the bridges over 600 m span length, it has a long list of applications, and this means a super system from the technical and economical points of view.

For the bridges the shorter than 600 m, it may not be economically competitive with other systems although it depends on the site conditions.

In the suspension bridge system, the bulk of the load between the towers is transferred through the main cables to the towers which bear the vertical load and to the anchorages which sustain both the horizontal and vertical loads.

As the horizontal strength required for the anchorages is trememdous, this system will not be economically applied to this project as the foundation soils are not sound supporting rock.

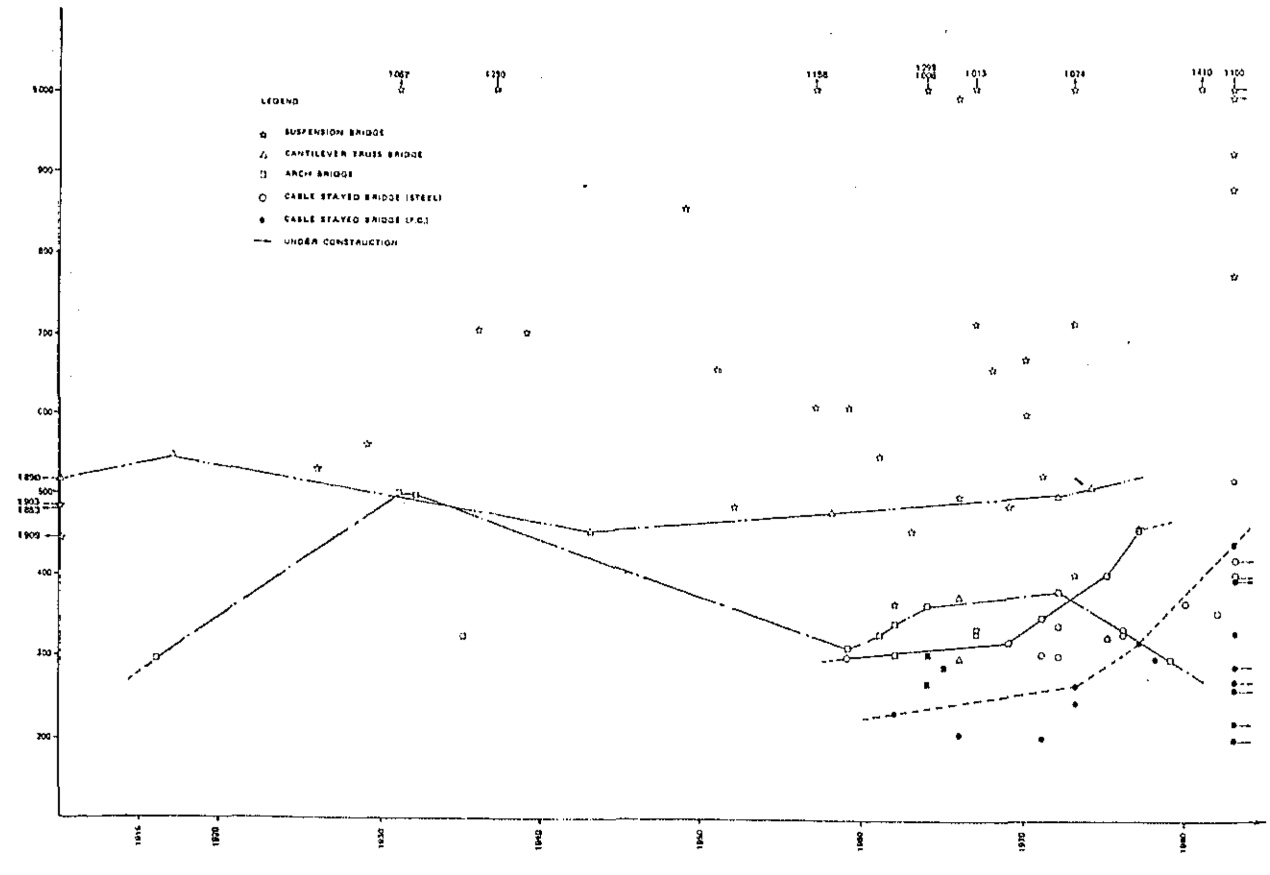


Fig. 6.3.17 LONG SPAN BRIDGE CONSTRUCTED IN THE WORLD BY YEAR

Table 6.3.21 SUSPENSION BRIDGE

Bridge Name	Nation	Main Span Length	Completion
Humber	U.K.	1410 m	1981
Verrazano Narrows	U.S.A.	1298	1964
Golden Gate	U.S.A.	1280	1937
Machinac Straits	U.S.A.	1158	1957
Minami Bisan Seto	Japan	1100	Under construction
Bosporus	Turkey	1074	1973
George Washington	U.S.A.	1067	1931
Salazar	Portugal	1013	1967
Forth Road	U.K.	1006	1964
Kita Bisan Seto	Japan	990	Under construction
Severn	U.K.	988	1966
Shimotsui Seto	Japan	920	Under construction
Oh-Naruto	Japan	876	Under construction
New Tacoma Narrows	U.S.A.	770	1949
In-no-shima	Japan	770	1983
Kam-mon	Japan	712	1973
Orinoco	Venezuera	712	1967
San Francisco Oakland Bay	U.S.A.	704	1936
Bronx Whitestone	U.S.A.	701	1939
Frontenac	Canada	668	1970
Delaware Memorial	U.S.A.	655	1951
2nd Delaware Memorial	U.S.A.	655	1968
Walt Whitman	U.S.A.	610	1957
Tancarville	France	608	1959
Little Belt	Denmark	600	1970
Chesapeake Bay			
Ambassador	U.S.A.	564	1929
Throgs Neck	U.S.A.	549	1961
Benjamin Franklin	U.S.A.	533	1926
Skjomen	Norway	525	1971
Metadi	Zaire	520	1983
Emmerich	West Germany	500	1966
Newport	U.S.A.	488	1969
Chesapeake Bay	U.S.A.	488	1952
Williamsburg	U.S.A.	488	1902
Brooklyn	U.S.A.	486	1883
Hirado	Japan	460	1977
Vincent Thomas	U.S.A.	457	1963
Manhattan	U.S.A.	448	1909
Nan-hai	Korea	404	1973
Waka-do	Japan	367	1962

Structural materials used for this bridge type include steel for stiffening beams and steel or R.C. for the towers.

As shown in Fig. 6.3.18, side spans of this bridge are on land and do not require to be of suspension type structure, and this, if not suspended, can reduce costs to some extent. However, application of different systems in the design of the side spans will reduce the aesthetic merit of the suspension type since the elegant beauty of a basic catenary curve cofinguration will not be emphasized.

In summary, application of the suspension bridge type is not a good choice for this project for the following reasons:

- Tremendously heavy anchorages are required to sustain the horizontal load of the main cables. This invitably raises the total construction cost to a subtantial extent.
- Careful design consideration is required in the stability of the suspended structure under wind load.
- Longer construction period is required as compared to any other structural type, due to the involvement of cable installation work.
- Almost all required materials have to be imported.

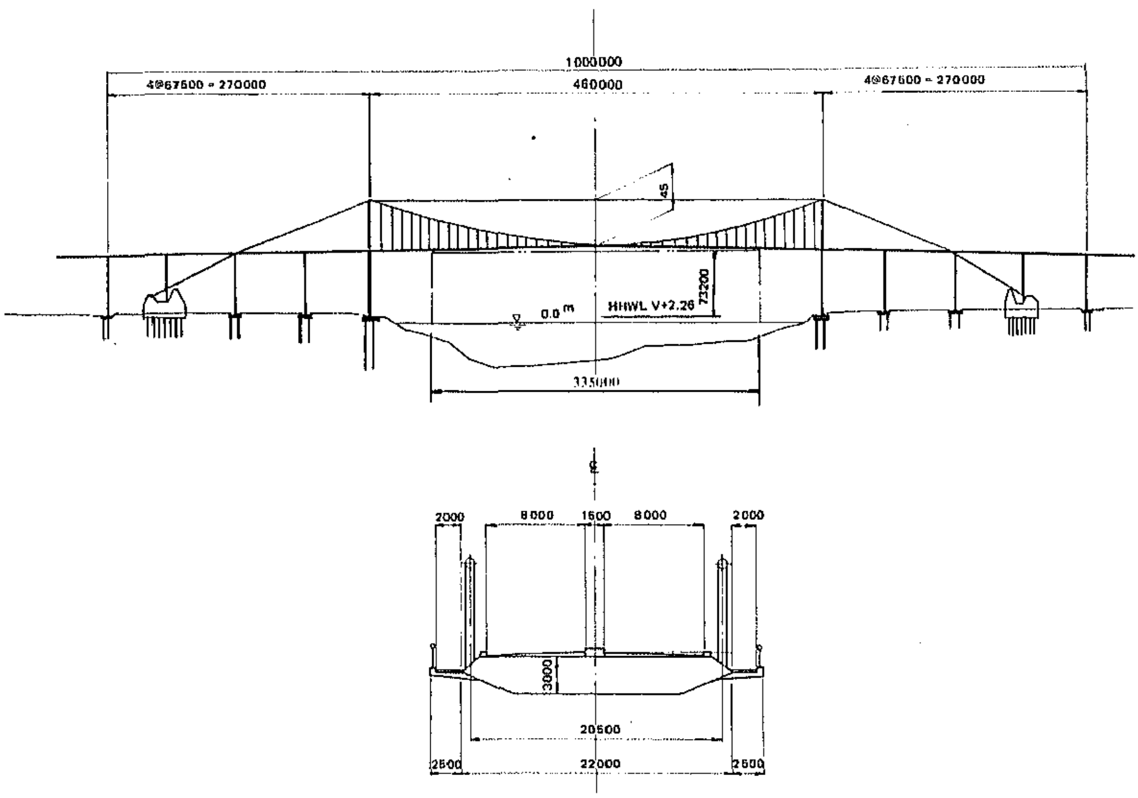


Fig. 6.3.18 SUSPENSION BRIDGE

(2) Cantilever Truss Bridge

Cantilever Truss System can be applied to the construction of long bridges, as it occupies the ranking next to suspension bridges in span length. Some of the longest bridges of this typs in the world are as listed in Table 6.3.22.

For the construction of the bridges with main span length ranging about 400 to 500 m, it is competitive with other structural types.

Table 6.3.22 LONGEST CANTILEVER TRUSS BRIDGES IN THE WORLD

Bridge Name	Nation	Main - span length	Completion
Quebec	Canada	549 m	1917
Forth	U.K.	521	1890
Minato	Japan	510	1974
Delaware River	U.S.A.	501	1972
Greater New Orleans	U.S.A.	480	1958
Howrah	India	457	1943

When applied to this project, the cantilever truss bridge requires a greater main span length than those of a suspension or a cable stay bridge structures.

This structural type, being statically determined, is not sensitive to the displacement of substructure due to nor-uniform settlement and rotation. In addition, it also has the following advantages as compared to others:

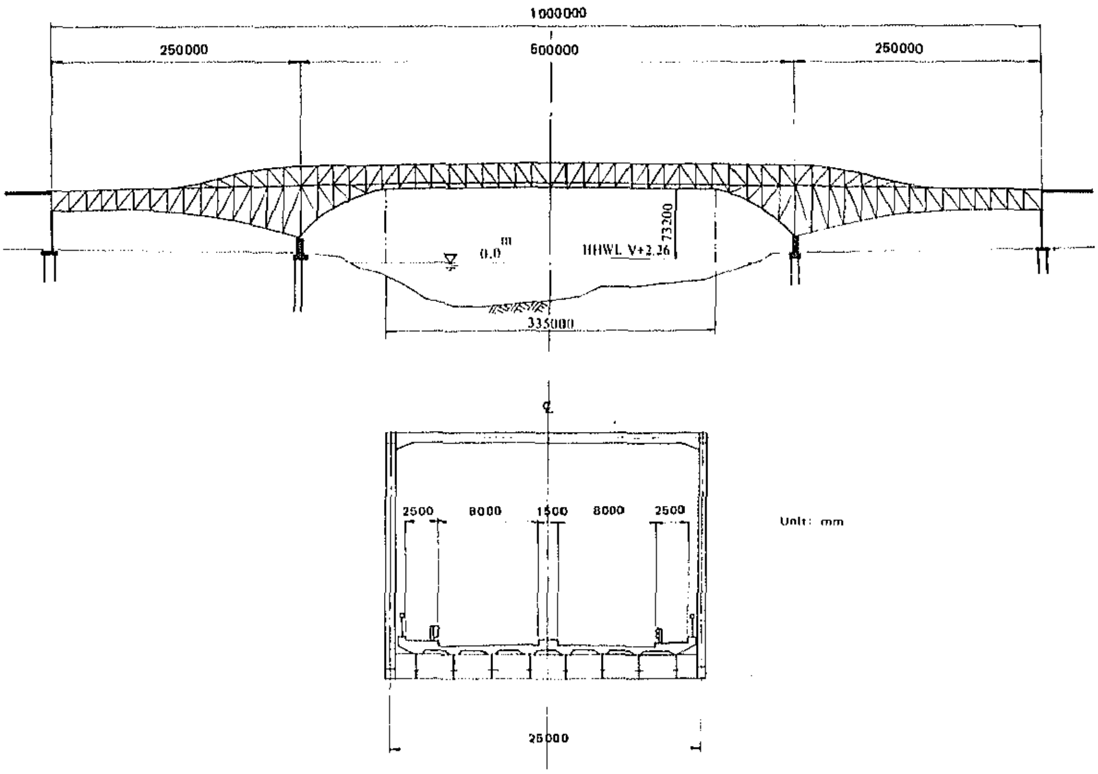


Fig. 6.3.19 CANTILEVER TRUSS BRIDGE

– High rigidity, high wind resistant stability with small deflection

– Less demand for work precision than other structural types and shorter period of construction

– Less influency by temperature changes.

On the other hand it has the following disadvantages that make it unsuitable for the use in the crossing bridge:

– Being constructed almost completely of steel materials which mostly have to be imported

– Biggest painting area due to the largest number of structural members, resulting in the highest maintenance cost

– Least aesthetic merit and unsuitable for the resort area.

(3) Arch Bridge

The records of the long span bridges constructed of this type in the world are as shown in Table 6.3.23.

The type is most economic when the bridges having rise to span ratio of about 1/6 to 1/8. Fig. 6.3.20 illustrates the conceptual outline of the bridge with a 1/6 rise to span ratio.

The arch bridge features the elegant beauty of parabolic configuration, high structural rigidity and wind resistant stability. But it also has the following disadvantages:

– Even though it is considered technically feasible the span length required is greater than any existing arch bridge.

– Great horizontal reaction will constantly work on the arch supports. Since displacement of these supports have significant effects on the overall soundness of the structure, a stable foundation is required for such reaction. This makes the total construction cost uneconomic.

– Because of steel structure, supply of the required material will almost completely depend on imports.

– Construction work is very difficult and the erection equipment cost is expensive.

Based on the above considerations, the arch bridge is not recommended for the crossing bridge of this project.

Table 6.3.23 LONGEST STEEL ARCH BRIDGES IN THE WORLD

Bridge Name	Nation	Main span length (m)	Completion
New River	U.S.A.	518	1976
Bayonne	U.S.A.	504	1931
Sydney Harbour	Australia	503	1932
Fremont	U.S.A.	383	1972
Port Mann	Canada	366	1964
Thatcher	Panama	344	1962
Trio-Rivieres	Canada	335	1967
Zdakov	Czechoslovakia	330	1967
Runcorn-Widnes	U.K.	330	1961
Birchenough	Rhodesia	329	1935
Glen Canyon	U.S.A.	313	1959
Lewiston-Queenston	U.S.A.	305	1962
Hell Gate	U.S.A.	298	1916
Oh-Mishima	Japan	297	1979

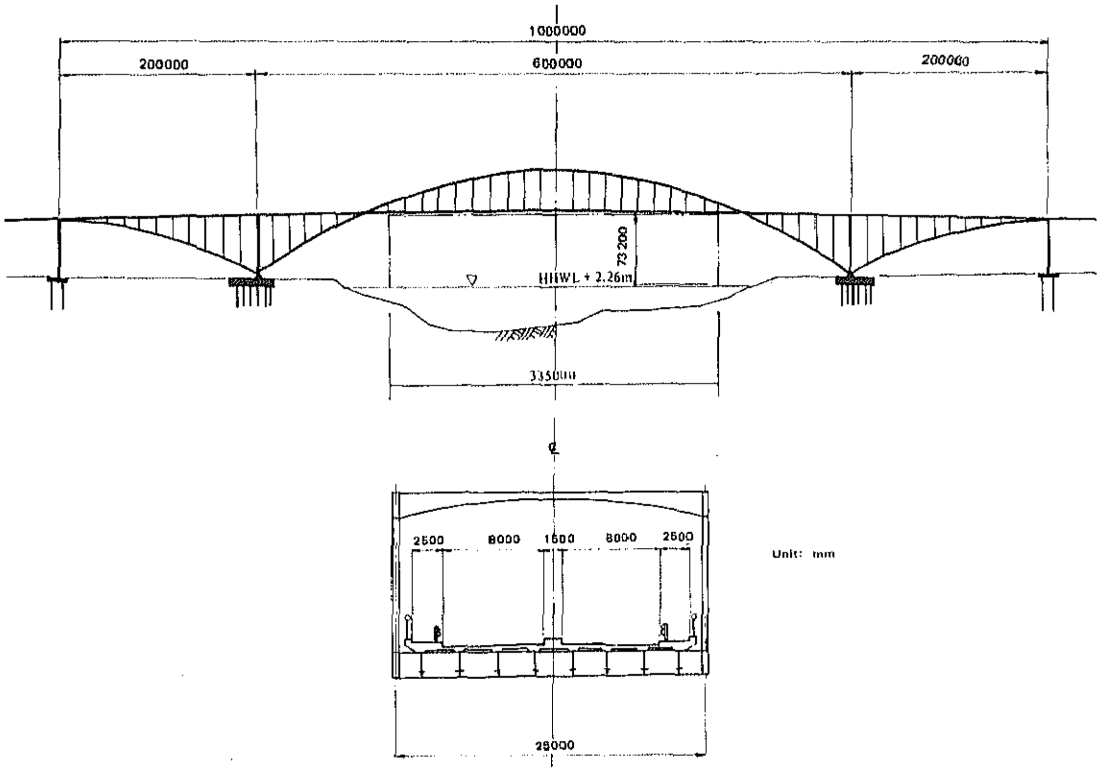


Fig. 6.3.20 ARCH BRIDGE

(4) Cable-Stayed Bridge

The cable-stayed bridge is a comparatively new type of bridge which has become rapidly and widely applied in the last three decades. But there is no precedent case to the bridge over 500 m span constructed in this type as shown in Table 6.3.24. In the near future however, it is believed that the construction of 600 to 700 m span length will become technically and economically feasible.

