approximately ten (10) passenger cars and it is free of charge. At present, therefore the boat is carrying about 1,000 V/D to and from the Kilifi creek. There is only one jetty.

(5) Water Supply Pipe Line

A water supply line which runs from the Kilifi Club to the Mnarani Club is located at the bottom of the sea.

(6) Airstrip

There is one airstrip which is located on the Mnarani side. This is used especially by tourists twice a day.

3.4.4 Future Development Plan for Kilifi Township

The future landuse plan for Kilifi (Kilifi Town side) township was planned by the Ministry of Urban Development and Housing Physical Planning Department. Discussions with Physical Housing Department showed that the study area has no other development plans along the Kilifi creek. The future development plan for the Kilifi township is illustrated in Fig. 3–15. On the other hand the future landuse of the Mnarani side has not been planned and only a conceptional plan is available and this is also illustrated in Fig. 3–15.

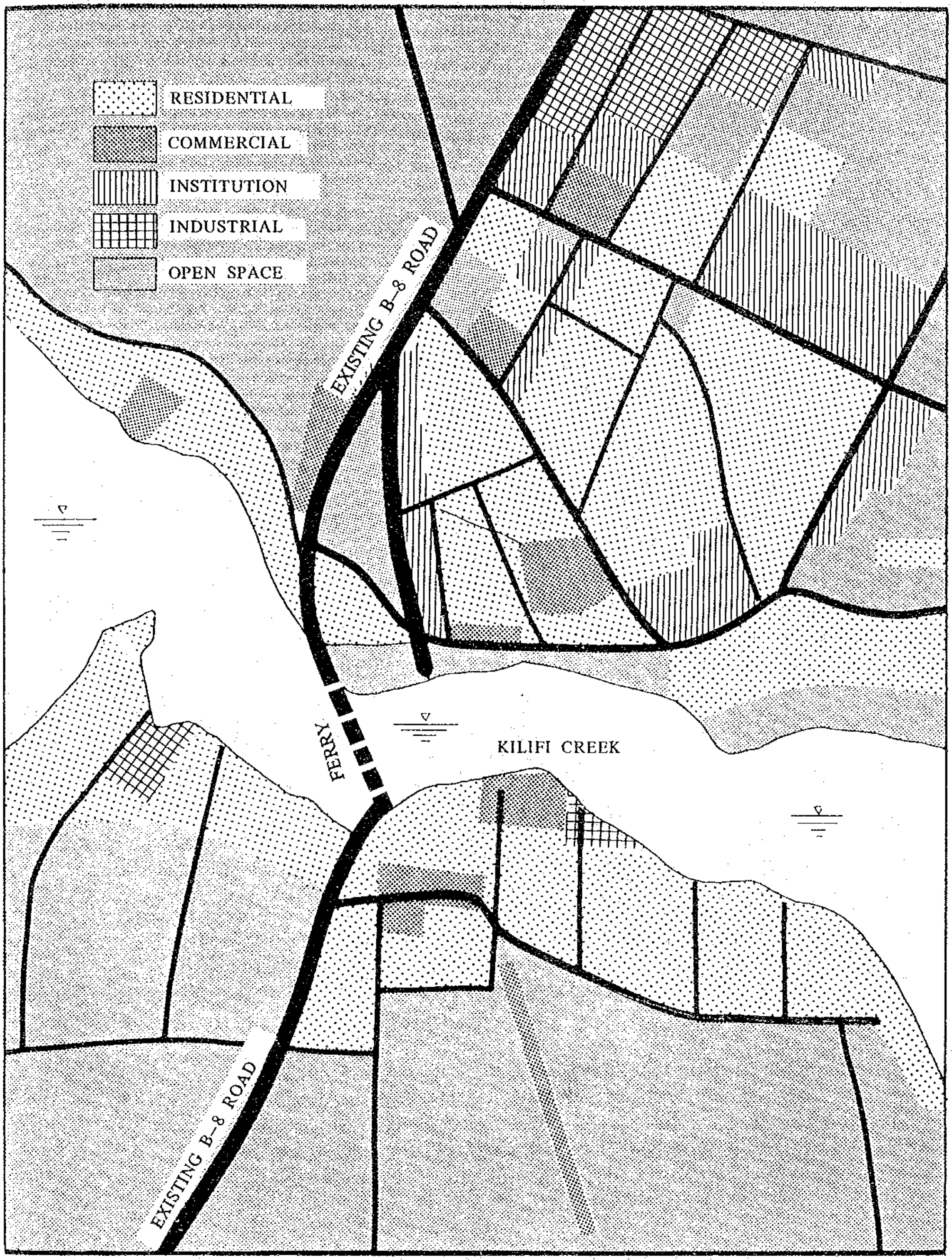


Fig. 3–15 FUTURE DEVELOPMENT PLAN OF KILIFI

4. ALTERNATIVE ROUTE STUDY

4.1 BASIC CONSIDERATION OF ROUTE LOCATION

4.1.1 General

Based on the function of the project road and the environmental considerations of the study area, the basic considerations for the route location are set out as follows.

a) To take into consideration the technical aspect.

b) To take into consideration the economic aspect.

c) To work for public benefit

d) To ensure safety

e) To preserve the good environment.

4.1.2 Control Point of Route Location

For studying the route location the following site conditions were considered:

(1) Kilifi Township

a) The centre business district (C.B.D.) is located in the centre of the Kilifi township. This area is mainly composed of residential, commercial stores and businesses which form the main community in the Kilifi town centre. Taking this point into account, the route location should be avoid passing through this area.

b) Electric Power Line

There are two (2) electric power lines of 132KV and 33KV respectively which pass across the Kilifi Creek. As it is very difficult to remove these two electric power lines, the route location should be such that it does not interfere with these power lines.

c) Telephone and Telegram Communications Line

There is one telegraphic line which passes across the Kilifi Creek, in between the ferry operation line and the 33KV electric power line. This telegraphic line is comparatively easier to shift. Later when the bridge construction is completed this line can be supported by the bridge.

d) Ferry

The ferry is one of the most important means of transportation for vehicles and passengers to and from the Northern/Southern parts of the coastal area. The anchorage of the ferries is located at the upper-stream of the existing ferry operating line. This anchorage area should also be avoided for the route location.

e) Water Supply Pipe Line

It is comparatively easy to remove the water supply pipe line but it is better for the route location to avoid passing through this area.

f) Airstrip

There is a small airstrip for cessna at the southern part of Mnarani Club. This is comparatively easy to remove.

4.1.3 Future Landuse

The future landuse plan of the Kilifi township has already been described. The route location should be avoid the disruption of the communities. According to the development plan of the Kilifi township set by the Ministry of Housing, the plan can be classified under three core or three communities. The route location should not be disrupted the communities and therefore it will have to pass along the boundaries of each of the communities.

4.1.4 Kilifi Creek Conditions

(1) Width of Kilifi Creek

The route location should be selected at the narrowest point of the Kilifi creek. Generally speaking, the construction cost of a bridge is much higher than that of the road earthworks. To minimize the construction cost, it is better to select the narrowest point of the creek.

(2) Depth of Kilifi Creek

The route can be selected at the shallowest depth of the Kilifi creek. To minimize the construction cost of the bridge, a shallowest area is selected.

4.2 DESIGN CRITERIA FOR ROAD

(1) Geometric Design Standards

The geometric design standards were prepared by the Ministry of Works, Road Department in January, 1979. The geometric design standards of this project road is derived from the above mentioned "Geometric Design of Rural Roads". The main design elements are shown in Table 4–1.