This type is different from the previous three alternatives in that both steel and P.C materials can be used.

This type of bridge presents an appearance of basically straight line composition, and is more economical when compared with others. Particularly when constructed in P.C structure, it is the lowest maintenance cost. Moreover, it has higher rigidity and wind-resistant stability than suspension bridges, in spite of the structure itself being suspended.

This structural type has high design flexibility. It can comprise a great variety of forms depending on the choice of different combination of main girder, tension members, and tower configurations as best suited to the project requirements.

On the other hand, it also has some disadvantages as follows:

– Relatively vulnerable to the impacts of temperature changes and settlement of supports
– Requires sophisticated construction techniques.

The comparison of two types (PC and steel) are made in the Preliminary Design.

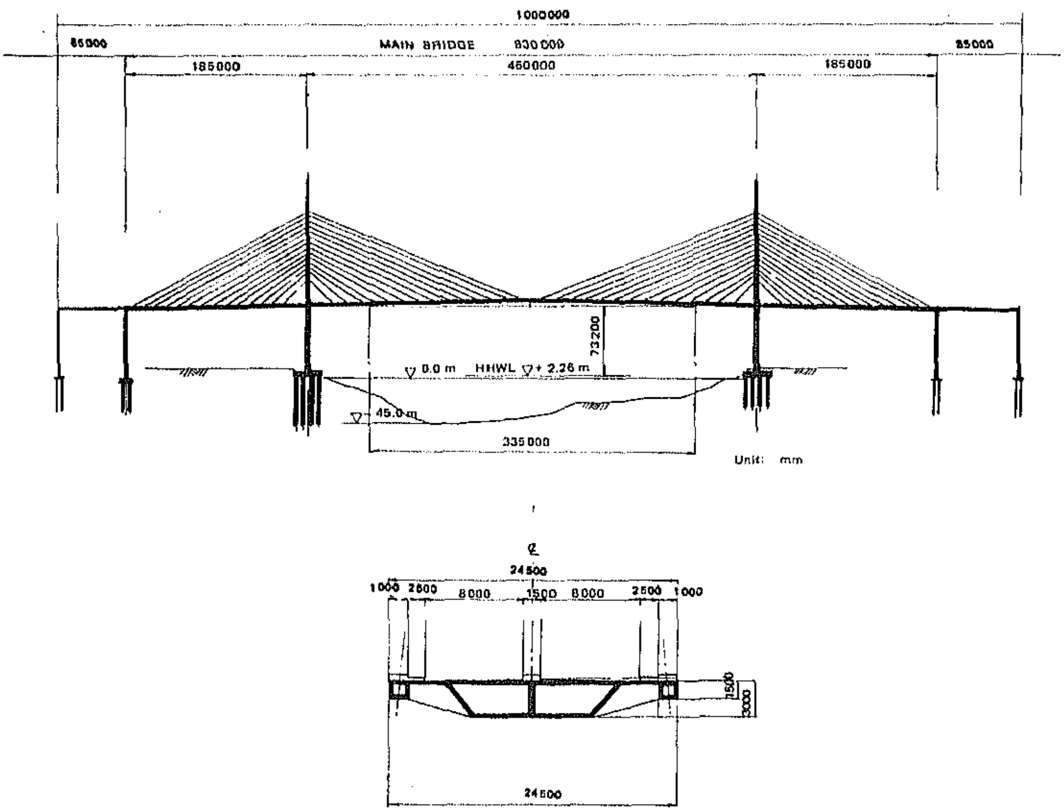


Fig. 6.3.21 CABLE STAYED BRIDGE

Table 6.3.24 LONGEST CABLE STAYED BRIDGES IN THE WORLD

	Bridge Name	Nation	Main-span length	Completion
Cable-Stayed Bridge - Steel	Yokohama-bay	Japan	460^{m}	Under construction
	Second Hooghly	India	457	1977
	Chaophya river	Thailand	450	Under construction
	Iguro-jima	Japan	420	Under construction
	Hitsuishi-jima	Japan	420	Under construction
	Meikoh-nishi	Japan	405	Under construction
	Saint Nazaire	France	404	1975
	Rande	Spain	400	Under construction
	Dame Paint	U.S.A.	396	Under construction
	Luling	U.S.A.	372	Under construction
	Flehe	West Germany	367	1978
	Yamato-gawa	Japan	355	1982
	Aji-gawa	Japan	350	Under construction
	Ncuenkamp	West Germany	350	1971
	Brazo Largo	Argentine	340	1972
	West Gate	Australia	336	
	Zarata	Argentine	330	1976
	Kohl Brand	West Germany	325	1975
	Knic	West Germany	320	1969
	Erskine	U.K.	305	1971
	Severin	West Germany	302	1969
	Bratislava	Czechoslovakia	303	1972

	Bridge Name	Nation	Main-span length	Completion
Cable-Stayed Bridge - P.C	Barrios de Luna	Spain	440	1983
	Saint Johnes	U.S.A.	396	Under construction
	Pasadas Encarnacion	Argentine	330	Under construction
	Brotonne	France	320	1977
	Pasco-Kennewick	U.S.A.	299	1978

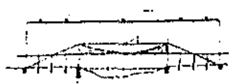
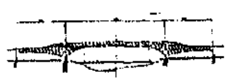
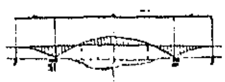
(5) Overall Evaluation

The comparative features of the four alternative types discussed so far can be summarized as given in Table 6.3.25.

Based on the above comparative studies and the result of cost estimates, it can be concluded that the cable stayed bridge is the most recommendable type for this project for the following reasons:

- The cable stayed type is the lowest in construction and maintenance costs, and if constructed as a P.C structure, the maintenance cost can also be lowest.
- If constructed as a P.C structure, domestic materials can be used.
- Application of the suspension bridge which suits longer span structures is uneconomical for this project, and the application of the cantilever truss and the arch bridge is the maximum limit of their applications. Whereas the application of the cable-stayed design is in the more suitable span ranges.

Table 6.3.25 COMPARISON OF ALTERNATIVE STRUCTURAL SYSTEMS FOR MAIN BRIDGE

	Bridge Type	Suspension Bridge	Cantilever Truss Bridge	Arch Bridge	Cable-Stayed Bridge
Configu-ration	Side view outline				
	Structure type	Catenary	Complex	Parabolic	Linear
Existing example of equal or large span in the world		Numerour	few	nil largest span of exsisting similar structure: 504 m	few
Principal material of superstructures		Steel for main girders, steel or R.C. for towers	Steel	Steel	Steel or P.C. for main girder and steel or R.C. for tower
Structural features	Specific features of structural system	Because of great horizontal reaction of the cables, it becomes uneconomic in case of weak foundation ground where heavy anchorage structure is required.	Only vertical reactions exist	Because of great horizontal reaction at the support of the arch, it becomes uneconomic in case of weak foundation ground where heavy foundation is required at such support.	Only vertical reaction exist. Such reactions are large.
	Deflection	Great	Small	Small	Relatively large
	Stability under wind load	Requires careful design consideration	No problem	No problem	Requires careful design consideration
	Effect of temperature change	In significant	No influence	Influenced	Influenced
	Statical feature	Statically indeterminate	Statically determi nate	Statically indeterminate	Statically indeterminate
	Effect of differential settlement of support	Almost unaffected	Unaffected	Affected	Relatively affected
	Interfacing with horizontal curve in side span	No problem	Difficult	Difficult	No problem
Each of construction	Technical aspect	Installation of cables and construction of anchorages require sophisticated construction knowhows, high precision of work and is therefore difficult to construct	Being statically determinable in structure, during construction by applying, cantilevers construction method, is relative less demanding in work precision, and is comparatively difficult to construct.	Employing slant - stay construction method with temporary staging, its construction, demands work precision, and involve wind resistant stability problem during construction, is relatively difficult to construct	Requires construction know-how and high work precision, and is difficult to construct.
	Construction period	Long	Shortest	Relatively long	Relatively short
	Availability of material in domestic market	Mostly unavailable	Mostly unavailable	Mostly unavailable	Mostly unavailable for steel structure but available for P.C. and R.C. structure
Economical features	Construction cost (Million Shilling)	1453	821	1054	814
	Maintenance	Relatively low cost of painting maintenance	Highest cost of painting maintenance	High cost of painting maintenance	Relatively low cost of painting maintenance in the case of steel structure and almost maintenance free in the case of P.C. structure
Other considera-tion	Travelling quality of bridge surface	Comparatively less satisfactory	Comparatively less satisfactory due to existence of expansion joint and visual discomfort from the enveloping trurs members	Good	Good
	Replacement of steel	Small	Large	Large	Relatively small in the case of steel structure, very small for P.C. structure
Overall Evaluation		- There are many example - Construction period is long due to cable work - Not recommendable due to highest cost,	Construction is relatively easy, but not recommendable due to the highest maintenance cost, lack of smooth drivability and aesthetical versions.	Not recommendable due to lack of examples precednet.	- There are a few examples but technically adoptable - Recommendable due to the construction and maintenance cost.

Note 1. Construction cost is the gross estimation of an assumed steel bridge structure of 1,000 m length inclusive of a main bridge span Chapter 8 is referred to this cost.

2. The cost breakdowns for the suspension and cable-stayed bridge are as follows;

Suspension Bridge

Superstructure and Tower	667	million K.Shs
Anchorage	421	"
Foundation of Tower	43	"
Approach Bridge	322	"
	Σ = 1,453	million K.Shs

Cable-Stayed Bridge

Superstructure and Tower	652	million K.Shs
Foundation of Tower	61	"
Approach Bridge	101	"
	Σ = 814	million K.Shs

6.4 Improvement Plans for the Existing Road Network on Mombasa Island

6.4.1 Improvement Plan

The traffic on Mombasa Island is already congested around the CBD. Future traffic conditions in case of "without improvement" for the existing road network are shown by target year in Figs. 6.4.1 through 6.4.3, which are drawn by adding the traffic to and from the project road. The capacity of existing roads are estimated at 12,500 pcu/day for 2-lane roads and 31,400 pcu/day for 4-lane roads as shown in Chapter 6.

The traffic-congested sections in 1990 are found to be in very limited areas. These congested sections will increase in number and length, and the major sections of the existing arterial roads will be congested by 2010.

For improvements to the existing road network the following two basic concepts should be adopted.

- For the regional development of Mombasa Metropolitan Area and to promote the specialized of central functions of Mombasa island, the development of road improvements should be stressed on forming the trunk road network including arterial streets in compliance with the Mombasa Transportation Plan and other master plans (Ref. Fig. 6.3.2).
- For the improvement of streets through densely developed areas and the CBD, consideration from city planning aspects is essential, since the improvements in these area will create social problems resulting from high investment costs and the long lead times required.

1) Improvements upto 1990

Miji Kenda Street to the north of Mombasa railway station has the major function of carring the traffic related to the port and industrial area, industrial area in Ganjoni, Liwatoni and adjacent area (Kinyozi), and the Likoni crossing traffic.

The street is already congested and in year 1990 the traffic demand will exceed the road capacity. The missing link behind the station (from Mwakilingo street to Lumumba Road) should be developed as a part of the future trunk road and a fly-over of the railway should be developed as early as possible. The fly-over will be a four lane carriageway including relocated street development.

2) Improvement upto 2000

The intersection of Mbaraki-Mnaji Moja Road should be improved to accompany the opening of the project. The improvement will be accomplished as a minor improvement by relocation of the existing roundabout and installation of traffic signals.

3) Improvement upto 2010

Two major improvements will be expected for the future trunk road section and inner ring road upto the year 2010.