Table 4–1 DESIGN ELEMENTS OF ROAD

No.	Items	Unit	Value	Remarks
1	Road class	–	Class – B	
2	Design speed	km/h	100	
3	Terrain	–	Flat	
4	Pavement type	–	Bitumen	
5	Lane width	m	3.5	
6	Shoulder width	m	2.5	
7	Reserve width	m	60	
8	Central Reservation	m	3.5	
9	Maximum Gradient	%	3	Rolling 4%
10	Stopping sight distances	m	140	Minimum
11	Passing sight distances	m	375	– do –
12	Minimum radius	m	600	
13	Superelevation	%	2.5	
14	Vertical curve crest	m	5500	
15	Vertical curve sug	m	3500	

(2) Design Speed

The adopted design speed for this project is 100 km/h. The speed is based on the following considerations.

a) The project road is classified under class B road in Kenya.

b) The running speed of the existing traffic on the existing B–8 road at Kilifi is about 80 km/h to 90 km/h due to the flat nature of the terrain around the area. Therefore the design speed is required to be higher than this running speed to ensure on undisturbed traffic flow.

c) The function of the project road is that of providing mobility for long distance trips which require that the road be designed to a higher design speed.

(3) Design Vehicles

For geometric design purposes, a semitrailer is adopted as the "design vehicle" due to project road having the definition of a primary distributor road.

(4) Intersection Design Standards

At-grade intersection

The at-grade intersection design standards are adopted from "A Policy on Design of Highways Arterial Streets".

(5) Pavement Design Standard

The pavement design standards are adopted from the road design manual which is prepared by MOTC in May,1981.

4.3 PROPOSED ALTERNATIVE ROUTES

4.3.1 Preparation of Proposed Alternative Routes

Taking into account the accurate reconnaissance survey, the basic considerations of the route location, the site conditions, three alternative routes: route A, route B and route C are proposed.

4.3.2 Description of Proposed Alternative Routes

(1) Alternative Route A

This route A is selected at the shallowest depth of the Kilifi creek which is located at 450 m upstream from the 132KV electric power line and which runs parallel to this electric power line.

It mainly passes through an open field area and a residential area which is located along the seashore. There is a ford which is located 300 meters offshore. The depth of this ford is between two(2) and seven(7) meters.

(2) Alternative Route B

Route B is selected at the narrowest section of the Kilifi creek through which the telephone and telegram communication line passes and which is 400 meters downstream from the existing ferry operating line. The width at this point of this creek is between 330 and 350 meters. The approach road on the Kilifi side passes through the open field which has cushewnut trees etc. and the approach road on the Mnarani side passes through the residential area. The route B should be avoid the 33KV electric power line, which cannot be shifted due to economic cost but the telephone and telegram communication line, if cannot be avoided, can be shifted. The length of the bridge is about 400 meters and the length of approach roads are about 3,770 meters (1,350 meters Kilifi side and 2,420 meters on Mnarani side). The total road length is about 4,170 meters.

(3) Alternative Route C

Route C is selected at a comparatively narrow and comparatively shallow area of the Kilifi creek. There is a ford which is located in front of the Kilifi Club, the depth of which is between 2 and 7 meters. This area is 150 meters off-shore. The approach road on Kilifi side is so located as to avoid the existing housing area but can not avoid the future housing plan which was planned by the Ministry of Housing. On the other hand, the approach road on the Mnarani side can be so located as to avoid the airstrip. The route passes through most of an open field which is being operated as a stock farm. The length of the bridge is about 610 meters and the approach roads are about 7,130 meters (2,720 meters on Kilifi side and 4,410 meters on Mnarani side).

The three alternative routes, namely route A, route B and route C are illustrated in Fig. 4–1.

4.4 SCREENING OF ALTERNATIVE ROUTES

4.4.1 Comparison of Alternative Routes

Table 4–2 shows the comparison of the characteristics of the alternative routes.

4.4.2 Selection

From table 4–2 the following evaluations are given:

(1) From Economical Points of View

Route B is the most economical

Route A and C are 1.60 and 1.55 times more expensive than route B respectively.

(2) From Technical Points of View

a) Route B has the bridge with the shortest total length.

b) Route B bridge has the smallest number of foundations under water.

c) Route B has the most favourable horizontal alignment and can be connected to the existing road most easily.

d) The construction of the approach roads of routes B and C will be easier than route A because they run through a flat terrain. Route A requires a great deal of excavation, embankment and small scale bridges because it runs through a rolling terrain.

e) Route B has some problems in relation to the existing facilities such as:

i) the telephone lines which should be removed

ii) the airstrip located on Mnarani side which may be removed in case a bridge type with high tower is required (e.g. the suspension bridge, the cable stayed bridge).

f) Route B is the most accessible from the cores of the township in the future development plan. In case of route B, it is possible to use the existing roads for the access roads.

(3) From Environmental Points of View

a) Route A has little influence on the future development plan. Route B is suitable for future development because it runs along the reserved space for the national trunk road in the future development plan. Route C will have undesirable influence on the future development of Kilifi townships because it runs through the central part of the residential area in the future development plan of the Kilifi township.

b) The landscape of the creek will be changed by the appearance of the bridge. The route C bridge that is located closest to the mouth of the creek is considered slightly superior to the others when viewed from the township. However the aesthetical impact is considered to depend on the bridge type rather than the crossing point of the bridge.

4.4.3 Conclusion

Taking into account the economic, technical and environmental aspects, the alternative route B is the most superior plan. With this, the alternative route B is selected.

Table 4-2 COMPARISON OF ALTERNATIVE ROUTES

Item		Route A	Route B	Route C
Topographical condition	Width of creek	620 m	360 m	480 m
	Width of Creek with water depth of more than 25 m	165 m	130 m	150 m
	Maximum water depth of the creek	39 m	36 m	37 m
	Terrain along the route	Rolling	Flat	FLat
Geological Condition		Sand/Sandstone and Shale		
Characteristics of Bridge	Total bridge length	710 m	400 m	610 m
	Cross-section width	12.5 m	12.5 m	12.5 m
	Surface area of bridge	8,875 m	5,000 m	7,625 m
	Alignment	Straight	Straight	Straight
	No. of foundation under water	7	2	5
Characteristics of Approach roads	Total length	4,250 m	3,770 m	7,130 m
	Width	16.0 m	16.0 m	16.0 m
	Min. alignment	800 m	1,000 m	600 m
Relations to housing		Kilifi side ....cut the residential area with a few scattered houses Mnarani side .... cut the agricultural area and residential area with a few scattered houses		
Adjustment for existing facilities		No need	Remove telephone line	No need
Relations to future development plan		Kilifi side........ cut the residential area planned along the creek Mnarani side .... cut the residential area planned along the creek		
Accessibility		Some problems	No problems	Some problems
Land acquisition		25.50 Ha	16.62 Ha	42.78
Rough costs ('000 KShs)	Construction cost of bridge	388,478	243,440	353,123
	Construction cost of approach rd	37,698	20,470	52,690
	Land acquisition cost	3,136	2,044	5,262
	Sub-total	429,312	265,954	411,075
	Engineering fee	64,397	39,893	61,661
	Contingency	42,931	26,595	41,108
	Total	536,460 (1.6)	332,442 (1.0)	513,844 (1.55)