Fig. 6.4.1 TRAFFIC FLOW ON MOMBASA ISLAND: 1990 WITHOUT IMPROVEMENT PLAN TO EXISTING ROADS

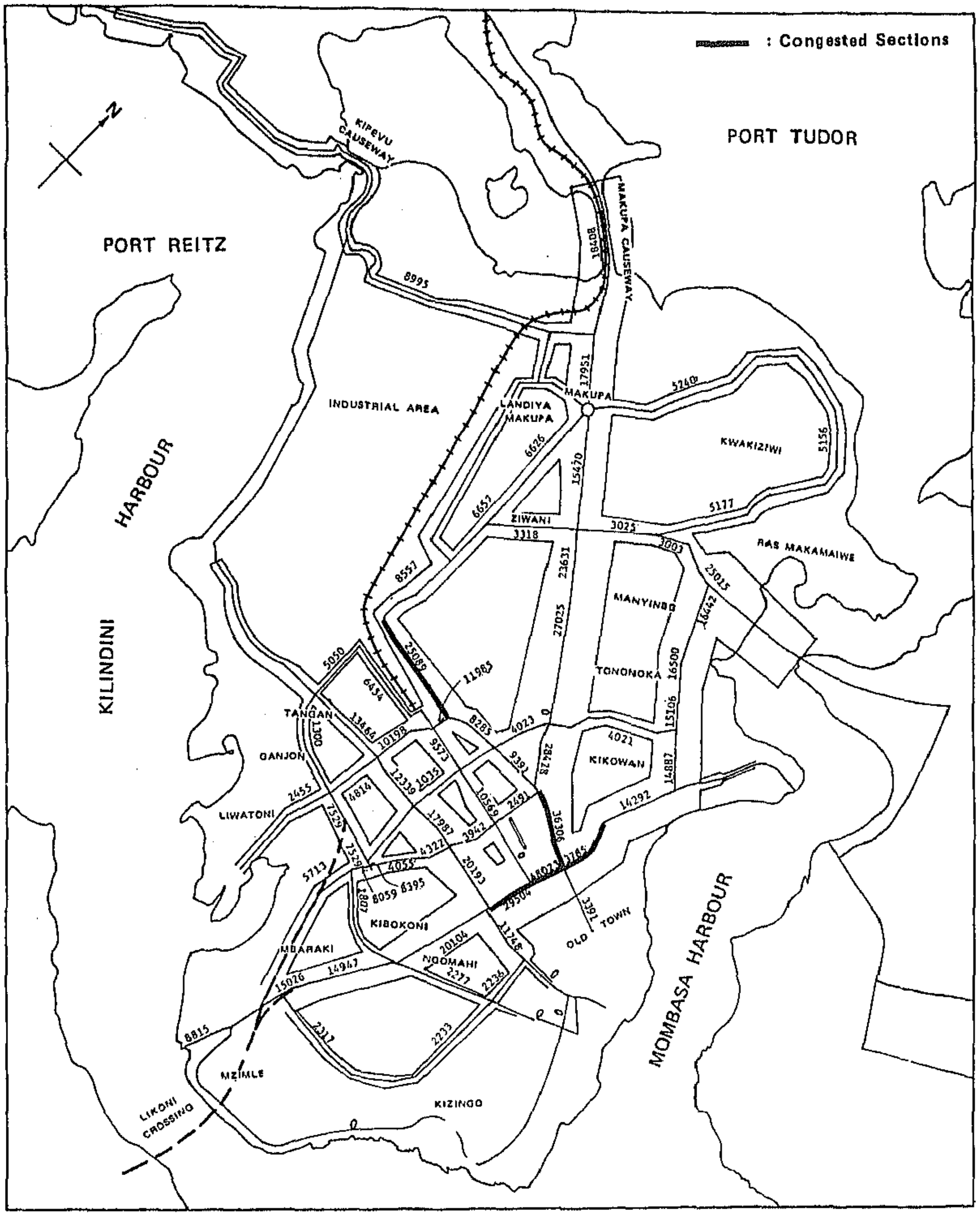


Fig. 6.4.2 TRAFFIC FLOW ON MOMBASA ISLAND : 2000 WITHOUT IMPROVEMENT PLAN TO EXISTING ROADS

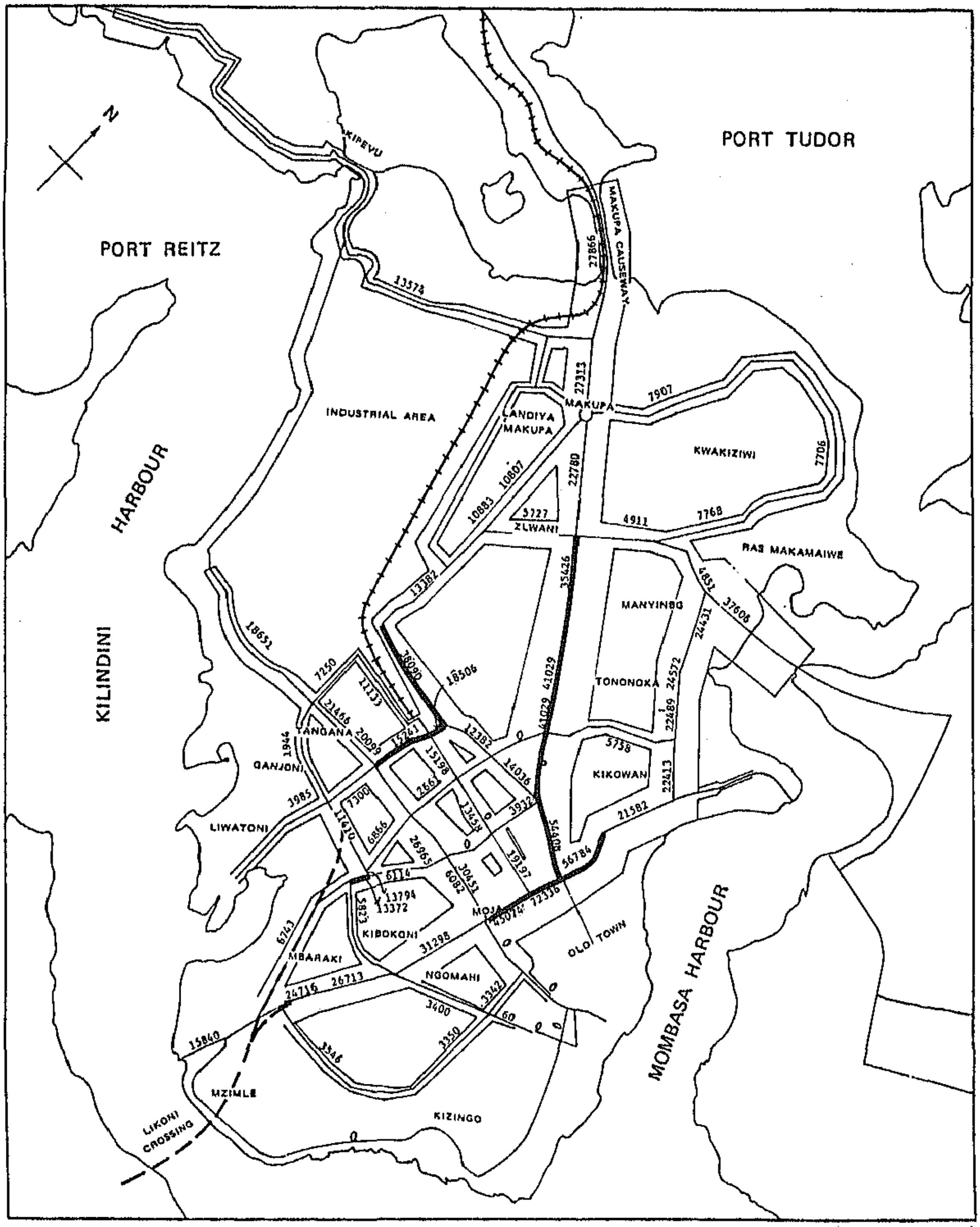
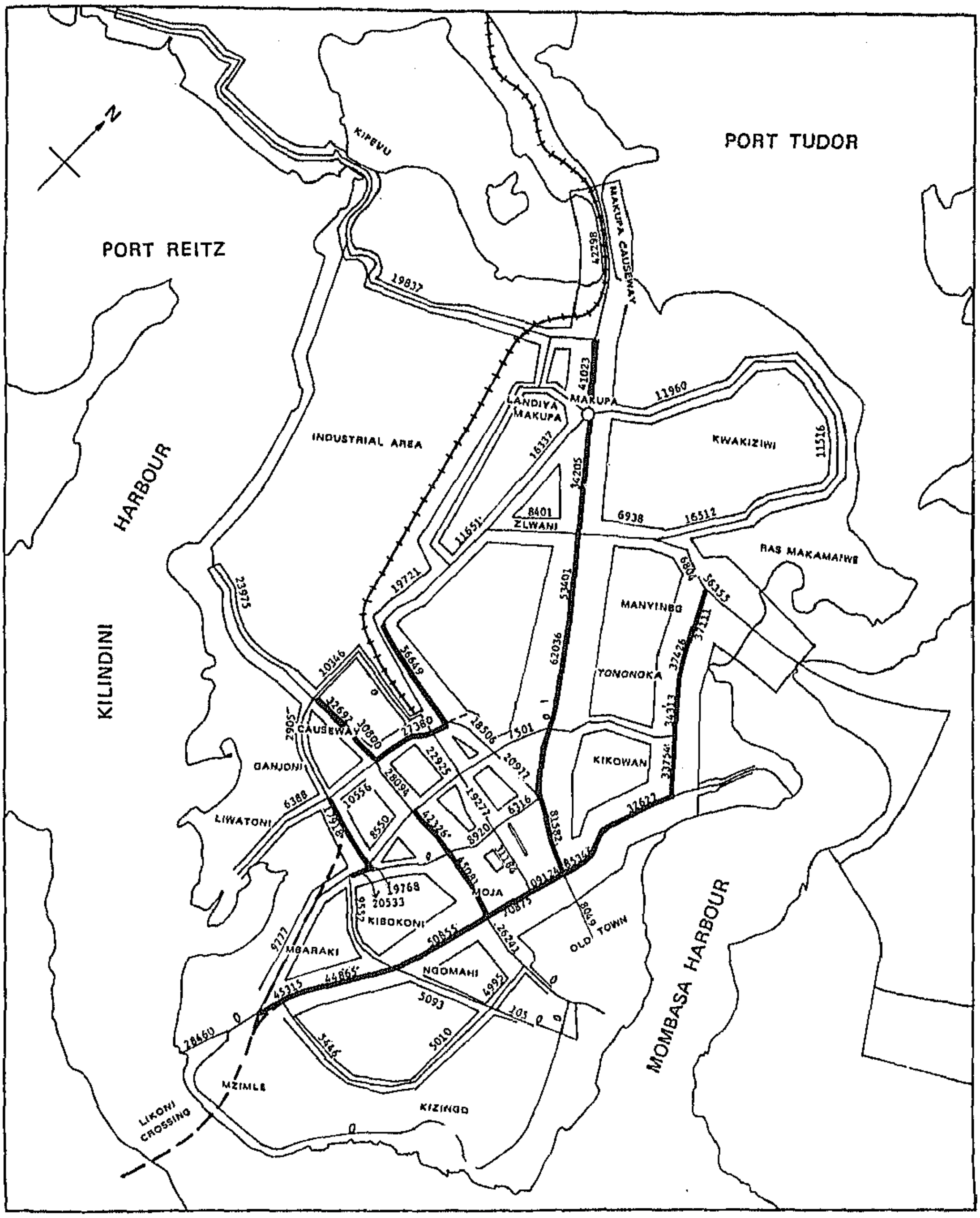


Fig. 6.4.3 TRAFFIC FLOW ON MOMBASA ISLAND : 2010 WITHOUT IMPROVEMENT PLAN TO EXISTING ROADS



In 2010 Archbishop Makarios Road will reach capacity and the intersection with Tangana road and Moi Ave. will also exceed capacity, as will Tangana Road (a part of Inner Ring Road).

Therefore the trunk road section between the end of the project to Mwakilingo street will be a continuous grade separation including a 4-lane expansion of Archbishop Makarios Road. The inner ring road section between Archbishop Makarios Road to Kenyatta Ave. will be expanded to 4-lane carriageway.

4) Further Improvements beyond 2010

The following may need improvement after 2010.

- Inner ring road extension to Tom Mboya Ave.
- Kenyatta Ave – Ziwani Road (Malindini Road extension) intersection
- Makupa circle
- Functional specialization for major arterial streets, Kenyatta, Digo, Abdel Nassir, Moi, Nyerere Ave., etc.

6.4.2 Rough Construction Cost Estimation

The improvement proposed above is not included in the project and the construction cost is estimated as follow:

(1) Upto 1990

Fly-over development (behind Mombasa ST.)	175 Million Shs.

(2) Upto 2000

Intersection improvement of Mbaraki/Mnagzi Moja ST.	6 Million Shs.

(3) Upto 2010

Grade separation for Archibishop Makarios R.	235 Million Shs.
Improvement of Inner Ring Road	25 Million Shs.

CHAPTER 7 PRELIMINARY DESIGN

CHAPTER 7 PRELIMINARY DESIGN

7.1 General

A comprehensive analysis and preliminary design related to different engineering aspects for the project roads is discussed in this Chapter.

Engineering investigations including topographic and soils and materials survey were undertaken by engaging the services of local firms.

Based on the engineering investigations and forecast of traffic demand types and dimensions of road facilities were determined in order to estimate the quantities, project cost and construction schedule.

7.2 Soils and Materials Survey

7.2.1 *General*

The purpose of the survey is to obtain the necessary data for the preliminary design and construction cost estimate for the structures, embankment and pavement of the project.

The soils survey was carried out in July 1983 and Aug. 1983 by a local geothechnical company, Geodrill Ltd. Mombasa. The materials survey was conducted to obtain information on sand, crushed stone, etc. The field work and laboratory testing for the soils survey were planned and supervised by the Team.

7.2.2 Geological Characteristics and Soils Survey

1) Geological Characteristics

The geological conditions in the study area are described in the GEOLOGY OF THE MOMBASA-KWAIE AREA, GEOLOGICAL SURVEY OF KENYA in 1953, and is shown in Fig. 7.2.1.

The geology in the area is called as the Cainozoic rocks composed of the representatives of the Pliocene, Pleistocene, and Recent periods as classified below:

Recent		Alluvia. Oyester Beds
Pleistocene	Upper	Red wind-blown sands Kilindini sands
	Middle	North Mombasa Crag Raised coral feef
	Lower	Flat-bedded yellowish sands and clays
Pliocene	Upper	Magarini sands

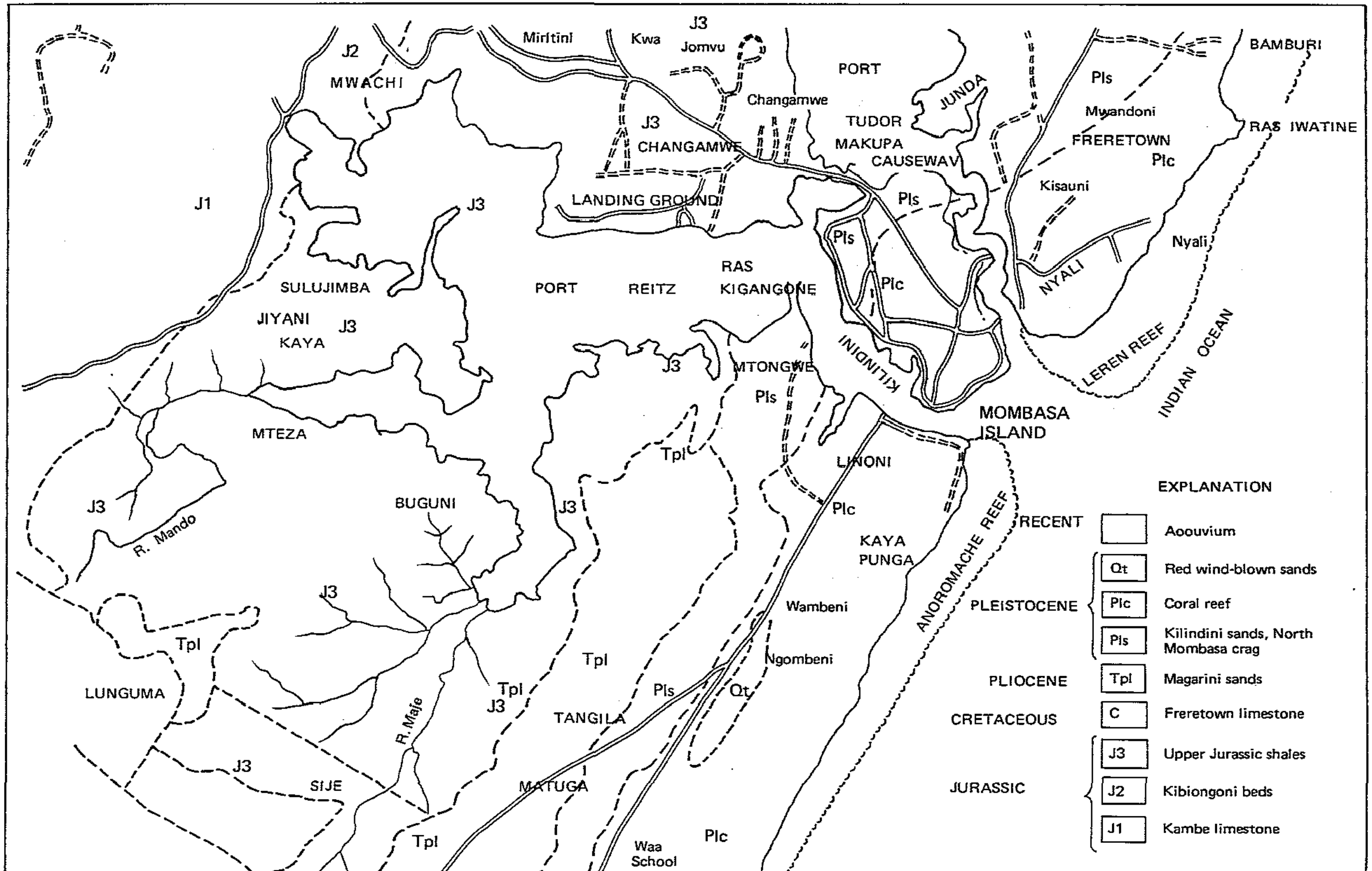


Fig. 7.2.1 GEOLOGICAL MAP

Same significant points are described in the EAST AFRICAN PORTS DEVELOPMENT STUDY, KENYA PORTS, Oct. 1977.

- The natural harbours of Mombasa and Kilindini on either side of Mombasa Island were formed only in Pleistocene and Recent times, before which the castline was 3 – 4 km inland from its present position.

  The line of this relic coast marks a distinct boundary in the ground conditions in the harbours. On the seaward side, the ground is favourable with a mixture of sands, cemented sands and coral. The slope of the bed-rocks is very steep, about 8° to the ESE.

- The principal factor determining the present shore configuration has been the rise and fall of sea level associated with the periods of global glaciation in Pleistocene and Recent times.

  One such rise, from a starting level of about 60 m CD, was accompanied by growth of a fringing reef on the edge of an off-shore platform cut by wave-action into the tertiary coastal rocks. As the reef grew the lagoon behind filled with sands and coral debris, the proportion of the latter decreasing away from the reef. The highest level of the reef was about +30 m CD.

  Subsequently when the sea level fell, the rivers were reactivated, and cut deep channels to the sea through the coral and lagoon deposits to form the natural harbours of today. These channels reach depths of more than 50 m below chart datum, indicating the level to which the water fell.

  Subsequent variations of sea level have resulted in further cycles of filling and erosion, so that the sediments are very varied in their age and content.

The project bridge crosses over the area between the sourtheastern corner of mombasa Island and the northeastern corner of South Mainland, centering the Kilindini Harbour. The topography along the channel is very steep and formed upper terrace (coral, approximately 10 m above MSL) and lower terrace (coral reel located around MSL). Besides these there are serveral small terraces eroded by sea water.

It is found that the bed rock location is 130 m below the existing ground by the well drilling project executed by the geotechnical company.

2) Soils Survey

The soils survey consisting of 8 bore holes was conducted at the locations shown in Fig. 7.2.2. Standard penetration tests at 2 m to 4 m intervals were also conducted for selected bore holes. The samples taken with samplers were tested in the laboratory for moisture content, particle size analysis, wet density, unconfined compression test, etc. The geological profile is shown in Fig. 7.2.3.

Fig. 7.2.2 BOREHOLE LOCATION

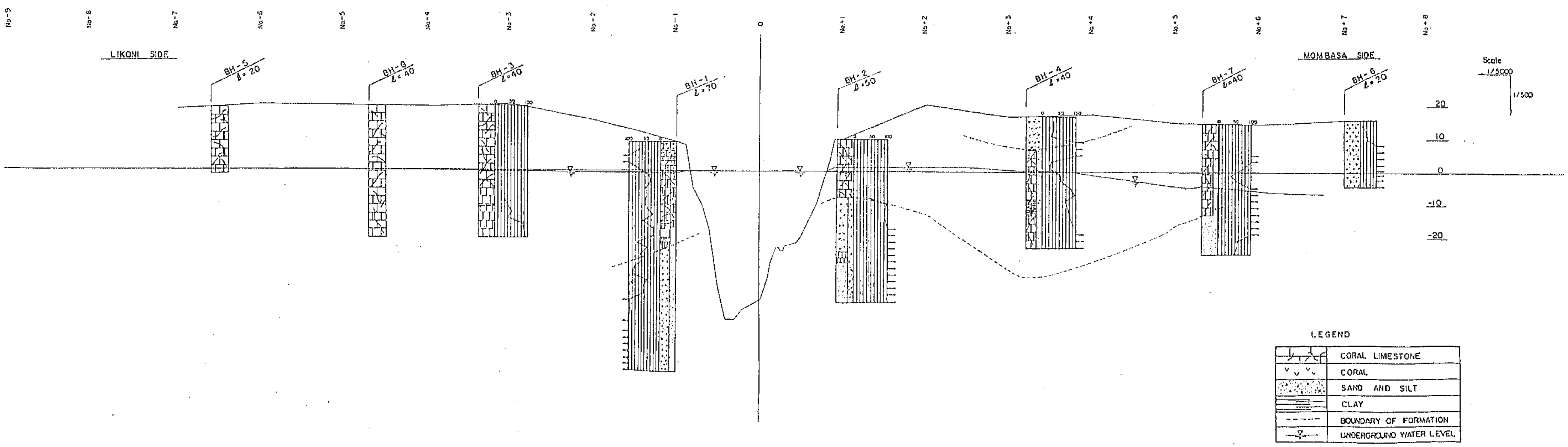


Fig. 7.2.3 GEOLOGICAL PROFILE

(1) BH-1

The subsoil conditions are found: hard coral in the depth of 0 – 7 meters, broken coral with sand in the depth of 7 – 28 meters and yellow silty sand in the depth of 28 – 50 meters. The N-values shows considerably large figures. The bored length was extended to 70 meters from the scheduled 55 meters since it is necessary to identify the subsoil conditions of the deeper layers to plan the foundation of the long span bridge. As the result it is found that in the depth of 50 – 70 meters there exist very hard sand layers with a mixture of yellowish-brown coral gravel. The samples were in half solid state and their relative densities are considerably high.

– Subsurface Coral

The subsurface coral is hard enough to function as the bearing stratum and shows porousity, non-uniform layers and a mixture of sand seams. The subsurface can not be considered suitable for the large scale foundation since it has the following problems:

– Coral is weathered by the penetration of the rainwater, the cracks of the coral develop due to the deformation of the silt.

– The foundation will be affected by the deformation of the soft layers underneath the coral with thickness of only 7 m.

– Intermediate Broken Coral and Silty Sand Layers

In the intermediate depth hard and soft layers exist at random with clayey sand layers having the N-value of about 30. This depth is not suitable for bearing as the strata has problems with bearing capacity and deformation.

– Silty Sand Layers in Deep Soils

The deep soils consist of very hard silty sand layers unlike the intermediate depth having a mixture of soft layers. The samples of sand showed solid state although containing water. This depth will have no problem for bearing as the strate has high density and an N-value of more than 50.

(2) BH-2

The soft topsoil is as thin as 1.0 meter, and underneath there exists coral limestone layers about 15 meters thick. These layers are not so porous but cracks have developed in them. A field permeability test showed a large value of $K = 6 \times 10^{-3}$ cm/sec as the coefficient of permeability. Underneath were found fine sand layers of about 14 meter thickness and layers of weathered sandstone of about 7.5 m thickness underneath with hard sand layers underlying.

The coarse sand layers around 35 m below the ground are with the N-value of more than 50 and is suitable as the bearing strata for the large scale tower foundation.

(3) BH-3, BH-5 and BH-8

The borings made on the Likoni side revealed all the soils consisting of coral stone layers in which a remarkable difference in weathering by depth was noticed. The general conditions are as follows:

- The data of boring BH-1, BH-3 and BH-8 show that the weathering in the vicinity of the sea coast is more developed, and is less landwards, and the materials are in a rock form having excessively high permeability due to the predominant coral limestone.
- The weather shows variation by depth, and the surface portion shows considerable rock formation while the lower is more weathered to sandy formation.
- Due to the existence of hollows, which is a characteristic of weathered coral limestone, layers are non-uniform, having small N-values by location.

The soil engineering characteristics of coral limestone is a subject left unsolved. Although the sampled materials show a high strength in the laboratory test (q_u = 11 kg/cm² ~ 75 kg/cm²), at this point of time the weathering of coral limestone is unknown. For the design of the foundation further study is required according to the size of the foundation taking into account the above factor.

(4) BH-4, BH-6 and BH-7

Generally the subsoil of Mombasa Island consists of coral limestone and silty sand layers, and their longitudinal continuity is very complicated. Four boring logs were carried out, and it is difficult to grasp the overall characteristics.

The following considerations can therefore be given in respect of the foundation type.

- The bearing force of the soil layers can be judged in principle by the N-values.
- Covering the locations of BH-4, BH–6 and BH-7 including BH-2, the coral limestone is very soft in the portion which is weathered although less porous than the similar situation on the Likoni side. This can be considered due to the hollows formed by the weathering.

  In this layers clayed materials are found. This would result in sinking after receiving a heavy load.

3) Other Soils Survey Data

Some informations and data on the actual sub-surface conditions in the harbour were obtained from KPA.

(1) Data of Harbour Bed

KPA executed boring tests for the port development in 1977. A total 5 boring were carried out in the harbour. These boring tests were carried out to the comparatively shallow depth upto 25 m below the harbour bed.

Based on the test results, generally the N-value of soil around the harbour mouth shows higher value and becomes weaker toward the interior of the harbour. The higher composition of clay, silt layers (including clayey sand, sandy clay etc.) are found in the inner harbour, while that of the mouth is more sand, sand stone etc., as shown below:

– At bore Hole (BH-12 west of the Likoni ferry terminal)

0 – 5m	Very loose dark grey sandy silt with shelly and pieces of coral. Strong odour, N = 0 – 39
5 – 8	Dense light grey m.c. sand with small fragments of shell and occasional hard pieces of coral. Broken pieces of coral with coarse sand and shells. N = 30 – 42
8 – 18	Weakly cemented whitish m.c. sand with occasional hard pieces of coral. N = 17 – 65
18 – 25	Hard white calcereous sandstone weathering to soft brown sandy marl, occasional pockets of blue clay. Weakly cemented light yellow m.c. sand with occasional hard pieces of coral. N = 50 – 58

(2) Data for Mbaraki Berth Extension

KPA carried out a total of 15 bore holes and 6 soundings for the extension of Mbaraki bearth in Aug. 1976, of which two bore holes (B-114 and B-115) were drilled at the end of the berth adjacent to Mbaraki Creek.

The test results show that silty sand and sand layers are predominant, with traces of decomposed vegitation and fine to coarse gravels comprising cemented sands. The N-value varies by depth. The silty sand layers with the N-value of 10 around the harbour bed become harder (up to 70) at a depth of 29 m (dense sand layer). The deeper layer of dense sand is recorded as harder, up to the N-value of 120 to 150.

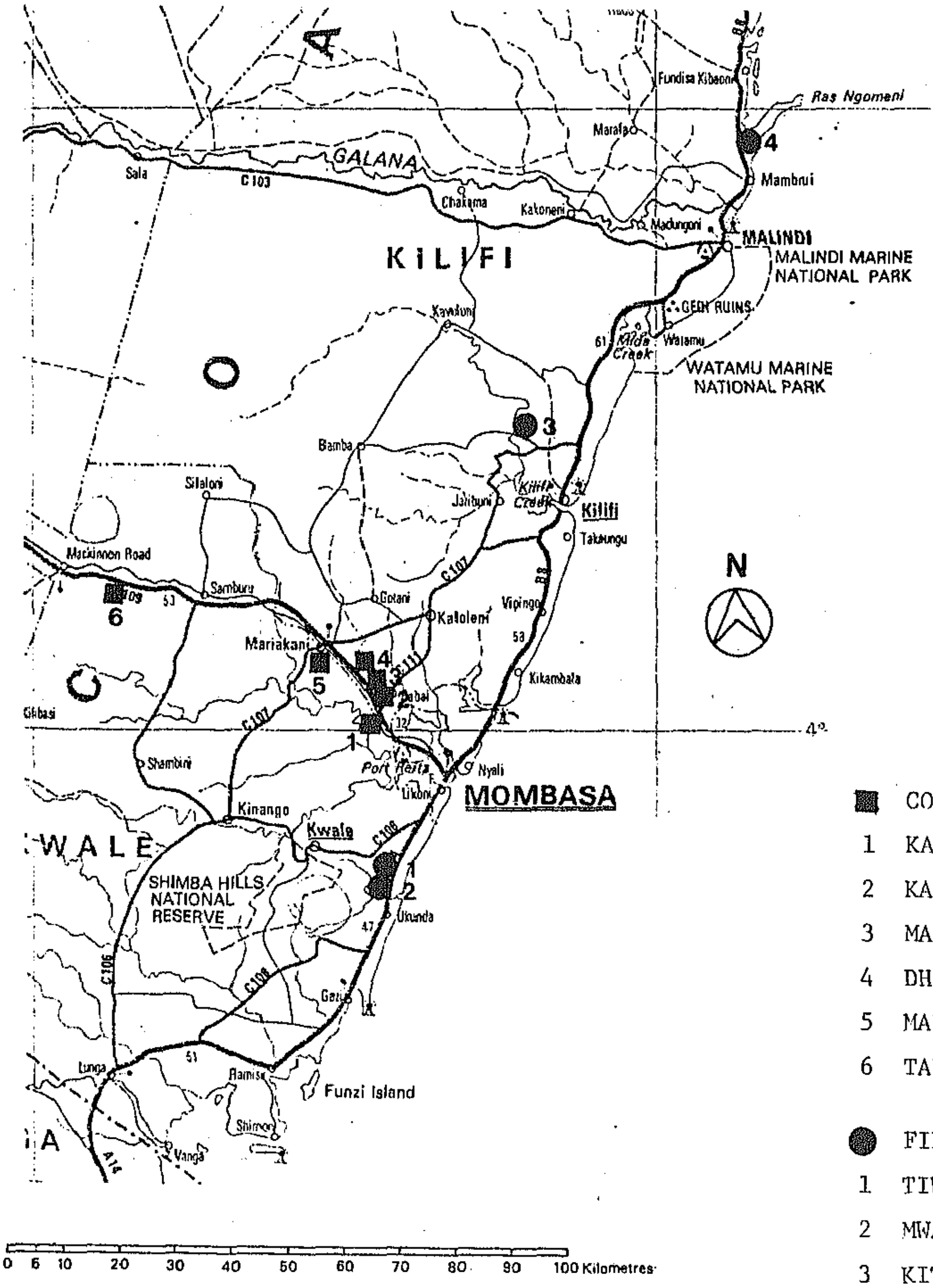
7.2.3 Materials Survey

The team contacted the MOTC, Mines & Geological Department, Ministry of Environmental & Natural Resources, and collected the data and information regarding sources of construction materials. Site investigations were also carried out by the soils engineer with reference to the topographic and geological maps.

1) Aggregates and Sand

Fig. 7.2.4 shows the locations of quarry sites and Table 7.2.1 shows the survey results.

Quarry sites that can produce coarse aggregage near Mombasa are situated along Mombasa-Nairobi Road (A109), at Kambe, Kaydee, Mavji, Dhanjal, etc.



■ COARSE AGGREGATE

1 KAMBE

2 KAYDEE

3 MAVJI

4 DHANJAL

5 MARIAKANI

6 TARU

● FINE AGGREGATE

1 TIWI

2 MWACHEMA

3 KITANGANI

4 MBONI

Fig. 7.2.4 LOCATION OF QUARRY SITES

Table 7.2.1 DESCRIPTION OF QUARRY SITES

(1) COARSE AGGREGATE

Quarry	Formation	Kind of Rock	Operation
Kambe	Kambe Limestone	Sandy Limestone	Under operation
Kaydee	Mazeras Sandstone	Calcareous Sandstone	Under operation
Movji	Mazeras Sandstone	Calcareous Sandstone	Under operation
Dhanjal	Mazeras Sandstone	Calcareous Sandstone	Produced by Rod Mill
Mariakani	Mariakani Sandstone	Sandstone	Shutdown
Taru	Mariakani Sandstone	Sandstone	Shutdown

(2) Sand

Location	Occurrence	Deposit Volume	Remarks
Tiwi (Pit Sand)	Diluvial deposit	Abundant	Fine grained with organic matter and silt
Tiwi (River Sand)	River Deposit	Not Abundant	Good quality
Mwachema	River Deposit	Not Abundant	Good quality, but mining not started
Shimba	Diluvial Deposit	Abundant	Fine grained with silt
Kitangeni	Riverq Deposit	Abundant	Fine grained with silt
Tinboni	Coast Terrace Deposit	Abundant	Very coarse

Required quality and quantity are available except from Mariakani and Taru quarries.

Four sites were investigated for sand and the results are described as follows:

– Vicinity of Tiwi

Two kinds of sand, pit sand, and river sand, are excavated in the area. The pit sand is mainly deposited at western side of Lunga Lunga Road.

The sand is mained by manual workers and is find grained with organic matter and silt. This sand is not suitable for concreting work.
The Mabu River which lies in the west of the above-mentioned diluvial deposit is about 1.5-meter wide stream. The sand is so well graded as to be most suitable as fine aggregate material. It is also mined by manual workers and collected by trucks. The haul road is in bad condition.

In Mwachema, South of Tiwi, a much wider river of 10 m width has the same good quality as the Mabu River. However mining has yet to start.

– Kitangani

The sand which is supposed to be of the Pleistocene or Diluvium is mined at Kitangani, about 15 km northwest of Kilifi. However this is fine grained containing much silt.

– Tinboni

In Tinboni about 20 km north of Malindi, mining is conducted to sand which exists in the coast a terraces of the Pleistocene or Diluvium having a mixture of shell flakes. The sand which is too coarse grained for use does contain less silt.

7.3 Preliminary Design of Road

7.3.1 Throughway

1) Description of the Project Road

The recommended routes for the Project road based on navigation clearance was designated in Subsection 6.3 of Chapter 6. The general description of the Project road is given for the Likoni area and Mombasa Island.

(1) Likoni Area

The topography in this area is flat comprising coral and limestone. The land use is villages in the inland area and the coastal area of Kilindini Channel, and resort and high class residential area long the Indian Sea coast. Recently the inland area along the project road was developed as a residential area by a private developer.

The project road starts at the intersection with Lumga Lunga Road during Phase-I. In Phase II the Project road should be extended to the South in compliance with the

Mombasa Transportation Plan. The starting point is established at the existing Mtongwe and Lunga Lunga Road intersection.

The project road will not be an obstruction to the Likoni area, but contribute to the enhancement of local traffic. Existing local traffic is served through at-grade intersections planned on the project road.

(2) Mombasa Island

The topography in the area is also flat composing coral, limestone and sand. The land use is resort and high class residential area at the eastern side and industrial and residential area at the western side of the project road.

The project road connects to Nyerere Ave. (H = 55 and 45 M) and the intersection of Nyerere Ave. and Mbaraki Road (H = 73.2 M) in Phase-I. In Phase-II the project road will extend to Mbaraki Road. The traffic distribution effect depends on the trunk road construction as proposed in Sub-section 6.4.

2) Required Number of Lanes

Based on the analysis of the throughway capacity, the number of lanes to be constructed by phase are shown as follows.

Phase-I

Navigation Clearance:

H = 73.2 M : Two lanes by 1992 except the main bridge

H = 55, 45 M : Two lanes by 1992 for the full length of the project road

Phase-II

All cases should be expanded to four lanes toward Mbaraki Road by 2001.

3) Horizontal Alignment

The horizontal alignment was determined considering the geometric design standards and control points listed as follows:

(1) Likoni Side

- The intersection of Lunga Lunga Road and Mtongwe Road
- Future extension to the South to be within the right-of-way in compliance with the Mombasa Transportation Plan
- A road to Sherry Beach Hotel
- Narrowest crossing to the residential area along the coast

– Electric power cable along the channel

(2) Island Side

– Electric power cable along the channel

– The intersection of Nyerere Ave. and Mbaraki Road (H = 73.2 M)

– Nyerere Ave. at Phase-I (H = 55, 45 M)

– Mbaraki Road at Phase-II (All cases)

– Archbishop Makarios Road (H = 73.2 M)

4) Vertical alignment

The preliminary design for vertical alignment was made simultaneously with the design for horizontal alignment.

Basic requirements controlling the vertical alignment were as follows:

– Minimum embankment height of 50 cm is maintained at the sholder edge in Likoni area.

– Minimum gradient of 0.30% is adopted for roadway surface drainage

– Maximum gradient of 4.30% is adopted for the approach section of the main bridge irrespective of navigation clearance alternatives.

– A combination of horizontal and vertical alignments is considered.

In addition to the basic requirement, the following primary control points were considered for the determination of vertical alignment;

(1) Likoni Side

– At-grade intersection with Lunga Lunga Road

– A road to Sherry Beach Hotel

(2) Island Side

– Eixsting ground height of Nyerere Ave. and Mbaraki Road to maintain entrances of private house.

– The intersection of Nyerere Ave. and Mbaraki Road.

– STA 1 + 820 Railway (H = 73.2 M).

5) Plan of Frontage Road

The frontage roads (described in Design Standard) are constructed in this project for the section, from the intersection of Nyrerere Ave. and Mbaraki Road to the intersection of Nyerere Ave. and Dedan Kimathi Ave. (H = 55, 45 M) and to the future trunk road section of Mbaraki Road.

In the Likoni area no frontage road is provided due to at-grade intersection with the Project road.

6) Abutment Height

The construction cost for the road structure of both embankment and viaduct types are compared. As a result, the embankment of 7 meter was found to be 60% cheaper than that of the viaduct. The embankment height is however determined as 7 m in the Likoni area considering to minimize the environmental isolation and a practical abutment height, and as 3 m in the island area considering the residential circumstances.

7) Coverage of the Project

(1) Facilities necessary for the Project.

Facilities necessary for the Project include throughway and intersections including toll-gates as described in 7.3.4.

Phase / Navigation Clearance	Phase-I	Phase-II
73.2 M	Throughway: STA-3-586∿ STA 1 + 780 L = 5,366 M Intersection: - Nyerere Ave. and Mbaraki Road - Lunga Lunga Road and Mtongwe Road	Throughway: STA-2-820∿ STA 2 + 160 L = 4,980 M
55	Throughway: STA-3-582∿ STA 1 + 708 L = 5,294 M Intersection: - Nyerere Ave. and Dedan Kimathi Ave. - Lunga Lunga Road and Mtongwe Road	Throughway: STA-2-820∿ STA 1 + 520 L = 4,340 M
45	Throughway: STA-3-586∿ STA 1 + 708 L = 5,294 M Intersection: - Nyerere Ave. and Dedan Kimathi Road - Lunga Lunga Road and Mtongwe Road	Throughway: STA-2-820∿ STA 1 + 370 L = 4,190 M

The facilities to be constructed in the Project are indicated by solid lines in Vol. III, "Drawings".

7.3.2 Intersections

1) General

The project road is closely related to the urban street network. The intersections between the project road and the existing street network play an important role for both systems in order to handle the traffic smoothly.

Capacity, speed and safety on most urban arterial streets depends primarily on the number, type and spacing of the intersecting streets. The layout and traffic control devices used for the intersections at-grade are the key elements for the safe and efficient operation of the arterial streets. Since intersections at-grade play such a vital part in the urban transportation system, the importance of their design and operation should not be under-estimated.

Capacity analysis is one of the most important considerations in the design of signal controlled intersections. The calculations of the capacity and the procedure adopted are in accordance with the recommendations of "Road Design Standard", Japan. the following are the basic elements for the capacity analysis.

The analysis was made for the year of 2000 as well as for the year 2010.

Basic Elements

– Traffic capacity per lane:

Through lanes	= 1,800 veh (PCU)/green hour
Right-turn and Left-turn lanes	= 1,200 veh (PCU)/green hour

– Other elements:

Peak factor	= 11%
Rate of direction at peak hour	= 60%

2) Preliminary Design of Intersections

The related intersections with the project road are analyzed and listed as follows:

– Nyerere Ave./Mbaraki Road Intersection (H = 73.2 M)

– Nyerere Ave./Dedan Kimathi Ave. Intersection (H = 55, 45 M)

– Lunga Lunga Road/Mtongwe Road Intersection (All cases)

Future traffic demand without road improvement (Ref. Figs. 5.2.10 through 5.2.12) is used for the analysis.

As a result all intersections can be treated by the signals analyzed in Appendix B.

(1) Nyerere Ave./Mbaraki Road Intersection

The analysis shows that the traffic can be managed with some allowance in year 2000. In year 2010 the traffic also can be managed by adding more lanes at the intersection. However the extension of the project road should be applied to Mbaraki Road considering the traffic congestion on Nyerere Ave. and CBD.

(2) Nyerere Ave./Dedan Kimathi Ave. Intersection

In year 2000 the traffic can be managed with some allowances. In year 2010 the traffic can not be managed due to the existing facilities around the intersection. The major traffic to CBD should be diverted to Mbaraki Road.

(3) Lunga Lunga Road/Mtongwe Road Intersection

In year 2000 the traffic can be handled. In year 2010 the through traffic to the South exceeds the existing Lunga Lunga Road capacity (18,400 PCU/day, two-way two lanes). Therefore, the extension plan to the South in compliance with the Mombasa Transportation Plan should be established.

The intersection plans are shown in Vol. III, "Drawings".

7.3.3 Pavement Design

1) General

Pavement design was made according to the methodology as described in "Material and Pavement Design for New Roads", ROAD DESIGN MANUAL, PART III, MOTC May, 1981.

The elements of design used are daily traffic volume, values of soil support and climate.

2) Elements of Pavement Design

(1) Traffic Class

Average vehicles equivalence factors are determined based on the data surveyed by Material Branch MOTC in July 1977 and the Manual, MOTC. These factors are as follows:

Vah. Type / Factor	Buses	Medium Goods	Heavy Goods	Oil Tankers
Equivalent Factors	1.0	3.0	10.0	5.0

The cumulative number of the standard axles is estimated to be 37 million for 10 year design periods based on the initial number of commercial vehicles and future forecast traffic demand.

With respect to the number of commercial vehicle the commercial vehicles will be increased due to the opening of Tanzania border and expected conversion from medium truck to heavy truck are considered as described in Chapter 10.

The traffic class can be defined as T_1 based on the Manual.

(2) Climate

The mean annual rainfall was recorded as 1,073 mm in Mombasa area. The area is classified as wet tropical climate according to the Manual. Bituminous surfacing should therefore be applied.