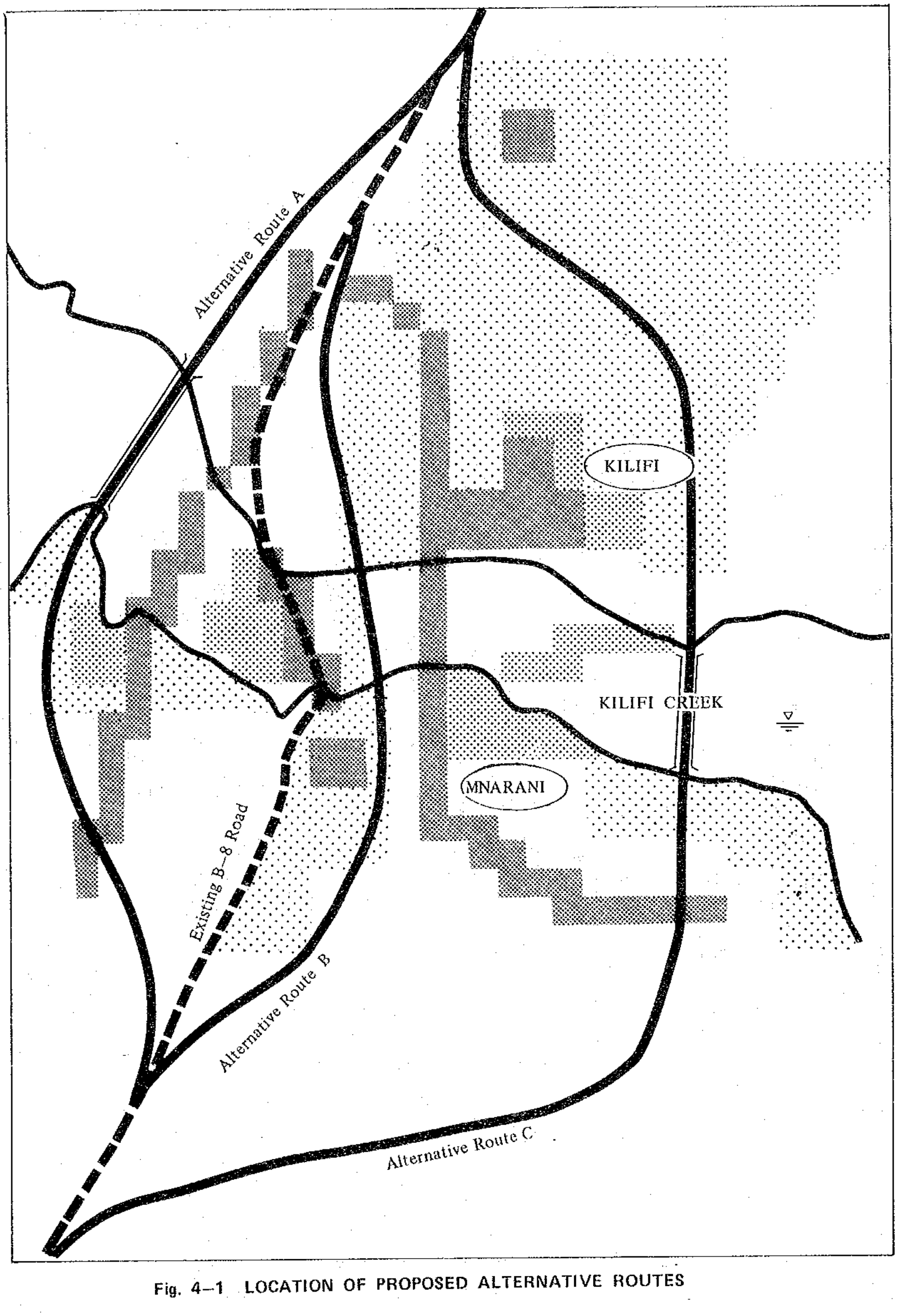


Fig. 4–1 LOCATION OF PROPOSED ALTERNATIVE ROUTES

5 . ALTERNATIVE BRIDGE STUDY

5.1 INTRODUCTION

As a first step, several bridge designs which are regarded as feasible and suitable for the proposed route "B" are selected. For each of these alternative bridge designs, the skeleton and the major configuration of structures are determined, the conceptional work procedure established and the quantity of the major construction materials as well as the construction cost are computed. The study of these rough alternative bridge designs is carried out specifically based on the design criteria identified in Chapter 7.

The evaluation among these bridge alternative designs is performed from three basic perspectives, the economic, technical and environmental perspectives.

5.2 ALTERNATIVE BRIDGE DESIGN

5.2.1 Bridge Length

Considering the profile of the topographic feature and subsurface condition, the bridge length should be 420 meters. The starting point is the Station No. 22 + 35 M and the end point is the Station No. 26 + 55 M.

5.2.2 Positioning of Piers

For the positioning of piers, four cases which are illustrated in Fig. 5–1 are considered. These are based on the following considerations:

a) As the profile of the seabed shows, the central part of the approximate 200 meters width of the creek has a water depth of more than 25 meters. The construction of the foundations of the bridge at a position of such a great water depth will be comparatively difficult and expensive. In order to ensure easier construction and a lower total construction cost of the bridge, the number of piers at this depth should be minimized at not more than two piers.

b) Single-span and double-spans bridges are eliminated because of their huge superstructures that would push up the construction costs enormously.

c) The topographical features is almost symmetrical as shown in the profile, the piers should be placed in symmetry in order that the structural mechanics of the superstructure may be balanced with the desirable scenery.

Case (1) and case (2) are triple-spans bridge designs, while case (3) and case (4) are five-spans bridge designs. In general, an increase in the length of the central span would cause an increase of the construction cost of the superstructure. On the other hand the shallower the water at the piers for the central span are positioned, the lower would be the construction cost for these piers. The four alternatives of piers-positioning are specifically drawn to display these two cost relationships.

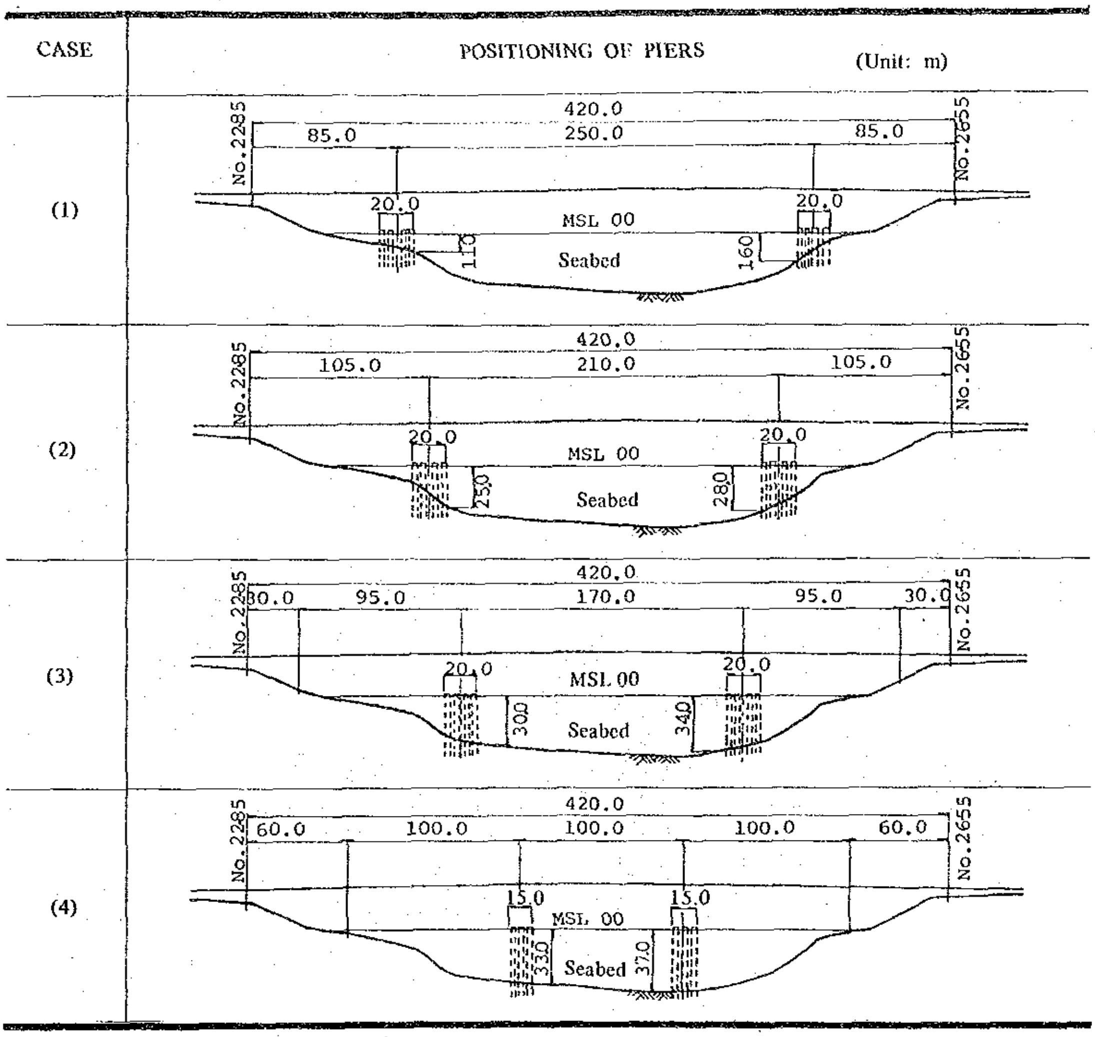


Fig. 5–1 POSITIONING OF PIERS

In case (1), the piers are located at the shallowest position among the four alternatives. In such a case, the landing stage may be used for constructing the piers. In case (3) and case (4), the piers for the flanking spans are so located as near to the abutments. This would allow drywork for their construction.

5.2.3 Superstructure Design

With reference to the four piers-positioning alternatives; case (1) to case (4); the length of the central span of the superstructures would accordingly vary from 100 meters to 250 meters.

Bearing in mind the past concrete construction experiences, the present technical construction level available, and the site conditions, the following five types of structure designs are selected as possible design for this project.

(Steel Bridges)

a) Cable stayed steel girder
b) Steel truss
c) Steel girder (with orthotropic steel deck)

(Concrete Bridges)

d) Cable stayed concrete girder
e) Concrete girder (by free cantilevering-method)

Other design alternatives such as the arch-type and the suspension-type are eliminated basically for their high construction cost for this particular site.

The adaptation of these five alternative structure designs to the variation of the central span length from 100 meters to 250 meters is carefully considered. In general, steel bridges are more feasible than concrete bridges for relatively long span bridges. Further, for both the steel and concrete groups, the girder type is more suitable for relatively short span bridges while the cable-stayed or truss-type are more suitable for long span bridges.

With this reasoning, the proposition of these five structural design alternatives to suit the previously suggested four cases of piers-positioning alternatives is shown in Table 5–1 below. Profiles of these five bridge alternatives namely, alternative (1) to alternative (5) are shown in Fig. 5–2.

Table 5–1 ALTERNATIVE SUPERSTRUCTURE TYPES

Positioning of Piers	Main Span Length	Alternative Superstructure Types
Case (1)	250 m	(1) Cable stayed steel girder (2) Steel truss
Case (2)	210 m	(3) Cable stayed concrete girder
Case (3)	170 m	(4) Concrete girder with a hinge by free cantilevering method
Case 4)	100 m	(5) Concrete girder with three hinges by free cantilevering method

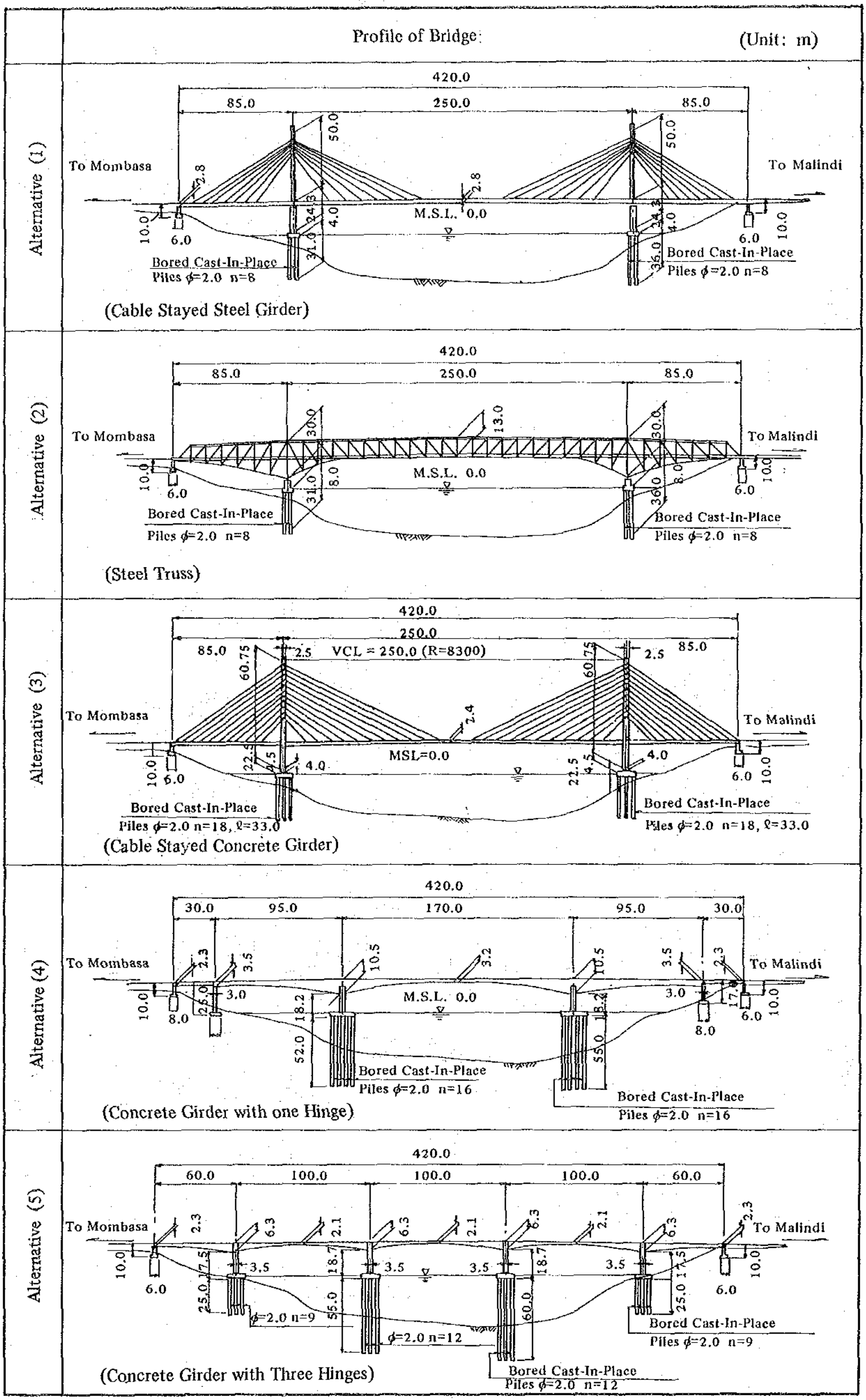


Fig. 5–2 ALTERNATIVE BRIDGE PLAN

5.2.4 Substructure Design

The spread footing type is applied for the abutments and piers on shore. This is done because it is found out that the coral limestone layer which exists at a relatively shallow depth from the ground surface is expected to be a reliable bearing layer for a direct spread footing foundation.

For the foundations of piers under water, we examined many alternative designs which may be applicable to the conditions at this site. These are caisson, pile bent, pile foundation and steel pile sheeting well. The characteristics of these types of foundation are described in the interim report. After examining the various types of foundations, the conclusion is that the pile bent type is the most practical and the most economical foundation type for this bridge.

The pile bent foundation has two alternative kinds which are distinguished by their materials. One is where the pile is made of steel and the other made of reinforced concrete. The steel pile has the advantages over the concrete pile in terms of shorter construction period and reliability of the material. It, however, has the disadvantages related to higher transportation costs of the material and less resistance to corrosion than the concrete pile. Examination of the construction costs for these two kinds of foundations indicate that the concrete pile is cheaper than the steel pile. For these reasons the concrete pile which will be cast in bore holes by means of the reverse circulation drill is recommended.

5.3 CHARACTERISTICS OF BRIDGE ALTERNATIVES

For the five bridge alternatives mentioned earlier, that is alternative (1) to (5), the conceptional construction work procedures are first established. Following this the skeleton of the structures, and the structural calculation are worked out, together with the major configurations of the bridges. The quantities of major materials and the construction costs are then duely computed.