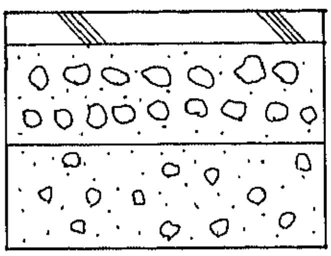
(3) Subgrade

According to the soil survey the ground in the area and embankment material to be used are both coral and limestone. The subgrade bearing strength is classified as S_6 based on the Manual.

For design purpose the class of S_5 (CBR = 15 ~ 30) is applied considering the non-uniformity of the ground and embankment material.

(4) Pavement Types

Two types of pavement structure (Type 12 and 15) can be selected based on the Manual. Considering the traffic demand, subgrade, material availability and construction ease, Type 12 is selected as shown below.



7.3.4 Toll Gate

1) General

The project road is not a normal tollway of which the construction, maintenance and operation cost is paid back after opening. In this study the toll rate should not cover the whole construction cost.

In this sub-section engineering aspects such as location, required number of tollgates, etc. are studied.

2) Outline of Tollgate

(1) Location

Tollgate locations should be determined from traffic safety, land availability, convenience of tollgate worker, and construction cost.

The island area along the project road is highly developed and the planned road structure is mostly bridges. The tollgate should therefore be located in Likoni area.

Considering at-grade intersections, future grade-separation for those intersections and alignment (R $\geqq$ 1,000 M, Grade $\leqq$ 2%), the tollgate is determined to be located at STA-2-200.

(2) Toll Levy System

A flat toll system, which is uniform tariff rate by vehicle type, is adopted to minimize the construction cost due to smallest handling time.

(3) Traffic Lanes at Tollgate

The number of traffic lanes at a tollgate was determined from the traffic volume (interval of arrival), the service time per vehicle, and also the service level provided (planned length of waiting queue).

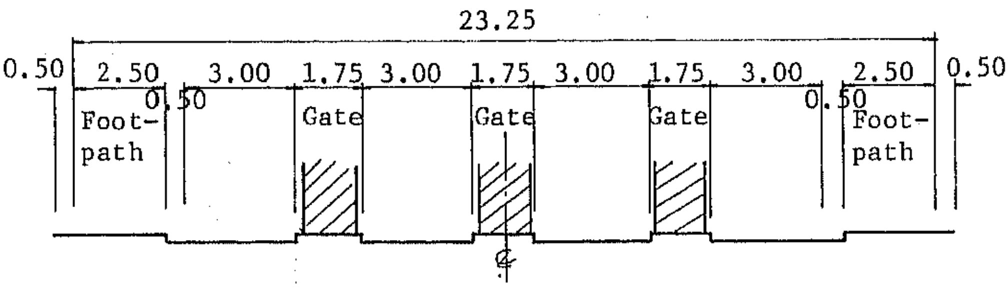
The above factors were established based on the "Road Design Standard" Japan. Average 650 veh./hr/lane can be treated at service time of 5.5 sec/veh.

On this basis the number of tollgates is calculated by Phase.

In the year 2001, a total of 4 lanes (including in and out traffic) are required and 6 lanes in year 2010, while the number of tollgates is 3 in year 2001 and 5 in year 2010 with one gate being made reversible.

(4) Cross-section at tollgate

The tollgate section with 4 lanes is shown below.



7.4 Prelimary Design of Bridge

7.4.1 General

The objectives of the preliminary design dealt with in this Chapter are to establish design concepts and determine basic structural features together with preliminary estimation of quantities and costs to be involved in the construction. The preliminary design has such depth and accuracy as it may not occur any serious discrepancy or inconsistency with the economic evaluation described later in this Report or with the detailed design development to be performed in the phase of implementation.

As to the selection of the bridge type over the main span, which is a highlight of this study, reference is made to Subsection 6.3.7, Alternative Main Bridge Study. It is noted however that the Alternative Main Bridge Study has not yet arrived at the conclusion of wheather PC or steel cable-stayed bridge is to be selected or which vertical clearance is to be used amongst three alternatives H = 73.2 m, 55 m and 45 m.

A total length of the project bridge become 3,835 m, 2,415 m and 2,045 m, each correspondign to H = 73.2 m, 55 m and 45 m, including the main bridge of 830 m.

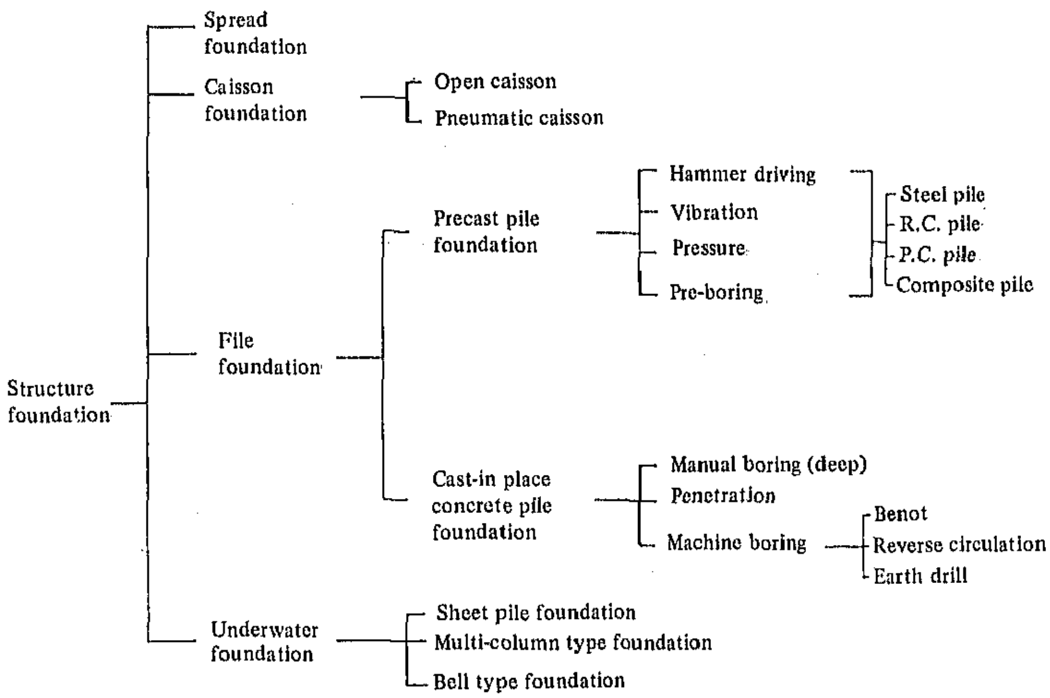
7.4.2 Determination of the Tower/Pier Foundation of the Cable-Stayed Bridge

1) General

The types of bridges foundation are generally classified into a spread foundation, caisson foundation and pile foundation, and further sub-classified as shown in Table 7.4.1, according to the methods and materials to be used. Their uses are generally based on the subsoils conditions, structural characteristics, construction conditions, etc. In this subsection discussions are made on the foundation type of the proposed bridge, and on determination of the most appropriate foundation type.

As reported in Subsection 7.2 in Chapter 7, subsoils conditions at the proposed bridge site mainly consists of coral and sandy soils, and are not geotechnically solid and homogeneous due to their poor continuity.

Table 7.4.1 CLASSIFICATION OF FOUNDATIONS BY TYPE



As to coral, even modern geotechnology still leaves many engineering characteristics unclear (non-homogeneity, distribution of hollows, weathering, etc.) and this makes the study difficult.

2) *Type of Tower/Pier Foundation of the Cable-Stayed Bridge*

The foundation of the tower or pier of a cable-styed bridge is subject to massive loads and its superstructure is affected by a non-uniform settlement of the bridge piers. In these respects a foundation type with high rigidity and stability is desirable. BH-1 and BH-2 show the boring logs explored to design the tower foundations. These logs suggest possibilities of using s spread foundation, caisson foundation and cast-in-place large diameter pile foundation. For the following reasons, however, the first two above are not adopted.

(1) Spread Foundation

This type is economical when a bearing structure exists at the depth of 5 to 10 meters, but the following reasons make this type unapplicable for this project.

At the Mombasa Island side the subsoils of about 16 m deep consisting of thick coral layers have a possibility of sinking due to the existence of hollows, being weathere by rainwater, development of cracks, etc., and the subsoil itself lacks homogeneity. In fact, falures arising from the use of coral as a large scale foundation are reported in Mombasa Island. At the Likoni side the sandy layers underneath the weathered coral layers alternately containing soft silt layers and sandy layers, cause sinking when the subsoils is subject to massive loading. Considering that the reliabile layers that exist about 50 meters below the ground surface are chosen as the bearing layer, spread foundation is not suitable.

(2) Caisson Foundation

This type has higher rigidity and larger bearing area as compared with the pile foundation. Generally it assures high construction accuracy and has many experiences of being used for large scale bridge foundation. However, it has disadvantages such as comparatively higher construction cost, longer construction period, less safety for construction workers, etc.

The following problems make this type unapplicable for this project.

At the Mombasa Island side sinking of a caisson body into the coral layers as mentioned in the spread foundation will be difficult. On both sides the subsoils consist of high permeability materials, and both sides are located very close to the sea shore. In this situation the adoption of a pneumatic caisson method may be considered. The good bearing layers at the Likoni side exist about 35 meters below MSL, which will make the construction impossible because the air pressure used is generally limited to 3 kg/cm^2.

Therefore the caisson foundation is not suitable.

The most effective method of bear a large loading is to found on the underlying solid sandy layers, avoiding the unknown factors of the coral layers or the interposition of soft silt layers in the sand layers. Under these subsoil conditions, cast-in-place piling methods, except for deep foundation piling method and reverse circulation drilling method, will not be applicable. The deep foundation piling method can not be used where manual excavation of more than 10 times the diameter of the pile is required and the groundwater in flow exceeds 50 liters per minute making the work impossible.

The reverse circulation drilling method has some weak points in coping with the sliding of soft sandy soils in the interposed sand layers and water in flow, etc. In the absence of boulders, however, these problems can be solved by using bentonite liquid under proper control. This method was used for the construction of the new Nyali Bridge under the similar soils conditions.

From the above discussions the reverse circulation drilling method can be considered most suitable.

7.4.3 Planning and Design of the Cable-Stayed Bridge

1) General

The planning of the bridge requires careful technical consideration since its main span length of 460 m ranks as one of the longest cable-stayed bridges constructed in the world. Particularly in the case of H = 55 m and H = 45 m, the bridge needs to be designed to have a 2-lane width at the initial-stage construction,which results in the extremely slender superstructure, raising problems of aerodynamical stability and stabilities during the erection. However, these problems have been theoretically ensured of its safety by this study, yet it should be re-examined and reassured by means of "Wind-tunnel" tests at the stage of detailed design development.

The planning and design of a cable-stayed bridge means a search for the optimal structural system combined with span length, height and type of towers, depth of dirders, arrangement of cables, bearing conditions, etc. A number of options are available amongst these determinants.

Attempts were made in this report to select the structural system considered best suited for the project. It is to be noted that the selected system should be refined through further examination in the future.

2) Planning and Preliminary Design of the Cable-Stayed Bridge

(1) Arrangement of Main Bridge Spans

The span length of 460 m over main channel is determined as subsection 6.3.7 in Chapter 6.

Total bridgespan of 830 m inclusive of side spans is arranged as a 5-span continuous cable-stayed bridge to have 93 m + 92 m + 460 m + 92 m + 93 m spans

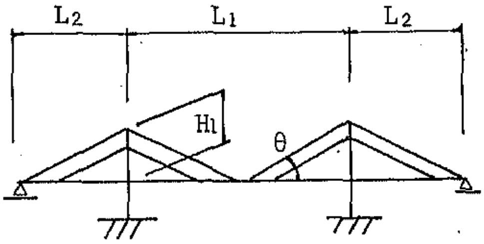
The reasons are:

- The ratio of the side to main spans will be economically selected as $\frac{L_2}{L_1} = 0.4 \sim 0.5$. In view of the horizontal alignment, it is preferred to arrange the beginning points of the curve as close to the bridge as possible but not to the extent that the curve falls within the bridge. Taking this into consideration, the minimum ratio of 0.4 shall be selected.

- In this case, however, uplifting forces occur at the side spans and the tower has to resist the bending of the cantilever erection used for the main span. To avoid this, piers shall be provided at the middle of the side spans, thus making the total 5-span continuous.

(2) Dimensions and Types of the Tower

It is generally admitted that the desirable height of towers are dominated by stayed cables, ratio of the length of centre span, and stayed angles of the cables to girders, and given by the following formula.



$$\frac{H_1}{L_1} = 0.16 \sim 0.25$$

$$\theta = 22 \sim 25°$$

In this study, $H_1/L_1 = 0.25$ is adopted to maximise the effects of the upper cables and thus the height of towers are determined.

As to the shape of a tower, there are many types as shown in Fig. 7.4.1. In this study, an A-shape tower is selected because of:

- High torsion regidity in the transverse direction, ensuring the most stable structural system.
- *In the case of a 2-lane geometry, a single-column,single-plane-cabling system cannot* be adopted.

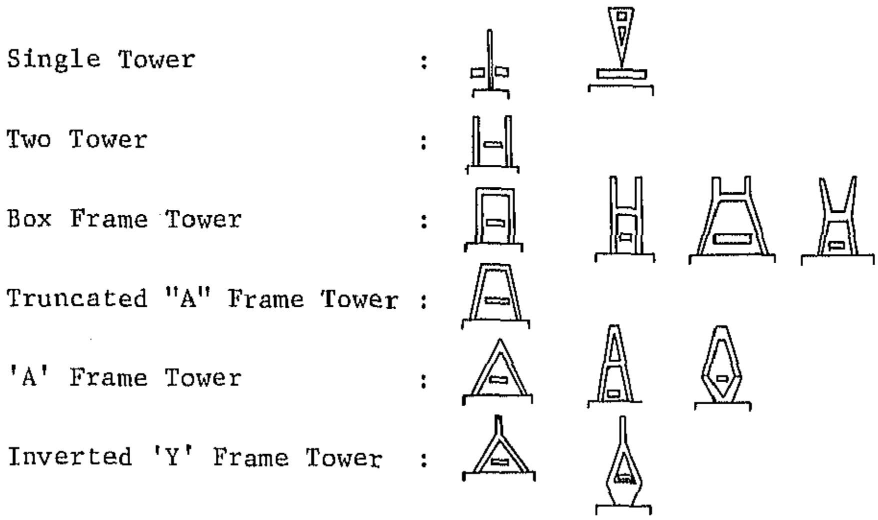


Fig. 7.4.1 SHAPE OF TOWER

(3) Staying system and Pitch of Cables

There are many cable-staying systems. Due to the large-scale of the bridge, it is essential to use a multi-cabling system that stays the cables in the shapes of "radial", "fan" or "harp" with single or double cabling.

A fan type with double cabling system is adopted in this study for the following reasons.

- A short depth of girders relative to its span length is applicable, resulting in advantages for erection and dynamic characteristics.
- A radial type creates difficulties of installing many cables at the top portion of towers.
- It is said that a fan type is superior to a harp type in terms of economy.
- Double cabling is superior to single cabling in terms of aerodynamic stability and double cabling naturally gives smaller diameter of cables.
- Cabling pitch is determined as 8 m for the P.C. bridge and 18 m for the steel bridge in consideration of the relationship between the pitch and the depth of girders, and a cycling time of erection.

(4) Shape of Girders

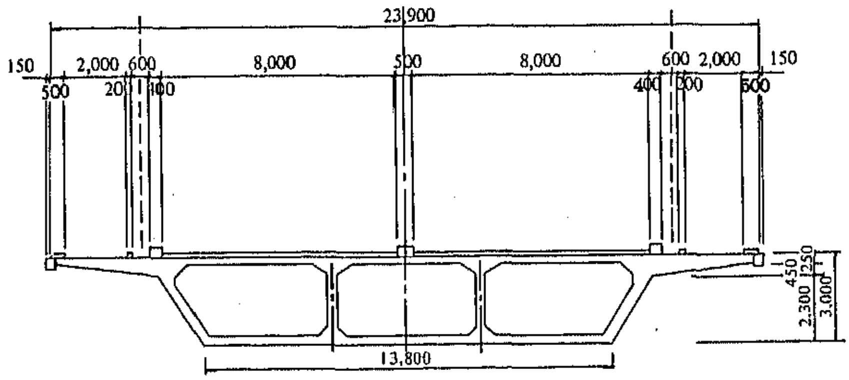
The ratio of the depth of girders to the length of main span is normally not to exceed $d/L_1 = \sim 1/180$.

The shorter depth of girders lends to the economy of long approaches. Taking this into consideration, the depth is determined as d = 3 m ($d/L_1 = 1/153$).

The shape of the girder is selected as a "trapezoidal" for girder in consideration of its high torsional rigidity, aerodynamical stability and ease for anchoring cables as shown in Figs. 7.4.2 and 7.4.3. The locations of anchoring cables differ between the cases of H = 73.2 m (4-lane) and of H = 55/45 m (2-lane) as shown in Vol. III "Drawings".

The shape of the steel bridge is also designated as a trapezoidal box-girder in consideration of aerodynamic stability. The girder having 460 m span length and 3 m depth requires high torsional rigidity. More specifically, detailed dimensions are determined as shown in Fig. 7.4.3, based upon the past results of wind-tunnel tests. The shape of the girder remained unchanged between 2-lane and 4-lane cross sections but shall be attached with brackets against 4-lane section.