The result of this study will be outlined hereafter.

5.3.1 Major Construction Procedure

(1) Alternative (1) (Cable stayed steel girder)

a) Superstructure

i) Transportation of fabricated steel members that are imported from abroad is made by deck barges from Mombasa. Unloading is done near the site.

ii) Erection of steel members is made gradually by using crane mounted on girders.

iii) Extension of superstructure is made gradually from both the two abutments towards the center simultaneously.

b) Substructure

i) The landing stage is used for construction.

ii) Installation of casing pipes into the creek's bottom is made by using crane mounted on the stage.

iii) Drilling inside the casing is made by using reverse circulation drill from the landing stage.

iv) Casting of concrete is made by using cable crane to made cast-in-place piles and piers.

(2) Alternative (2) (Steel truss)

a) Superstructure

i) Transportation of fabricated steel members that are imported from abroad is made by road from Mombasa.

ii) Erection of steel members is made by crane mounted on upper chords.

iii) Extension of superstructure is made gradually from both the two abutments towards the center.

b) Substructure

As in Alternative (1)

(3) Alterntive (3) (Cable stayed concrete girder)

a) Superstructure

i) Transportation of fabricated cables that are imported from abroad is made by road from Mombasa.

ii) Main towers are made by using tower cranes.

iii) Erection of the first concrete block on the piers is made by using crane mounted on barge and/or simple SEP.

iv) Extension of superstructure is made symmetrically by using wagon together with cable joining from the two piers to the center.

b) Substructure

i) Floating crane and simple SEP are used for the pile construction.

ii) Hoisting and installation of casing pipes are made by using floating crane.

iii) Drilling inside the casing is made by using reverse circulation drill mounted on simple SEP stabilized in creek.

iv) Casting of concrete is made by using cable crane to make cast-in-place piles and piers.

(4) Alternative (4) (Concrete girder by free cantilevering method)
Alternative (5) (Concrete girder by free cantilevering method)

a) Superstructure

i) Transportation of major equipment and materials that are imported is made by road from Mombasa.

ii) Erection of the first concrete block on the piers is made by using crane mounted on floating barge and/or simple SEP.

iii) Extension of superstructure is made symmetrically by using wagon from the two piers to the center.

b) Substructure

As in alternative (3).

5.3.2 Construction Material Quantities

The quantities of the major construction material for the bridge alternative (1) to (5) are tabulated in Table 5–2 and 5–3.

5.3.3 Project Cost

The project cost for the bridge alternatives (1) to (5) is tabulated in Table 5–4.

Table 5–2 STEEL BRIDGE CONSTRUCTION QUANTITIES

ITEM	SUB-ITEM	CLASS	UNIT	ALTERNATIVE (1) CABLE STAYED BR.	ALTERNATIVE (2) TRUSS BR.
Superstructure:–					
Steel	Cable Strand	SWPR (145/165)	T	165	–
	Plate & Shape	SS41, SM50, SM50Y	”	2,575	2,840
	Others		”	194	133
Steel TOTAL			”	2,934	2,973
Painting	Out-side	2-layers	M^2	17,500	48,300
Concrete	Slab	σck = 240 Kg/cm^2	M^3	786	1,380
Form	– do –	Steel Form	M^2	2,290	6,200
Reinforcement	Deformed Bar	SD30	T	55	276
Pavement	Carriageway	Asphalt t = 50 mm	M^2	–	3,570
	– do –	t = 80 mm	M^2	3,570	–
	Pedestrianway	t = 30 mm	M^2	1,680	1,680
Handrail	Aluminium	h = 1.2 m	M	860	860
Guard Rail	Steel	h = 0.9 m	M	850	–
Lighting	Aluminium	h = 10 m	No.	15	15
Substructure:–					
Concrete		σck = 240 Kg/cm^2	M^3	3,337	2,857
Form		Steel Form	M^2	2,786	2,199
Reinforcement	Deformed Bar	SD30	T	251	169
Timbering & Scaffolding		Steel Pipe	M^3	2,085	1,169
Excavation & Filling			M^3	2,031	2,286
Cast-in-Place Pile	R.C.D. ϕ 2.0m.	σck = 300 Kg/cm^2	M	536	536
Temporary Works			Lump Sum	1	1

Table 5–3 P.C. BRIDGE CONSTRUCTION QUANTITIES

Item	Sub-Item	Class	Unit	Alternative (3) Cable Stayed Br.	Alternative (4) 3-Span Girder Br.	Alternative (5) 5-Span Girder Br.
Superstructure:–						
Concrete		σck =350 Kg/cm^2	M^3	4,600	6,460	4,850
Form		Steel Form	M^2	18,400	21,900	17,200
Reinforcement	Deformed Bar	SD30	T	470	560	370
P.C. Rod		SBPR95/110,120	"	265	598	420
Cable	ϕ 7 mm	SWPR(145/165)	"	260	–	–
Shoe			"	6	18	14
Expansion Joint			M	29	62.5	62.5
Erection Equipments			Lump Sum	1	1	1
Timbering			Lump Sum	1	1	1
Pavement	Carriageway	Asphalt t = 50 mm	M^2	3,570	3,570	3,570
	Pedestrianway	t = 30 mm	M^2	1,680	1,680	1,680
	Carriageway	Concrete	M^3	395	395	395
Handrail	Aluminium	h = 1.2 m	M	860	860	860
Guard Rail	Steel	h = 0.9 m	M	1,700	–	–
Lighting	Aluminium	h = 10 m	No.	15	15	15
Substructure:–						
Concrete		σck = 240 Kg/cm^2	M^3	2,017	5,208	5,436
Form		Steel Form	M^2	6,622	3,761	4,515
Reinforcement	Deformed Bar	SD30	T	520	346	404
Scaffolding & Timbering		Steel Pipe	M^3	7,795	4,190	2,673
Excavation & Filling			M^3	1,936	2,180	2,920
Cast-in-Place Pile	R.C.D. ϕ 2.0m	σck = 300 Kg/cm^2	M	1,552	1,712	1,830

Table 5–4 PROJECT COST

(Unit 1,000 K Shs.)

Item	Alternative (1) Cable-Stayed Steel Girder	Alternative (2) Steel Truss	Alternative (3) Cable Stayed Concrete Girder	Alternative(4) Concrete Girder With One Hinge	Alternative (5) Concrete Girder With Three Hinges
Bridge	296,889	282,292	282,655	283,877	274,697
Approach Road	24,239	24,239	24,239	24,239	24,239
Construction Cost	321,128	306,531	306,894	308,116	298,936
Engineering Fees	38,535	36,784	36,872	36,974	35,872
Land Acquisition	2,664	2,664	2,664	2,664	2,664
Compensation	450	450	450	450	450
Sub-Total	362,777	346,429	346,835	348,204	337,922
Contingency	36,278	34,643	34,684	34,820	33,792
Project Cost	399,055	381,072	381,519	383,024	371,714
Maintenance Cost	1,424	6,138	549	669	669

5.4 SUPPLEMENT TO BRIDGE ALTERNATIVES ANALYSIS

The following is a supplementary study to the bridge alternatives analysis as described in the previous section. Specifically, this supplementary study made a further examination on the construction costs of the cable-stayed concrete girder bridge and the concrete girder bridge.

5.4.1 Re-examination of Concrete Girder Bridge

This study is carried out to assess the fact that although the concrete girder proposed in Alternative (4) has a central span of 170 meters in length, a similar kind of bridge with a 210 meters central span proposed as case (2) in the pier-positioning study may have a lower cost compared to the alternative (4). The result of this investigation is shown in Table 5–5 and 5–6. As shown in the tables, the 210 meters center span bridge has a 3% higher construction cost over the original alternative (4) design.

Table 5–5 COMPARISON OF BRIDGE CONSTRUCTION COST

(1,000 K.Shs.)