(1) 4-LANE



(2) 2-LANE

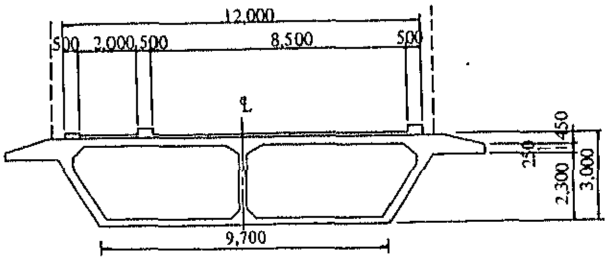
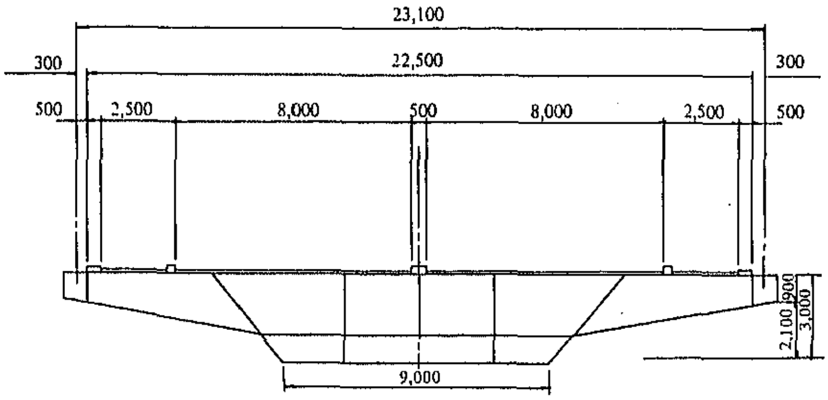


Fig. 7.4.2 CORSS SECTION OF P.C. GIRDER

(1) 4-LANE



(2) 2-LANE

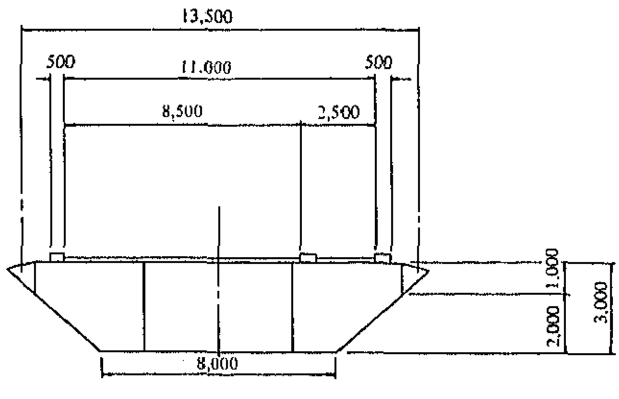


Fig. 7.4.3 CROSS SECTION OF STEEL GIRDER

(5) Boundary Conditions

There are two alternative methods related to the relation between towers and piers, i.e. either monolithic or structurally separate, and to the relation between towers and cables, i.e. fixed or movable. Furthermore there are two alternative methods of making a mid-span continuous and providing a hinge at the middle of the mid-span. These alternative methods have merits and demerits, but herein this study a type having a monolithic tower and pier with structurally separate continuous girder (without the hinge) is selected for the following reasons.

– To reduce uplifting forces on side spans and assure transversal rigidity and smooth surface, a continuous main span is adopted.

– Considering the dispersing effects of thermal stresses or stressed due to seismic forces and the difficulties of dealing with the large displacement at the free end of the girder, a "floating" support of the girder is adopted.

For the ease of construction, cables are fixed to the tower.

(6) Analytical Method and Results

Analyses were made not only to a completed structural system but also to the system during the erection period. The structural system of a cable-stayed bridge is of the high-degree statically indeterminate structure, which requires use of computer for analysis.

When analysis, it becomes necessary to choose small-displacement theory or finite-displacement theory and this may be dependent upon the structural rigidity of a system.

In this study, the following considerations are given to the analyses.

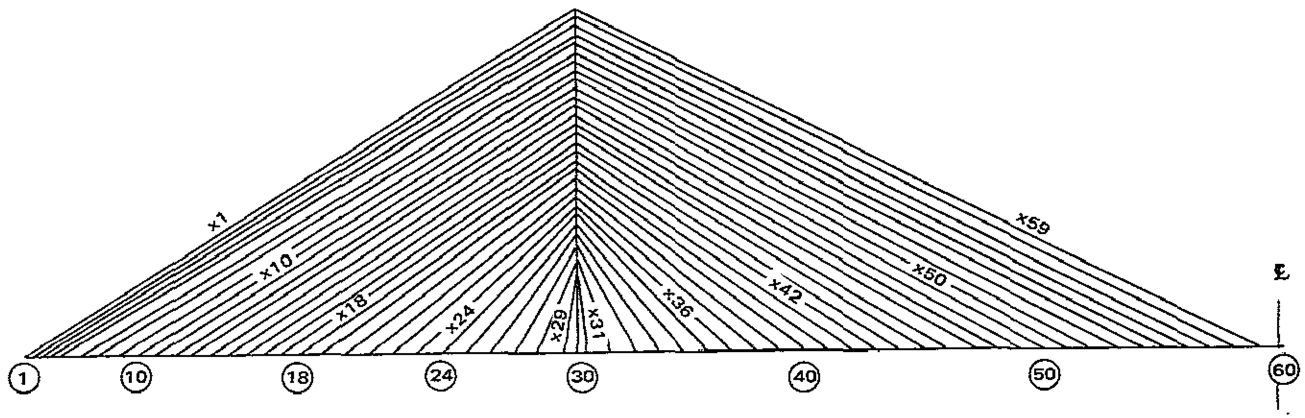
– P.C. cable-stayed bridge is analysed by the small-displacement theory recognising that little difference may occur between the results by the small displacement and finite-displacement theories.

– Steel cable-stayed bridge is analysed by the finite-displacement theory because the structure is light and flexible.

The results are shown in Table 7.4.2.

Table 7.4.2 RESULTS OF ANALYSIS

(1) P.C CABLE-STAYED BRIDGE (73.2M, 4-lane)



CABLE

	Tensile force (Ton)	Number of Strands
x 1	901	88
x 10	840	80
x 18	826	80
x 24	693	68
x 29	430	44
x 31	444	44
x 36	606	60
x 42	748	72
x 50	850	80
x 59	837	80

(Strand: 15.2 mm)

MAIN GIRDER

				10	18	24	30	40	50	60
Bending Moment and Axial Force	Ordinary Case	Bending Moment (Max.)	M (t.m)	6,642	7,664	5,224	2,231	1,877	4,066	4,867
			N (t)	6,657	12,793	14,382	15,512	13,509	7,085	261
		Bending Moment (Min.)	M (t.m)	-5,789	-9,079	-3,360	-1,580	-3,300	-1,711	-2,215
			N (t)	7,367	11,499	16,143	17,237	14,446	7,460	-10
	Seismic Case		M (t.m)	±4,876	±11,081	±14,198	±1,912	±4,424	±3,255	±55
			N (t)	∓ 622	∓298	∓272	∓652	∓952	∓567	∓25
Thickness	Upper Slab		(m)	0.30	0.30	0.30	0.70	0.30	0.30	0.30
	Lower Slab		(m)	0.25	0.25	0.25	0.70	0.25	0.25	0.25
	Outer Web		(m)	0.355	0.355	0.355	0.827	0.355	0.355	0.355
	Innter Web		(m)	0.25	0.25	0.25	0.70	0.25	0.25	0.25

CROSS SECTION
OF MAIN GIRDER

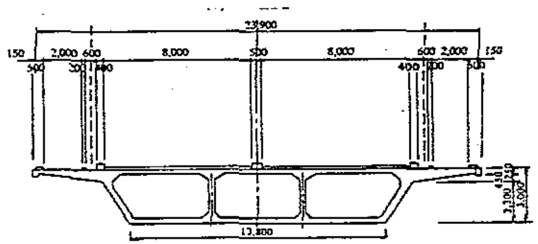
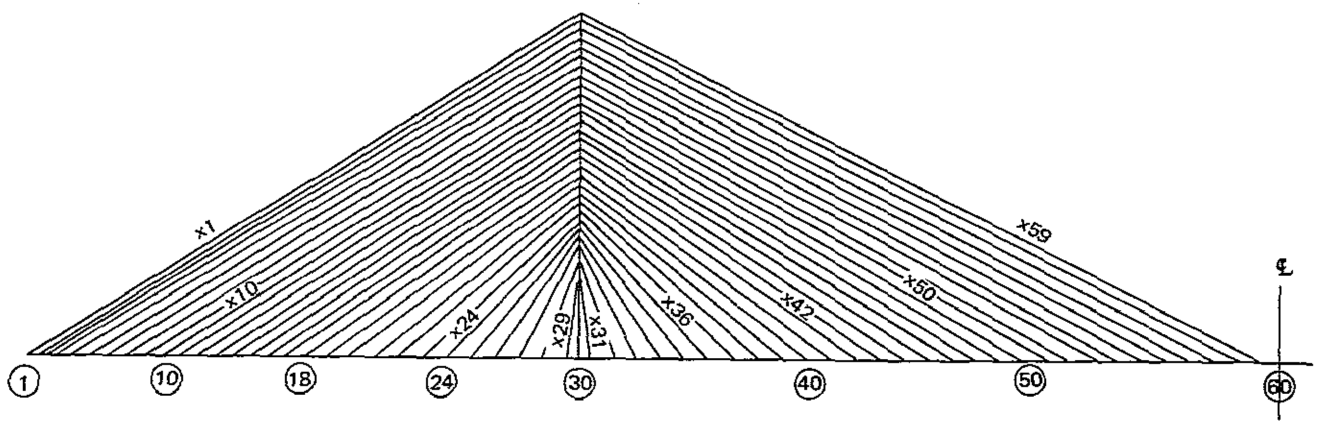


Table 7.4.2 RESULTS OF ANALYSIS

(2) P.C CABLE-STAYED BRIDGE (55^{M}, 45^{M}, 2-lane)



CABLE

	Tensile force (Ton)	Number of Strands
x 1	650	64
x 10	563	56
x 18	560	56
x 24	518	52
x 29	331	32
x 31	344	36
x 36	450	44
x 42	528	52
x 50	584	56
x 59	597	60

(Strand: 15.2 mm)

MAIN GIRDER

				10	18	24	30	40	50	60
Bending Moment and Axial Force	Ordinary Case	Bending Moment (Max.)	M (t.m)	-2,765	8,272	4,291	-314	-282	2,414	3,246
			N (t)	4,640	8,736	10,338	10,952	9,076	4,947	192
		Bending Moment (Min.)	M (t.m)	-8,915	-1,097	-585	-2,464	-3,998	-1,777	-1,172
			N (t)	5,106	7,875	10,876	11,993	9,613	5,183	31
	Seismic Case		M (t.m)	±3,564	±7,017	±10,753	±1,098	±3,070	±2,531	0
			N (t)	∓350	∓224	∓51	∓417	∓681	∓360	0
Thickness	Upper Slab		(m)	0.30	0.30	0.30	0.30	0.30	0.30	0.30
	Lower Slab		(m)	0.40	0.70	0.40	0.70	0.40	0.25	0.25
	Outer Web		(m)	0.40	0.80	0.40	0.80	0.40	0.30	0.30
	Inner Web		(m)	0.40	0.80	0.40	0.80	0.40	0.25	0.25

CROSS SECTION
OF MAIN GIRDER

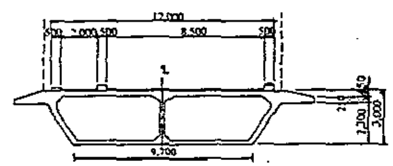
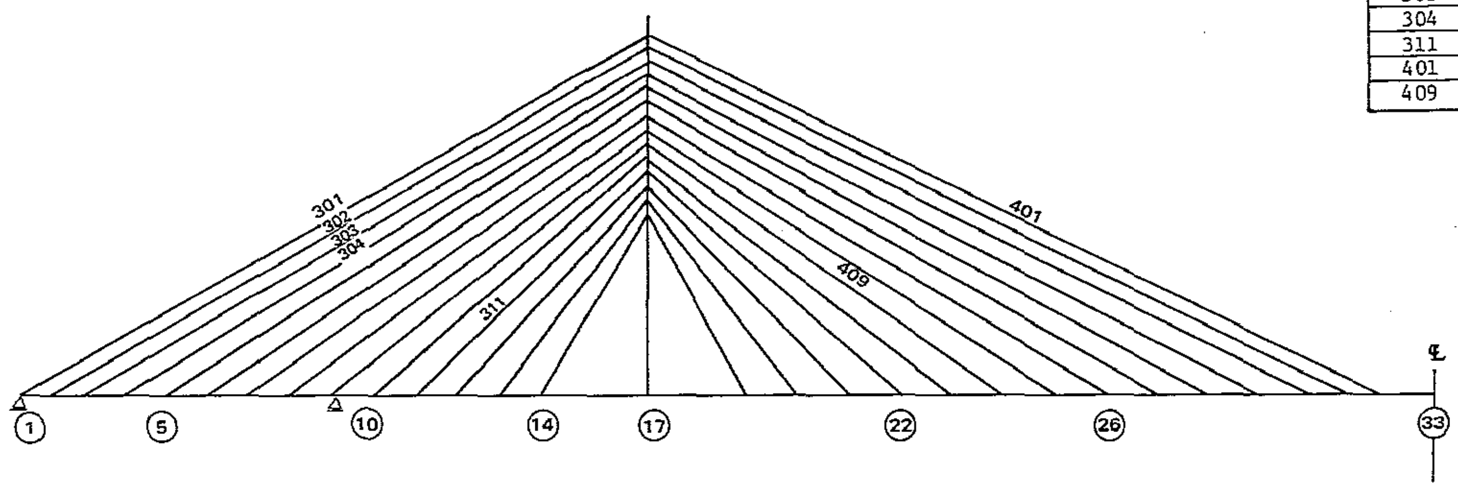


Table 7.4.2 RESULTS OF ANALYSIS

(3) STEEL CABLE-STAYED BRIDGE (73.2^{M}, 4-lane)

CABLE

	Tensile force	Number of Wires
301	439 t	PWS 349
302	430	" 341
303	421	" 335
304	411	" 331
311	308	" 271
401	407	" 331
409	323	" 271



			5	10	14	17	22	26	33
Bending Moment and Axial Force	Ordinary Case	M (t.m)	-5,651	-6,748	-4,135	+2,003	-3,347	-2,767	+4,951
		N (t)	-3,547	-5,299	-7,818	-7,819	-6,725	-4,757	+81
	Seismic Case	M (t.m)	-5,404	-7,942	-10,125	-3,976	-5,482	-3,372	+3,064
		N (t)	-3,140	-5,527	-7,272	-7,609	-5,260	-4,355	+51
Thickness	Upper Flange	(mm)	12	12	12	16	12	12	12
	Lower Flange	(mm)	12	12	12	14	12	12	12
	Outer Web	(mm)	12	12	12	12	12	12	12
	Inner Weg	(mm)	12	12	12	12	12	12	12

CROSS SECTION OF MAIN GIRDER

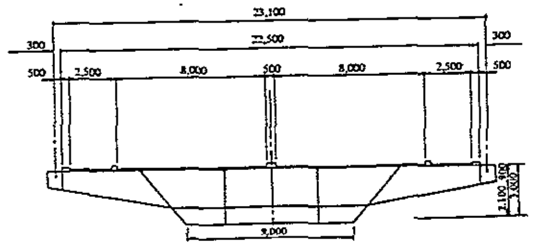
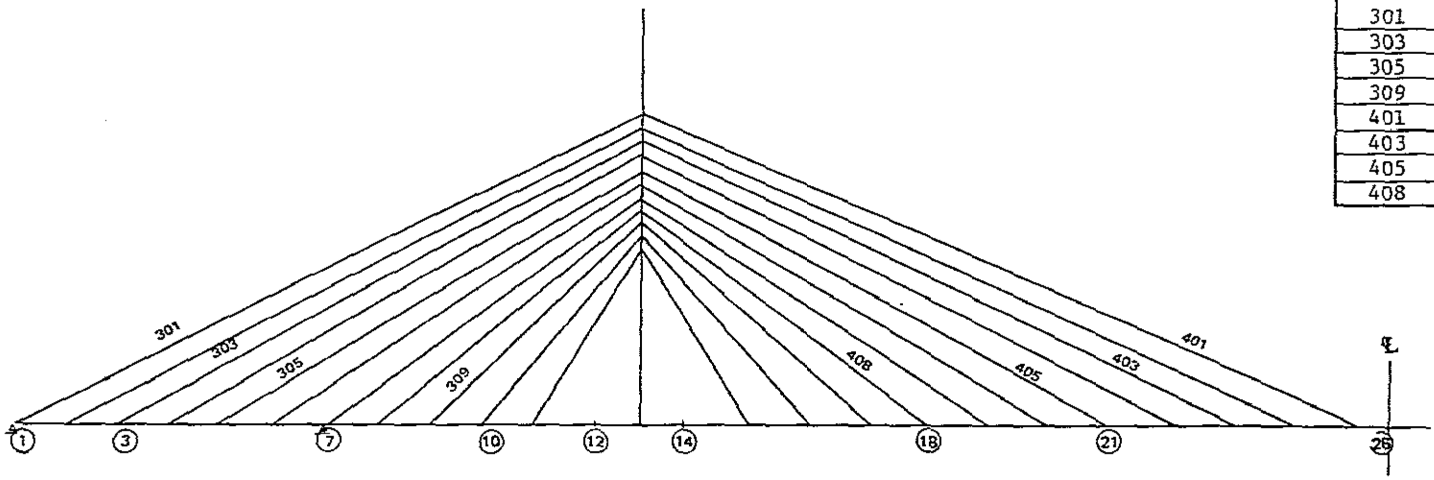


Table 7.4.2 RESULTS OF ANALYSIS

(4) STEEL CABLE-STAYED BRIDGE (55 ^{M}, 45 ^{M}, 2-lane)



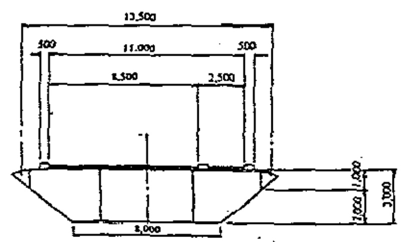
CABLE

	Tensile force (Ton)	Number of Wires
301	411	PWS 331
303	314	" 271
305	271	" 217
309	201	" 169
401	387	" 331
403	314	" 271
405	267	" 217
408	204	" 169

MAIN GIRDER

			3	7	10	12	18	21	26
Bending Moment and Axial Force	Ordinary Case	M (t.m)	-3,239	-3,511	-1,452	+1,681	-1,278	-1,395	+4,948
		N (t)	-1,950	-3,019	-4,554	-3,809	-4,005	-2,901	+27
	Seismic Case	M (t.m)	-2,927	-3,583	-5,144	+2,353	-2,124	-399	+3,405
		N (t)	-2,074	-3,420	-3,896	-3,953	-3,033	-1,950	+22
Thickness	Upper Flange	(mm)	12	12	12	14	12	12	12
	Lower Flange	(mm)	12	12	12	14	12	12	12
	Outer Web	(mm)	12	12	12	14	12	12	12
	Inner Web	(mm)	12	12	12	12	12	12	12

CROSS SECTION
OF MAIN GIRDER



7.4.4 Planning and Preliminary Design of Approach Bridges

1) *General*

The number of the types of the approach bridges to be examined will become large when considering the cases of 3 clearances on both Likoni and Mombasa sides where plan and profile are aligned differently. Particularly in Mombasa side, there are the ramps that access to the Nyerere Ave.

In this study, only an essential part on the planning and design is described, avoiding the bulk of the comparison amongst the cases involved.

2) Concepts on the Selection of the Foundations and Substructures of the Approach Bridges

When selecting the type of the foundation of the approach bridge, the following shall be taken into consideration.

- Fairly large-scale superstructures and high piers, particularly these close to the over-sea bridge, require solid and reliable foundation.
- Care shall be directed to a non-uniform settlement (sinking) when superstructures are continuous type.

The subsoil conditions at the proposed site have the following characteristics that make a difficult foundation design.

- Longitudinal (along the alignment) discontinuity of the formation of soils (layers).
- The coral layers contain in places different grades of weathered corals some of which contain pores and cavities. The corals vary from the ones having unconfined compression test results of 70 kg/cm^2 to those of weathered to sandy soils.
- Even rocky corals may possibly fail and consolidate when weathering will further develop.

  Some accidents of the collapse of spread foundation are reported in ASCE, GEOTECHNICAL JOURNAL (August 1975). . . . . Failures in Limestones in Humid Subtropics by George. F. Sowers, F. ASCE. The corals thus have uncertainties in terms of their bearing capacity which are to be clarified through further research and investigation.
- The silty layers that exist in places in the sandy layer may be consolidated when subjected to heavy and continuous loading.

Although there are many uncertainties as mentioned above, the following foundation type is selected.

(1) The approach bridges close to the main bridge will be planned as a continuous P.C girder having high piers. Taking into consideration the fairly large-scale of the bridges, the foundation is designed to be a piled foundation founded in the status of the hard sandy layer underneath the sandy layer (BH – 4 at the depth of about 10 m) and the

coral layer (BH – 3). Piles are cast by the reverse-circulation method having diameters of ϕ3.0 m, ϕ2.5 m and ϕ2.0 m.

(2) The approach spans located towards the ends of the bridge can use the more economical spread foundation due to relatively small reaction forces. The allowable bearing capacity of 20 t/m² can be used for design as do the existing high- and mid-rise buildings on Mombasa Island.

The substructures are selected as wall type or cellar box type for the following reasons.

(1) The rigid-frame high piers used for the continuous P.C. girder are cellar box type.

(2) To avoid visual obstruction, the piers of the approach bridges in built-up urban areas are selected as a T-shaped type for the low pier and box type for the high pier.

PIER TYPE	HEIGHT (H) 10 20 30 40 m	EVALUATION
(1) T-TYPE, WALL TYPE		○
(2) RIGID FRAME TYPE		
(3) I-SECTION WALL TYPE		
(4) BOX TYPE		○

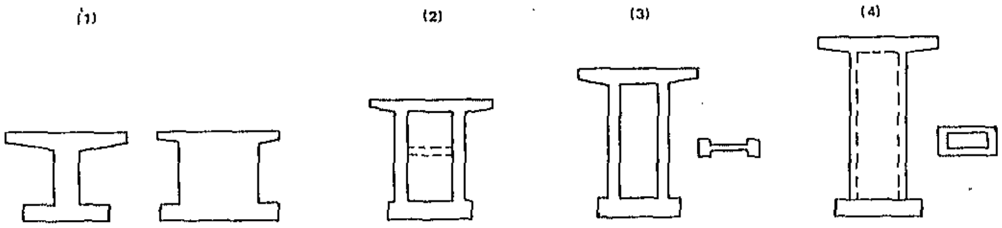


Fig. 7.4.4 TYPES OF PIER AND ITS APPLICABLE HEIGHT

3) Concepts on the Selection of Superstructures

The optimum type of superstructures shall be selected in relation to span lengths and other conditions involved.

In this study, 3 types of Bridge (R.C Hollow slab, P.C T-Beam and P.C continuous box with rigid-frame) were selected by reference to Table 7.4.3 and for the following reasons.

- Steel bridge will not be economical due to the comparatively small seismic loads and fine subsoil conditions. It may not be preferred to use different designs of superstructures mainly from aesthetics and construction economy.

- R.C hollow slab that may be cast in-situ and will be used for the bridges having less than 15 m span and clearance.

Table 7.4.3 COMPARATIVE EVALUATION FOR TYPE OF SUPERSTRUCTURE AND ITS APPLICABLE SPAN LENGTH

	Type of Superstructure	Standard Applicable Span: Bridge Span (m) (10 20 30 40 50 60 70)	Comparative Evaluation: Ride quality	Appearance: Side Elevation	Appearance: Soffit Appearance	Execution & Maintenance: Availability of Materials	Execution & Maintenance: Control of Quality	Execution & Maintenance: Construction Period	Execution & Maintenance: Disturbance to Traffic during construction	Execution & Maintenance: Ease of Construction	Execution & Maintenance: Maintainability	Application to Special Types of Bridge: Curved Bridge of small radius	Application to Special Types of Bridge: High Piers	Application to Special Types of Bridge: Tapered Decks	Economy in Construction (Including substructure)	Total Evaluation
R C	R.C. Simple Girder		B	B	C	A	B	B	C	B	A				A	
R C	R.C. Hollow Slab		A	A	A	A	B	B	C	B	A	O		O	A	O
R C	R.C. Rigid Frame		A	C	C	A	B	B	C	B	A				A	
P C	P.C. Hollow Slab		A	A	A	B	B	B	C	B	A	O			B	
P C	P.C. Simple Composite Girder		B	B	C	B	B	A	A	B	A				B	
P C	P.C. Simple T. Girder		B	B	C	B	B	A	A	B	A				B	O
P C	P.C. Simple Box Girder		B	B	A	B	B	C	C	C	A				B	
P C	P.C. Continuous Box Birder (In-situ)		A	A	A	B	B	C	C	C	A	O			B	
P C	P.C. Box Girder (Incremental Launching)		A	A	A	B	B	C	C	C	A	O	O			
P C	P.C. Continuous Box Girder (Cantilever)		A	A	A	B	B	C	C	C	A	O	O		B	O
Steel	Steel Simple Composite Girder		B	B	C	C	A	A	A	A	C		O	O	C	
Steel	Steel Simple Box Girder		B	B	B	C	A	A	A	A	C		O		C	
Steel	Steel Continuous Box Girder		A	A	B	C	A	A	A	A	C	O	O		C	

Note: A: Excelent
B: Normal
C: Inferior

– Post-Tensioning P.C T-beam will be used for the bridges having the span length of 25 ∿ 40 m and the clearance of 15 ∿ 25 m.

– P.C continuous box with rigid-frame high piers will be used for the bridges having the span of more than 50 m. Erection will be done by the "cantilever method".

The R.C hollow slab and P.C T-beam are provisionally considered to be simply-supported; however, if the future studies allow, a continuous beam is preferred in view of driving comfort by reducing the number of expansion joints.

4) Determination of Span Arrangements and Preliminary Design

Determination of span arrangements is an essential part of the bridge planning and may crucially affect the total economy. In this study, the following conditions and factors are considered.

(1) Conditions of Existing Arterial Road

– Existing width and clearance shall be maintained for the Nyerere Ave., Mbaraki Road, Mbaraki Birth Road on the island and Sherry Beach Hotel Road on Likoni side.

– The intersection of the Nyerere Ave. and Mbaraki Road follows the intersection planning as described in sub-section of 7.3.

– In the case of H = 73.2 m, a clearance shall be maintained against the freight-depot rails.

(2) A rigid-frame structure will become more economic when the ratio of its span (L) to height (H) be approximately $L/H = 1.5$.

(3) A rigid-frame structure will become more economic and advantageous for erection when the ratio of its mid-span (L_1) to side spans (L_2) is $L_2/L_1 = 0.5 \sim 0.8$.

(4) The separated (in and out bound) bridges are for aesthetics to have the piers on the same transverse alignment.

(5) Piers should be located under the "nose" of converging or diverging alignments where expansion joints are also to be provided.

The results and the types of foundation piles determined by the preliminary design are shown in Tables 7.4.4 and 7.4.5.

Table 7.4.4 PROJECT BRIDGE LENGTH

(Unit: Metre)

	Navigation Clearance		
	73.2 M	55 M	45 M
Main Bridge (Cable-Stayed Bridge)	830 x 2 =1,660	1,660	1,660
Approach Bridge			
Likoni Side			
In-Bound	940	505	285
Out-Bound	940	505	285
Island Side			
In-Bound	1,545	695	635
Out-Bound	1,545	815	650
Access in Phase-I	1,040	650	575
Total Length	7,670	4,830	4,090

Note: The above figures indicate the length of 2-lane bridge.

Table 7.4.5 LIST OF APPROACH BRIDGES

(1) 73.2^M

	Station (Km)	Type	Width (m)	Length (m)	Span Arrangements (m)	Foundation (m)
Likoni "Out"	-0.415 ∿ -0.860	PC Rigid Frame	11.0	445	60 + 5@65 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	-0.860 ∿ -1.130	PC T-Beam	11.0	270	30 x 9 spans	Spread Footing
"	-1.110 ∿ -1.355	RC Hollow	11.0	225	15 x 15 spans	"
Likoni "In"	-0.415 ∿ -0.860	PC Rigid Frame	10.5	445	60 + 5@65 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	-0.860 ∿ -1.130	PC T-Beam	10.5	270	30 x 9 spans	Spread Footing
"	-1.130 ∿ -1.355	RC Hollow	10.5	225	15 x 15 spans	"
Mombasa "Out"	0.415 ∿ 0.775	PC Rigid Frame	10.5	360	75 + 2@105 + 75	Pile (RCD ϕ = 3.0)
"	0.775 ∿ 1.065	"	10.5	290	2@50 + 2@65 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	1.065 ∿ 1.395	PC T-Beam	10.5	330	30 x 11 spans	Spread Footing
"	1.395 ∿ 1.800	RC Hollow	10.5	405	15 x 27 spans	"
"	1.800 ∿ 1.840	PC T-Beam	10.5	40	40 x 1 span	"
"	1.840 ∿ 1.960	RC Hollow	10.5	120	15 x 8 spans	"
Mombasa "In"	1.415 ∿ 1.775	PC Rigid Frame	10.5	360	75 + 2@105 + 75	Pile (RCD, ϕ = 2.5, 3.0)
"	1.775 ∿ 1.155	"	10.5	380	60 + 4@65 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	1.155 ∿ 1.395	"	10.5	240	4@60	Spread Footing
"	1.395 ∿ 1.605	PC T-Beam	10.5	210	30 x 7 spans	"
"	1.605 ∿ 1.800	RC Hollow	10.5	195	15 x 13 spans	"
"	1.800 ∿ 1.840	PC T-Beam	10.5	40	40 x 1 span	"
"	1.840 ∿ 1.960	RC Hollow	10.5	120	15 x 8 spans	"

Note: 1) The location of STA 0+00 is in the middle of the existing channel.
2) The plus stations (length) are forward the Mombasa Island from STA 0+00
3) The minus stations (length) are forward the South Mainland from STA 0+00

Table 7.4.5 LIST OF APPROACH BRIDGES

(2) 73.2^M

	Station (Km)	Type	Width (m)	Length (m)	Span Arrangements (m)	Foundation (m)
Railway Cross, Frontage Road	1.650 ∿ 1.800	RC Hollow	11.75	150	15 x 10 spans	Spread Footing
"	1.800 ∿ 1.840	PC T-Beam	11.75	40	40 x 1 span	"
"	1.840 ∿ 1.960	RC Hollow	11.75	120	15 x 8 spans	"
"	1.650 ∿ 1.800	RC Hollow	11.75	150	15 x 10 spans	"
"	1.800 ∿ 1.840	PC T-Beam	11.75	40	40 x 1 span	"
"	1.840 ∿ 1.960	RC Hollow	11.75	120	15 x 8 spans	"
Access to the Nyerere Ave.	0.415 ∿ 0.755	PC Rigid Frame	12.0	340	70 + 2@100 + 70	Pile (RCD, ϕ = 3.0)
"	0.755 ∿ 1.095	"	12.0	340	70 + 2@100 + 70	Pile (RCD, ϕ = 2.5, 3.0)
"	1.095 ∿ 1.215	PC T-Beam	12.0	120	40 x 3 spans	Spread Footing
"	1.215 ∿ 1.335	"	12.0	120	30 x 4 spans	"
"	1.335 ∿ 1.455	RC Hollow	12.0	120	15 x 8 spans	"

Table 7.4.5 LIST OF APPROACH BRIDGES

(3) 55^{M}

	Station (Km)	Type	Width (m)	Length (m)	Span Arrangements (m)	Foundation (m)
Likoni "Out"	-0.415 ∿ -0.605	PC Rigid Frame	11.0	190	60 + 70 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	-0.605 ∿ -0.845	PC T-Beam	11.0	240	30 x 8 spans	Spread Footing
"	-0.845 ∿ -0.920	RC Hollow	11.0	75	15 x 5 spans	"
Likoni "In"	-0.415 ∿ -0.605	PC Rigid Frame	10.5	190	60 + 70 + 60	Pile (RCD, ϕ = 2.0, 2.5)
"	-0.605 ∿ -0.845	PC T-Beam	10.5	240	30 x 8 spans	Spread Footing
"	-0.845 ∿ -0.920	PC Hollow	10.5	75	15 x 5 spans	"
Mombasa "Out"	0.415 ∿ 0.580	PC Rigid Frame	11.0 ∿ 21.5	165	50 + 65 + 50	Pile (RCD, ϕ 2.0, ϕ 2.5)
"	0.580 ∿ 0.760	PC T-Beam	10.5	180	30 x 6 spans	Spread Footing
"	0.760 ∿ 1.020	PC Rigid Frame	10.5	260	40 + 3@60 + 40	Pile (RCD, ϕ 2,0, ϕ 2.5)
Mombasa "Out"	1.020 ∿ 1.080	PC T-Beam	10.5	60	30 x 2 spans	Spread Footing
"	1.080 ∿ 1.395	RC Hollow	10.5	315	15 x 21 spans	"
Mombasa "In"	0.415 ∿ 0.580	PC Rigid Frame	10.5	165	50 + 65 + 50	Pile (RCD, ϕ2,0, ϕ2.5)
"	0.580 ∿ 0.730	PC T-Beam	10.5 ∿ 16.75	150	30 x 5 spans	Spread Footing
"	0.730 ∿ 0.820	"	10.5	90	30 x 3 spans	"
"	0.820 ∿ 0.870	"	10.5	50	25 x 2 spans	"
"	0.870 ∿ 0.900	"	10.5	30	30 x 1 span	"
"	0.900 ∿ 0.110	RC Hollow	10.5	210	15 x 14 spans	"
Access to the Nyerere Ave.	0.580 ∿ 0.730	PC T-Beam	11.0	150	30 x 5 spans	"
"	0.730 ∿ 0.820	"	17.25 ∿ 11.0	90	30 x 3 spans	"
"	0.820 ∿ 0.900	"	11.0	80	40 x 2 spans	"
"	0.900 ∿ 1.065	RC Hollow	11.0	165	15 x 11 spans	"

Table 7.4.5 LIST OF APPROACH BRIDGES

(4) 45^M

	Station (Km)	Type	Width (m)	Length (m)	Span Arrangements (m)	Foundation (m)
Likoni "Out"	-0.415 ∿ -0.595	PC T-Beam	11.0	180	30 x 6 spans	Spread Footing
"	-0.595 ∿ -0.700	RC Hollow	11.0	105	15 x 7 spans	"
Likoni "In"	-0.415 ∿ -0.595	PC T-Beam	10.5	180	30 x 6 spans	"
"	-0.595 ∿ -0.700	RC Hollow	10.5	105	15 x 7 spans	"
Mombasa "Out"	0.415 ∿ 0.580	PC T-Beam	11.0 ∿ 21.5	165	33 x 5 spans	"
"	0.580 ∿ 0.760	PC T-Beam	10.5	180	30 x 6 spans	"
"	0.760 ∿ 1.020	PC Rigid Frame	10.5	260	40 + 3@60 + 40	Pile (RCD, ϕ = 2,0, 3.0)
"	1.020 ∿ 1.230	RC Hollow	10.5	210	15 x 14 spans	Spread Footing
Mombasa "In"	0 415 ∿ 0.580	PC T-Beam	10.5	165	33 x 5 spans	"
"	0.580 ∿ 0.730	"	10.5 ∿ 16.75	150	30 x 5 spans	"
"	0.730 ∿ 0.820	"	10.5	90	30 x 3 spans	"
"	0.820 ∿ 0.870	"	10.5	50	25 x 2 spans	"
"	0.870 ∿ 0.900	"	10.5	30	30 x 1 span	"
"	0.900 ∿ 1.050	RC Hollow	10.5	150	15 x 10 spans	"
Access to the Nyerere Ave.	0.580 ∿ 0.730	PC T-Beam	11.0	150	30 x 5 spans	"
"	0.730 ∿ 0.820	"	11.0 ∿ 17.25	90	30 x 3 spans	"
"	0.820 ∿ 0.900	"	11.0	80	40 x 2 spans	"
"	0.900 ∿ 0.990	"	11.0	90	15 x 6 spans	"

7.4.5 Eerection Plan

1) Preface

The basic concepts on the construction and erection of the P.C and steel cable-stayed bridges, and the approach bridges are described here but are subject to adjustments and modifications.

- How the existing ferry boats can be utilized for the construction. (In this study, the use of special barges is considered to transfer equipment and materials).
- Stock-yards and batching plant to be located on each side of the channel. Naturally, particular effort must be directed to prevent hampering navigation by the use of under-sea cabling, navigation - aids, navigation clearances, etc.

2) Construction of the P.C Cable-Stayed Bridge

The construction of the P.C cable-stayed bridge shall comply with the work procedures as shown in Fig. 7.4.5 and shall be executed simultaneously on both the Likoni and Mombasa sides.

(1) Foundation

The construction of foundation piles will be carried out after excavation reaches the bottom of the footings. The excavation for piles will be carried out by so-called "reverse-circulation" method. Caseful control of operations will be required to maintain the gravity and level of the "bentonite" in the casing tubes used in protecting the excavated holes. The construction shall follow the following process.

Driving of guide piles
↓
Setting of guides
↓
Installation of casing tubes
↓
Setting of excavator
↓
Excavation
↓
Removal of Excavator
↓
Placing reinforcements
↓
Slime treatment
↓
Placing concrete
↓
Treatment of piles heads
↓
Removal of casing tubes of piles heads

Two sets of excavators shall be used one each on the Likoni and Mombasa sides. The excavators shall be transferred to the construction of the side spans after completion of the foundations of the main piers. The application of the reverse-circulation method to subsoil (such as corals) of high permeability remains to be further researched, thus requiring test piling prior to the construction.