Item	PC Girder Center Span 210m	PC Girder (Alt. 4) Center Span 170m
Superstructure	83,284	72,930
Substructure	71,039	74,749
Temporary Work	62,600	62,600
Overhead	75,923	73,598
Total	292,846	283,877

Table 5–6 CONSTRUCTION COST OF P.C. GIRDER (ℓ = 210 m)

Item		Unit	Quantities	(K. Shs.) Rate	(1000 K. Shs.) Amount
Superstructure:–	kg/cm^2				
Concrete	σck = 350	M^3	8,000	1,250	10,000
Form		M^2	25,600	210	5,376
Reinforcement		t	640	14,000	8,960
P. C. Rod		t	735	48,000	35,280
Cable		t	–	110,000	–
Shoe		t	17	45,000	765
Expansion Joint		M	62.5	17,000	1,063
Erection Equipment		Lump Sum	1		13,200
Timbering		"	1		4,700
Pavement	Carriageway	M^2	3,570	110	393
	Pedestrianway	M^3	1,680	110	185
	Concrete	M^3	395	1,200	474
Handrail		M	860	2,800	2,408
Guard Rail		M	–	400	–
Lighting		No.	15	32,000	480
SUB-TOTAL (1)					83,284
Substructure:–	kg/cm^2				
Concrete	σck = 240	M^3	4,794	1,200	5,753
Form		M^2	2,260	210	475
Reinforcement		t	384	14,000	5,376
Timber & Scaffolding		M^3		180	755
Excavation & Filling		M^3	2,180	220	480
Cast-in-Place Pile		M	1,940	30,000	58,200
SUB-TOTAL (2)					71,039
Temporary Works (3)		Lump Sum			62,600
TOTAL (4)	(1) + (2) + (3)				216,923
Overhead (5)	(4) x 35%				75,923
GRAND TOTAL	(4) + (5)				292,846

5.4.2 Re-examination of Cable-stayed Concrete Girder Bridge

This brief study is carried out to gauge the difference in construction cost by changing the length of the central span of the cable-stayed concrete girder bridge design. An increase in the length of the central span while maintaining the total bridge length constant generally increases the cost of superstructure. Structurally explained, as the ratio of the length of the central span to the flanking span increases, the force which the stayed cables have to sustain increases. As a result, the negative reaction which have to be opposed by such devices as heavy counterweight also increases. However, there is a limit to the value of the ratio of the central span length to the flanking span length as determined by the structural theory. For a greater length of the span beyond this limit, the total bridge length would have to be extended.

The cost of constructing the foundation for the main tower that amounts to a comparatively large portion of the total construction cost will also varies as the length of the central span increases. As the latter increases, the main tower is shifted towards the land. This would mean a possible decrease in the length and number of foundation piles for the tower. In addition, the construction procedure for the foundation may varies, such as the use of floating crane or/and SEP or landing stage according to the depth of water where the foundation is located.

This brief supplementary study is carried out for the three alternatives shown in Table 5–7.

Table 5–7 VARIATIONS OF ALTERNATIVE (3)

Alt. \ Item	Span Length	Span Ratio
ALT. (3)–1	85 m + 250 m + 85 m = 420 m	(1 : 2.94 : 1)
ALT. (3)–2	95 m + 280 m + 95 m = 470 m	(1 : 2.95 : 1)
ALT. (3)–3	100 m + 320 m + 100 m = 520 m	(1 : 3.20 : 1)

The profiles of these three alternatives are shown in Fig. 5–3.

The structural calculation, the computation for the quantities of major construction materials and the preliminary cost estimates as before are done for these three variations of alternative (3) namely alternative (3)–1, (3)–2, and (3)–3. The result of this analysis is shown in Table 5–8 to 5–12. The result shows the alternative (3)–1, with a central span of 250 meters in length, being the most economical among the three. More detail analysis are:

a) The cost of the superstructure increases almost linearly with increases of the length of central span of the bridge.

b) The cost of substructure for alternative (3)–1 is relatively smaller than the other alternatives.

c) The cost for the foundation of the main tower decreases with the size of the central span in the order of alternative (3)–1, (3)–2 and (3)–3.

(Unit: m)

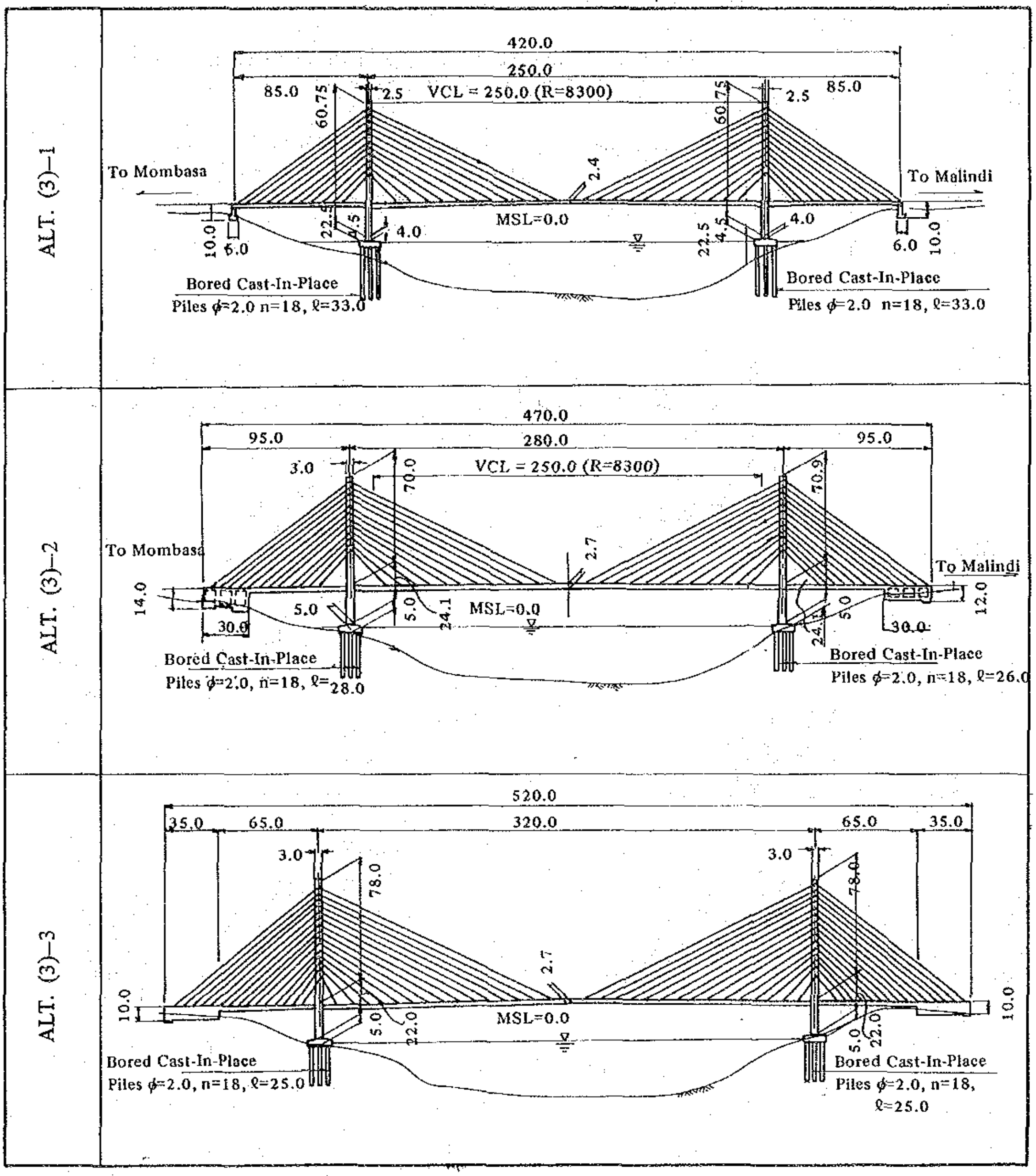


Fig. 5-3 PROFILES OF VARIATIONS OF ALTERNATIVE (3)

However, the high substructure cost for alternative (3)–2, and (3)–3 may be partly attributed to the cost of the heavy counter weight abutment which is needed for their longer central span.

Table 5–8 COMPARISON OF BRIDGE CONSTRUCTION COST (CABLE STAYED CONCRETE GIRDER)

(Unit: 1,000 K.Shs.)

Item	Alt. (3)–1 $\ell = 250m$	Alt. (3)–2 $\ell = 280m$	Alt. (3)–3 $\ell = 320m$	Alt. 3 $\ell = 210m$
Superstructure	89,760	110,159	132,821	81,297
Substructure	45,917	55,535	59,784	65,477
Temporary work	60,000	59,300	57,000	62,600
Overhead	68,487	78,749	87,362	73,281
Total	264,164	303,739	336,967	282,655

Note: ℓ = length of the central span

Table 5–9 CONSTRUCTION COST OF CABLE STAYED CONCRETE GIRDER (ℓ = 250 m) ALT. (3)–1

Item		Unit	Quantity	(K.Shs.) Rate	(1000 K.Shs.) Amount
Superstructure:–					
Concrete	$\sigma ck = \frac{kg/cm^2}{350}$	M^3	4,680	1,250	5,850
	$\sigma ck = \frac{kg/cm^2}{210}$	M^3	840	1,150	966
Form		M	18,900	210	3,969
Reinforcement		t	468	14,000	6,552
P. C. Rod		t	280	48,000	13,440
Cable		t	320	110,000	35,200
Shoe		t	6	45,000	270
Expansion Joint		M	29	17,000	493
Erection Equipment		Lump Sum	1		15,400
Timbering		Lump Sum	1		3,000
Pavement	Carriageway	M^2	3,570	110	393
	Pedestrianway	M^2	1,680	110	185
	Concrete	M^3	395	1,200	474
Handrail		M	860	2,800	2,408
Guard Rail		M	1,700	400	680
Lighting		Each	15	32,000	480
SUB-TOTAL (1)					89,760
Substructure:–					
Concrete	$\sigma ck = \frac{kg/cm^2}{350,300}$	M^3	3,750	1,250	4,688
	$\sigma ck = \frac{kg/cm^2}{240}$	M^3	3,320	1,200	3,984
Form		M^2	6,130	210	1,287
Reinforcement		t	630	14,000	8,820
Timber & Scaffolding		M^3	10,900	180	1,962
Excavation & Filling		M	1,936	220	426
Cast-in-Place Pile		M	990	25,000	24,750
SUB-TOTAL (2)					45,917
Temporary Works (3)		Lump Sum			60,000
TOTAL (4)	(1) + (2) + (3)				195,677
Overhead (5)	(4) x 35%				68,487
GRAND TOTAL	(4) + (5)				264,164

Table 5–10 CONSTRUCTION COST OF CABLE STAYED CONCRETE GIRDER (ℓ = 280 m) ALT. (3)–2

Item		Unit	Quantity	(K.Shs.) Rate	(1000 K.Shs.) Amount
Superstructure:–				KSL	1,000 KSL.
Concrete	$\sigma ck = \frac{kg/cm^2}{350}$	M^2	5,050	1,250	6,313
	$\sigma ck = \frac{kg/cm^2}{210}$	M^2	650	1,150	748
Form		M^2	19,660	210	4,129
Reinforcement		t	505	14,000	7,070
P. C. Rod		t	303	48,000	14,544
Cable		t	435	110,000	47,850
Shoe		t	4	45,000	180
Expansion Joint		M	14.5	17,000	247
Erection Equipment		Lump Sum	1		15,400
Timbering		Lump Sum	1		9,400
Pavement	Carriageway	M^2	3,995	110	439
	Pedestrianway	M^2	1,880	110	207
	Concrete	M^3	405	1,200	486
Handrail		M	940	2,800	2,632
Guard Rail		M	–	400	–
Lighting		Each	16	32,000	512
SUB-TOTAL (1)					110,157
Substructure:–					
Concrete	$\sigma ck \frac{kg/cm^2}{350,300}$	M^3	4,813	1,250	6,016
	$\sigma ck = \frac{kg/cm^2}{240}$	M^3	7,000	1,200	8,400
Form		M^2	8,155	210	1,713
Reinforcement		t	1,127	14,000	15,778
Timber & Scaffolding		M^3	13,179	180	2,372
Excavation & Filling		M^3	8,253	220	1,816
Cast-in-Place Pile		M	972	20,000	19,440
SUB-TOTAL (2)					55,535
Temporary Works (3)		Lump Sum			59,300
TOTAL (4)	(1) + (2) + (3)				224,992
Overhead (5)	(4) + 35%				78,747
GRAND TOTAL	(4) + (5)				303,739

Table 5–11 CONSTRUCTION COST OF CABLE STAYED CONCRETE GIRDER (ℓ = 320 m) ALT. (3)–3

Item		Unit	Quantities	(K.Shs.) Rate	(100 K.Shs.) Amount
Superstructure:–					
Concrete	σck = 350 kg/cm²	M^3	5,550	1,250	6,938
	σck = 210 kg/cm²	M^3	750	1,150	863
Form		M^2	21,060	210	4,423
Reinforcement		t	555	14,000	7,770
P. C. Rod		t	333	48,000	15,984
Cable		t	600	110,000	66,000
Shoe		t	4	45,000	180
Expansion Joint		M	14.5	17,000	247
Erection Equipment		Lump Sum	1		15,400
Timbering		Lump Sum	1		10,350
Pavement	Carriageway	M^2	4,420	110	486
	Pedestrianway	M^2	1,872	110	206
	Concrete	M^3	405	1,200	486
Handrail		M	1,040	2,800	2,912
Guard Rail		M	–	400	–
Lighting		Each	18	32,000	576
SUB-TOTAL (1)					132,821
Substructure:–					
Concrete	σck = 350,300 kg/cm²	M^3	5,500	1,250	6,875
	σck = 240 kg/cm²	M^3	8,000	1,200	9,600
Form		M^2	9,320	210	1,957
Reinforcement		t	1,288	14,000	18,032
Timer & Scaffolding		M^3	13,179	180	2,372
Excavation & Filling		M^3	13,400	220	2,948
Cast-in-Place Pile		m	900	20,000	18,000
SUB-TOTAL (2)					59,784
Temporary Works (3)		Lump Sum			57,000
TOTAL (4)	(1)+(2)+(3)				249,605
Overhead (5)	(4) x 35%				87,362
GRAND TOTAL	(4) + (5)				336,967

Table 5–12 CONSTRUCTION COST OF CABLE STAYED CONCRETE GIRDER (ℓ = 210 m) ALT. (3)

Item		Unit	Quantity	(K.Shs.) Rate	(1000 K.Shs.) Amount
Superstructure:–					
Concrete	$\sigma ck = 350$ kg/cm²	M³	4,600	1,250	5,750
	$\sigma ck = 210$ kg/cm²	M³	–	1,150	–
Form		M²	18,400	210	3,864
Reinforcement		t	470	14,000	6,580
P. C. Rod		t	265	48,000	12,720
Cable		t	260	110,000	28,600
Shoe		t	6	45,000	270
Expansion Joint		M	29	17,000	493
Erection Equipment		Lump Sum	1		15,400
Timbering		Lump Sum	1		3,000
Pavement	Carriageway	M²	3,570	110	393
	Pedestrianway	M²	1,680	110	185
	Concrete	M³	395	1,200	474
Handrail		M	860	2,800	2,408
Guard Rail		M	1,700	400	680
Lighting		Each	15	32,000	480
SUB-TOTAL (1)					81,297
Substructure:–					
Concrete	$\sigma ck = 350,300$ kg/cm²	M³	–	1,250	–
	$\sigma ck = 240$ kg/cm²	M³	7,017	1,200	8,420
Form		M²	6,622	210	1,390
Reinforcement		t	520	14,000	7,280
Timber & Scaffolding		M³	7,795	180	1,402
Excavation & Filling		M³	1,936	220	425
Cast-in-Place Pile		M	1,552	30,000	46,560
SUB-TOTAL (2)					65,477
Temporary Works (3)		Lump Sum			62,600
TOTAL (4)	(1)+(2)+(3)				209,374
Overhead (5)	(4) x 35%				73,281
GRAND TOTAL	(4) + (5)				282,655

5.5 SCREENING OF ALTERNATIVES

5.5.1 Evaluation of Alternatives

(1) Economic Evaluation

All the alternatives hitherto examined seem to have no huge discrepancy among them economically. However, the steel bridges of alternative (1) and (2) are economically inferior to the alternatives of the concrete bridge in terms of total costing including maintenance cost. Further, among the variant of alternative (3), namely alternative (3)–1 to (3)–3, two alternatives (3)–2 and (3)–3 should also be eliminated on the economic reason.