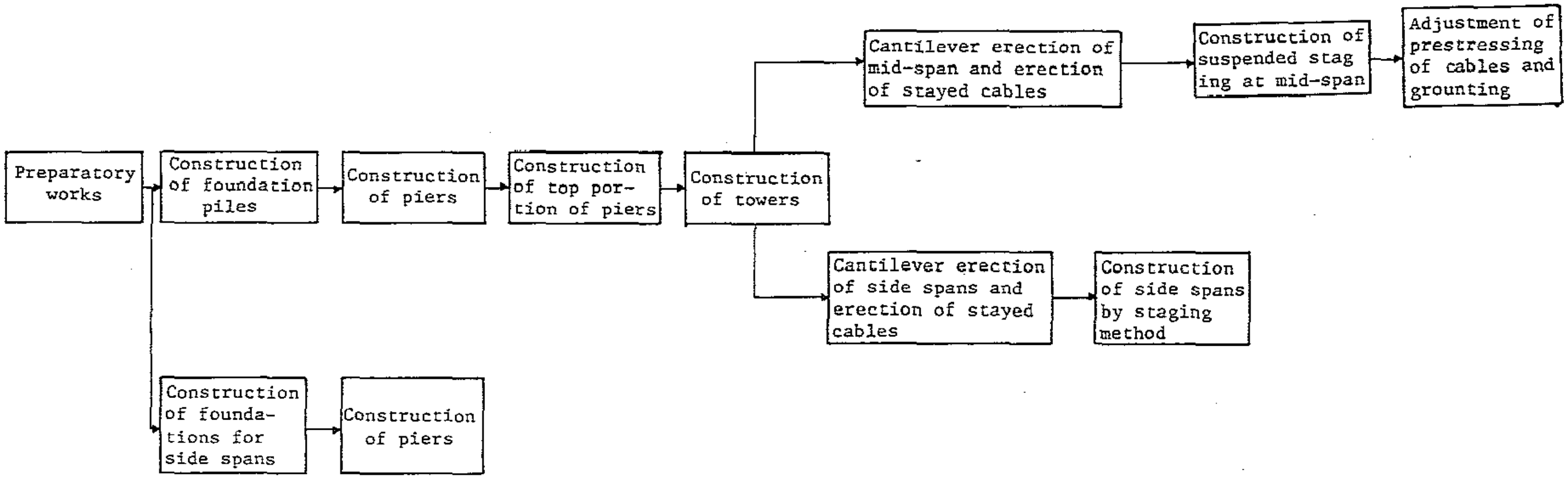


Fig. 7.4.5 WORK FLOW CHART OF P.C CABLE STAYED BRIDGE

(2) Construction of Piers and Piers Heads

The lower portion of piers may be cast by a staging method but after casting to a certain height, a "climbing shutter" method will be applied to cast the remaining portions (dual piers).

Lateral beams will be concreted by using suspended forms supported by steel beams. The piers of side spans will be constructed by a conventional shutter method by using large panel forms. In both the climbing-shutter and panel-form methods, the depth of one pour shall be 5 m and concrete shall be placed by concrete pumps.

The construction of the main girder above the piers shall be carried out by staging built on the lateral beam and brackets anchored on both sides of the piers. To obtain stability during the cantilever erection, the main girder shall be temporarily fixed with the piers and supported by the towers through "Teflon" shoes.

(3) Construction of Tower

The tower shall be constructed by the climbing-shutter method. Equipment and materials shall be handled by a tower-crane.

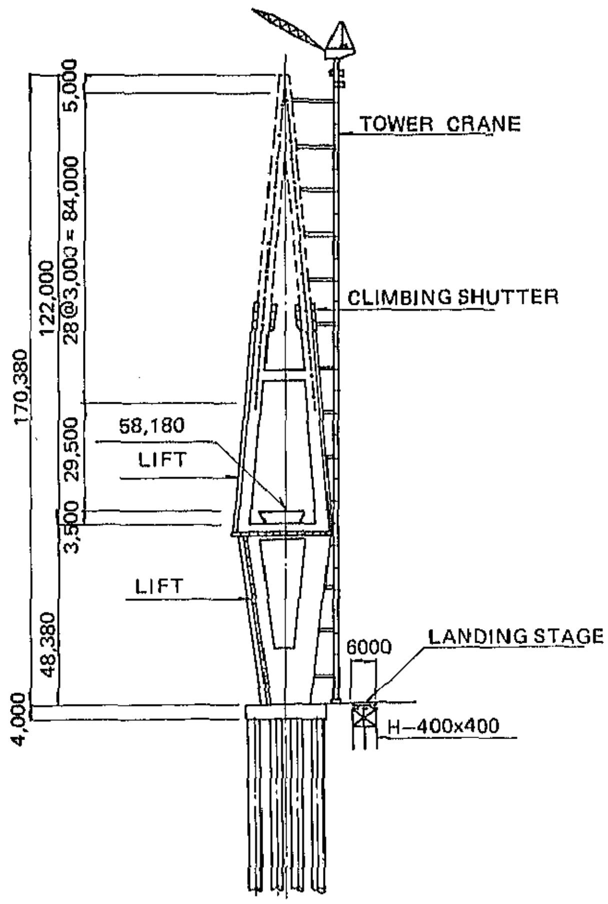


Fig. 7.4.6 CONSTRUCTION OF TOWER

(4) Construction of Main Girder

After building up a medium-size "wagen" (working chamber) on the side-span side and casting one section of the bridge beam, a wagen shall be built up on the main-span side. After casting one block on the main-span side, then each section shall be cast alternately on both sides keeping balance.

The length of one section shall be 4 m in consideration of the capacity of the wagen, but after arriving at the intermediate piers (of the side spans), the length of one section shall be changed to 3 m.

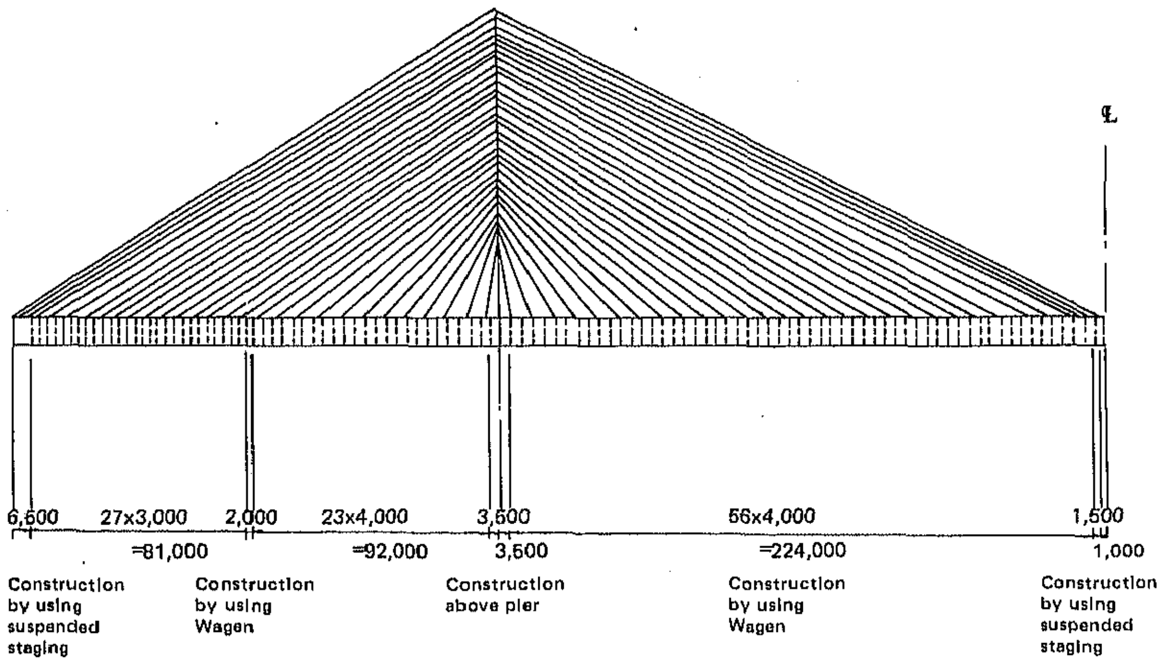


Fig. 7.4.7 CONSTRUCTION OF MAIN GIRDER

One cycle working process by the wagen is as follows:

- Moving and setting of the wagen
- Assembly of forms
- Arrangements of reinforcement bars and P.C. strands
- Placing and curing of concrete
- Pre-stressing of P.C strands and removal of forms
- Fixing of stay cables and stressing of the beam section.

After completion of the cantilever erection in both main and side spans, the closing section shall be cast by using a suspended staging.

(5) Erection of Stay Cables

The polyethylene tubes shall be first erected and connected to the jointing tubes previously attached to the towers. Wire strands shall be fed into the tubes one by one and then stressed. The erected cables shall be stressed twice, at the time of completion of the erection and during the erection of main beam. After checking the applied stresses, the cables will be grouted.

3) Construction of Steel Cable-stayed Bridge

The construction of towers and piers, foundations and substructures remain the same as the case of P.C cable-stayed bridge.

(1) Transportation

The bridge elements which have been completely assembled in work shops, will be shipped to the site according to the erection schedule. The weight of the elements that can be shipped at one time, will be approximately 3,000 ∿ 6,000 tons and the shipment shall be made by a heavy derrick cargo ship (6,000 ∿ 20,000 tons, derrick capacity about 40 tons) specially chartered for the purpose.

The elements thus shipped will be discharged at the berth of the Kilindini Harbour directly to trailer trucks and hauled to storage yards. Upon arriving at the storage yards, the elements will be unloaded by using heavy truck cranes or crawler cranes. To haul to the Likoni Side, a special barge will be used.

(2) Erection (Construction of Superstructure)

The work flow for piling, footing and substructure is the same as that of P.C cable stayed bridge. The overall work flow is shown in Fig. 7.4.8.

(3) Election of Side Spans (Fig. 7.4.9)

The superstructures of the side spans will be erected by using a crawler crane.

The maximum weight of steel members will amount to Wmax = 40 ton and the erection height will amount to H = 60 m. This will require the use of a heavy crawler crane with the lifting capacity of about 300 tons.

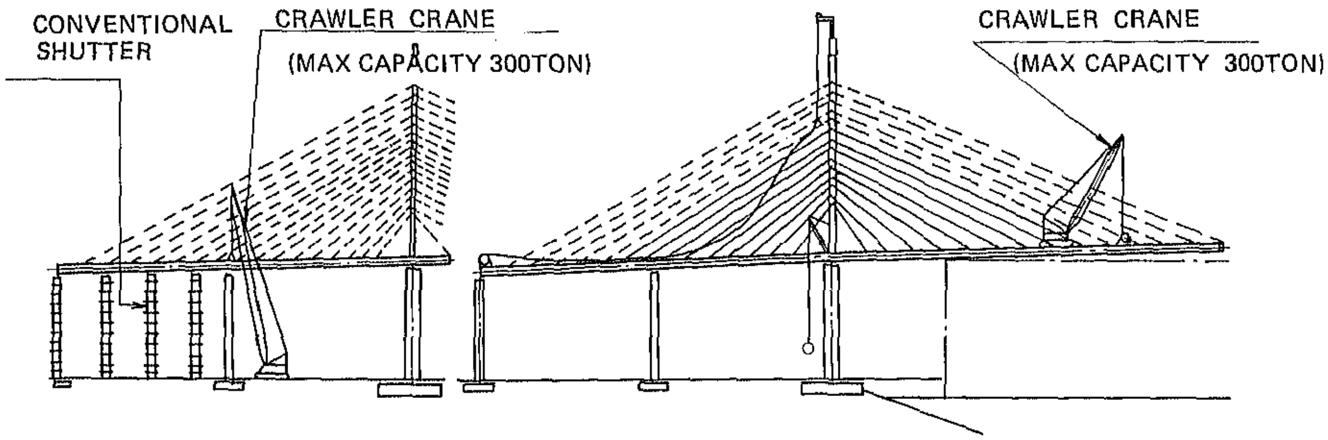


Fig. 7.4.9 ERECTION OF SIDE SPAN

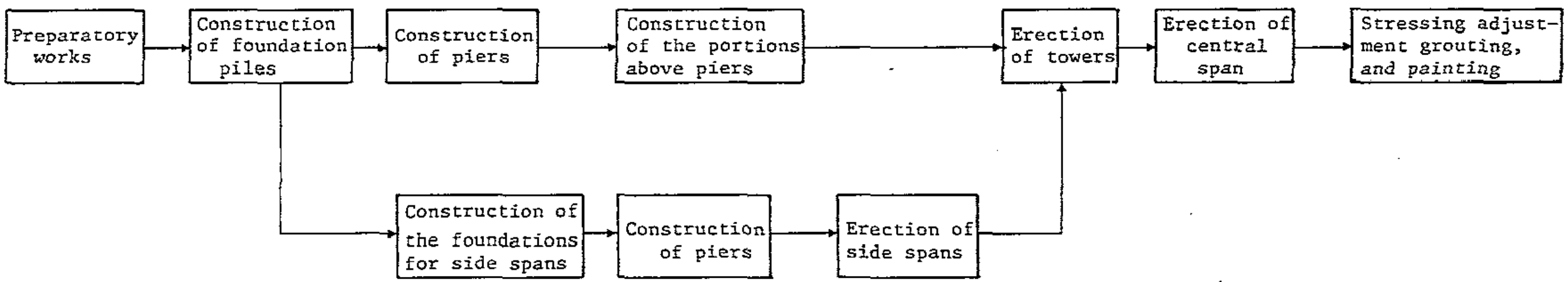


Fig. 7.4.8 WORK FLOW CHART OF STEEL CABLE-STAYED BRIDGE

(4) Erection of Tower

After completion of the erection of side spans, the crawler crane will be disassembled and reassembled on the side spans for erecting the towers.

The disassembly and reassembly of the crawler crane will be made by using a 127-ton truck crane operating at ground level. Lifting heavy parts may require a second 127-ton truck crane. In this case, the 127-ton crane installed on the other bank may be transferred temporarily for the purpose.

As the maximum weight of tower members is Wmax = 40 ton, the 300-ton crawler crane is only capable to lift upto the cable socket approximately 90 m above the bridge deck.

As the towers are A-shaped, the legs portions shall be temporarily supported with struts during the erection. The top portions beyond 90 m above the deck shall be erected by using a jib crane operable in all directions. The capacity of the jib crane shall be 20 tons and shall be self-climbing by using an electric winch equipped on the girder. This jib crane can be also used for fixing cables and other attachments to the tower.
The anchoring frames of the towers shall be installed before the top portions of the piers are placed with concrete. Setting of the anchoring frames requires high-level accuracy in terms of the level and position. The bottom portion of the towers will be installed onto the base plates of the anchoring frames and fixed with anchor bolts.

(5) Erection of Central Span

The central span will be repeatedly cantilevered out and fixed with cables by using crawler cranes at the cantilevered ends.

The maximum weight of steel members will be limited to Wmax = 40 tons. The steel members will be preassembled in the storage yard located beneath the side spans by using 127-ton truck cranes. After the preassembly the members will be lifted up to the bridge deck of the side spans by using separate jib cranes (capacity 40 tons) sited by the towers. The lifted members will then be moved to the cantilever ends by using a truck chassis (capacity 40 tons).

The cantilevered erection shall be carried out simultaneously from both ends of the main span and ultimately closed at the centre of the span.

Because of the length of the cantilevered spans and the weight of the crawler cranes, the steel girders normally require some reinforcement against temporary stresses during the erection.

(6) Fixing Cables, Adjustments, and Stressing

Cables will be shipped in the form of rolled reels and stored at site. Wheeled cables will be set onto the "unwheeler" and expanded on the bridge surface through rollers. Then the cables will be first pulled into the socket of the towers and thereafter into the socket of the girder. The sockets of the tower and girder will be temporarily fixed respectively by using the jib crane and the crawler crane. After temporary fixing, the cables will be stressed by the jacks equipped to the socket of the girder. The jacks consist of tension jacks and ram chairs, and the stressing controlled by oil-gauges. The main span when closed at its mid portion shall be readjusted by re-stressing the cables as the result of checking the camber and the stresses actually applied to the cables.

(7) Remaining Works

After the closure of the central span, grouting of the cables and painting of the field splices will be carried out.

(8) Comparison between 2-lane and 4-lane Compositions

As mentioned in 7.4.3, the shape of the trapizoidal box girder remains the same, regardless of 2-lane or 4-lane composition. Therefore, the maximum dimension of the members will also remain the same. Furthermore, unless the span length be changed, the height of tower can also remain unchanged.

This means there would be no significant changes of equipment and temporary facilities between the cases of 2 lanes and 4 lanes. Conversely, a 2-lane width may be too narrow to easily deck the bridge, thus affecting the work efficiency as compared with the 4-lane case.

4) Construction of the Approach Bridge

(1) Construction of Rigid-frame (Rahmen) Structures

The girder right above piers shall be constructed by using H-shaped beams and brackets, and a wagen shall be assembled at one end.

After casting one section, another wagen shall be assembled at another end and thereafter the girder shall be cantilevered out alternately at both ends. The end sections and the central section of the girder shall be cast by using suspended staging.

(2) Construction of Post-Tensioned P.C T-Beam

Beams shall be cast in the casting yard where a casting bed, mould manufacturing, reinforcement bars and P.C materials, etc., shall be provided. Several casting beds will be required.

After curing and prestressing, forms will be removed and the beams taken to storage. For exection these beams shall be transported using a chassis running on tracks laid on previously erected hollow slab beams. An erection girder will be used for erecting the beams.

(3) R.C Hollow Slab

The hollow slabs will be cast in place on staging and 2 ∿ 4 spans will be cast simultaneously. Ready-mixed concrete will be placed by concrete pumps.

7.4.6 Further Subjects at Detailed Design Stage

A cable-stayed bridge has a high design flexibility. It has a variety of structural types depending on the choice of different combinations of main girder, tension members and tower configurations.

The total stiffness of a cable-stayed bridge is dominated by the interrelationship of the individual stiffness of the girder, cables and tower (pylon). But due to its structural flexibility, a cable-supported system may be subject to considerable dynamic motions induced by wind forces.

The optimum span arrangement, stay geometry, tower (pylon) configuration, girder type, cable type and arrangement, method of anchorage, etc. must be thoroughly investigated. A wind tunnel test and the resultant aerodynamic analysis is most important to determine the structural features of the main girder.

Two dynamically responsive phenomena inherent in a flexible system are the restricted vibration and self-induced vibration. The restricted vibration occurs at relatively low wind speed and causes a fatigue of structural members, etc., while the self-induced vibration occurs at relatively high wind speed and possibly results in a catastrophic damage of the system. These vibrations depend on the wind characteristics (angle of attack, wind speed and their frequencies, etc.) and the configuration of bridge structures, and must be investigated in a wind tunnel.

In the detailed design stage these structural subjects, including type of bridge (P.C and steel), must be further studied associated with investigation and survey (soils and material, topographic survey and material testing, etc.).