(2) Technical Evaluation

The steel and concrete bridges have their respective technical merits and demerits.

a) For the steel bridges, a major portion of the structure has to be imported from abroad. On the other hand, the concrete bridges can make use of the available local materials, namely cement, aggregates, reinforcement for their construction.

b) The steel bridges require a large sum of maintenance cost for regularly repainting the surfaces of the steel members.

In this respect, the truss bridge which has a large surface area would requires a bigger cost of repainting than the cable-stayed steel bridge. The concrete bridges are less burdensome on maintenance. The steel cables employed in the cable-stayed concrete girder bridge are inserted in the tubes with their inner gaps grouted with mortar and hence are well protected from any corrosion by the elements.

c) Since the steel bridges are prefabricated and brought in from abroad while the concrete bridges need to be constructed on site, the latter would need a longer work term, however, provide more and longer employment for the local construction industry.

Further evaluation of each type of bridge is as follows:

d) The steel truss and concrete girder bridge with cantilever construction are popular, requiring no high technology in design and construction. On the other hand, the cable-stayed concrete girder bridge and the cable-stayed girder bridge composed of many complicated structures, requiring comparatively higher technology in design and construction. Moreover, verification of aerodynamic stability for these bridges is essential.

e) The pier-positioning case (3) and (4) both have their pier foundation located at position with water depth of more than 30 meters. This would therefore requires higher construction technique. Case (2) has its pier foundation at water of 28 meters deep while Case (1) has its pier-foundation located at very shallow water, 16 meters in depth. The latter would require no more than simple construction technique. In fact, it can be constructed using the landing stage.

5.5.2 Selection

The many analysis contained in this feasibility study points to the high feasibility of the cable-stayed concrete girder bridge with a central span of 250 meters in length, i.e. alternative (3)–1. Further reasonings are:

a) Without any specific economic merits, the alternative (4) and (5) with their pier-foundations at water of great depth should be preferably avoided.

   Firstly, the construction workability of foundations at the deep water as in bridge alternative (4) and (5) seems to be inferior to that of foundations at the shallower water. That is, the reliability of workmanship and easiness to overcome some trouble which may be encountered during the construction should be less expected as the depth of water increases.

   During the bore-hole tests along the proposed route, the two logs sunk to investigate the seabed conditions were compelled to be bored at a position with water less than 15 meters in depth because of the limited capability of the contractor. Since alternative (4) and (5) have their piers located very far from these bore-holes, any unexpected inferiority in seabed bearing conditions would pose great construction difficulties and raise its construction cost.

   Secondly, the extra long outstanding piles of foundation at these deep waters might have to be enlarged if the horizontal forces i.e. seismic force, wind force and brake force assumed in the actual future design are greater than these used in this study. This enlargement of piles foundation would definitely add to the construction cost.

b) For a long center span as proposed in the pier-positioning case (1) and (2), the cable-stayed concrete girder bridge is more economical than the concrete girder bridge. In fact, the construction cost for alternative (3)–1 is the minimum as shown in the alternative analysis.

c) This alternative (3)–1 has a wide underclearance, with ample rooms for the passage of ships both recreational and commercial. Moreover, the profile of his alternative is also compatible to the surrounding natural features.

6 . PRELIMINARY ROAD ENGINEERING

6.1 GENERAL

The three alternative routes namely route A, route B and route C have been analysed and thence alternative route B is selected. In this section, a more detailed study of route B's horizontal and vertical alignment, its intersection with the existing road and the pavement design is carried out. Based on the design criteria, the result of the various field surveys that include sounding survey, topographic survey and subsurface investigation and sufficient reconnaissance survey, the preliminary road engineering is carried out to the scales shown on Table 6–1 below:

Table 6–1 SCALE USED IN DESIGN

Items		Scale	Remarks
Road Design	Plan	1 : 500	Survey Map
	Profile	H = 1 : 500 V = 1 : 200	Survey Map
	Cross-Section	1 : 200	Survey Map
Typical Cross-Section		1 : 100	–
Intersection Design		1 : 500	Survey Map

The drawings of the preliminary road engineering are compiled in the Preliminary Engineering Plans.

6.2 ALIGNMENT

The horizontal alignment is planned on the basis of the alternative route B alignment.

The starting point and the ending point of the proposed road are the station No. 0+0.00 and station No. 41+16,770 respectively and the total proposed road length is 4,116 meters.

6.2.1 Horizontal Alignment

The location procedures of the alignment are as follows:

a) The alignment of the proposed route is located using a mosaic map with 1:4000 scale.

b) The alignment examined depends on the results of the conducted sounding survey.

c) The alignment is re-located on the basis of the mosaic map with scale 1:2500.

d) On the project site, the arrangement between the alignment on the map and actual site is carried out by highway engineer and the final alignment is arrived.

e) Plain table survey is carried out along the final alignment.

Main considerations for the route location are as follows and the final alignment is illustrated in Fig. 6–1:

a) Part "A" in Fig. 6–1, this area will be expected to develop into a residential area in the future. Therefore the alignment should if possible avoid passing through it.

b) Part "B" in Fig. 6–1, this area is also expected to develop into a residential area but from the viewpoint of the electric power line and Kilifi creek condition for the bridge, the alignment cannot be avoid passing through it.

c) Part "C" in Fig. 6–1, this area is classified as the existing residential area. The alignment is decided taking into account the conditions at the Kilifi creek such as the depth and width of the creek, preservation of the environment and non-disruption of the community.

d) On the Kilifi side, there exists a development plan prepared by the Ministry of Urban Development and Housing Physical Planning Department. The proposed bridge site for this project corresponds with the bridge site shown in the development plan and for this reason, the alignment is decided on the basis of this development plan.

6.2.2 Vertical Alignment

(1) Basic Consideration

The following main aspects are considered in this project when deriving the vertical alignment.

a) Existing ground level,
b) Navigation clearance, and
c) Depth of the bridge structure (main girder)

(2) Case Study

With the above basic concerns, two cases, namely case "A" and case "B" as shown in Fig. 6–2 are considered.

a) Case "A"

Case "A" proposes a bridge with a crest at the centre and its sides following as closely as possible to the existing ground level.

b) Case "B"

Case "B" proposes a bridge with a fixed gradient all along its length. Therefore, high embankments are required on both sides of the bridge.

The main reasons for adopting Case "A" in this project are as follows:

a) Case "A" is obviously more economical costwise than Case "B" because its construction involves a lower abutment structures and embankment than Case "B".

b) Generally speaking, the centre of a long span bridge appears to be bent and this appearance is often not agreable to the eye. To overcome this problem, a crest is introduced at the centre of the bridge in Case "A".

(3) Longitudinal Grade

The decision on the longitudinal grade to be used is made on the basis of such considerations as the vertical alignment, condition of the existing ground level, the aesthetics of the bridge and economic aspects.

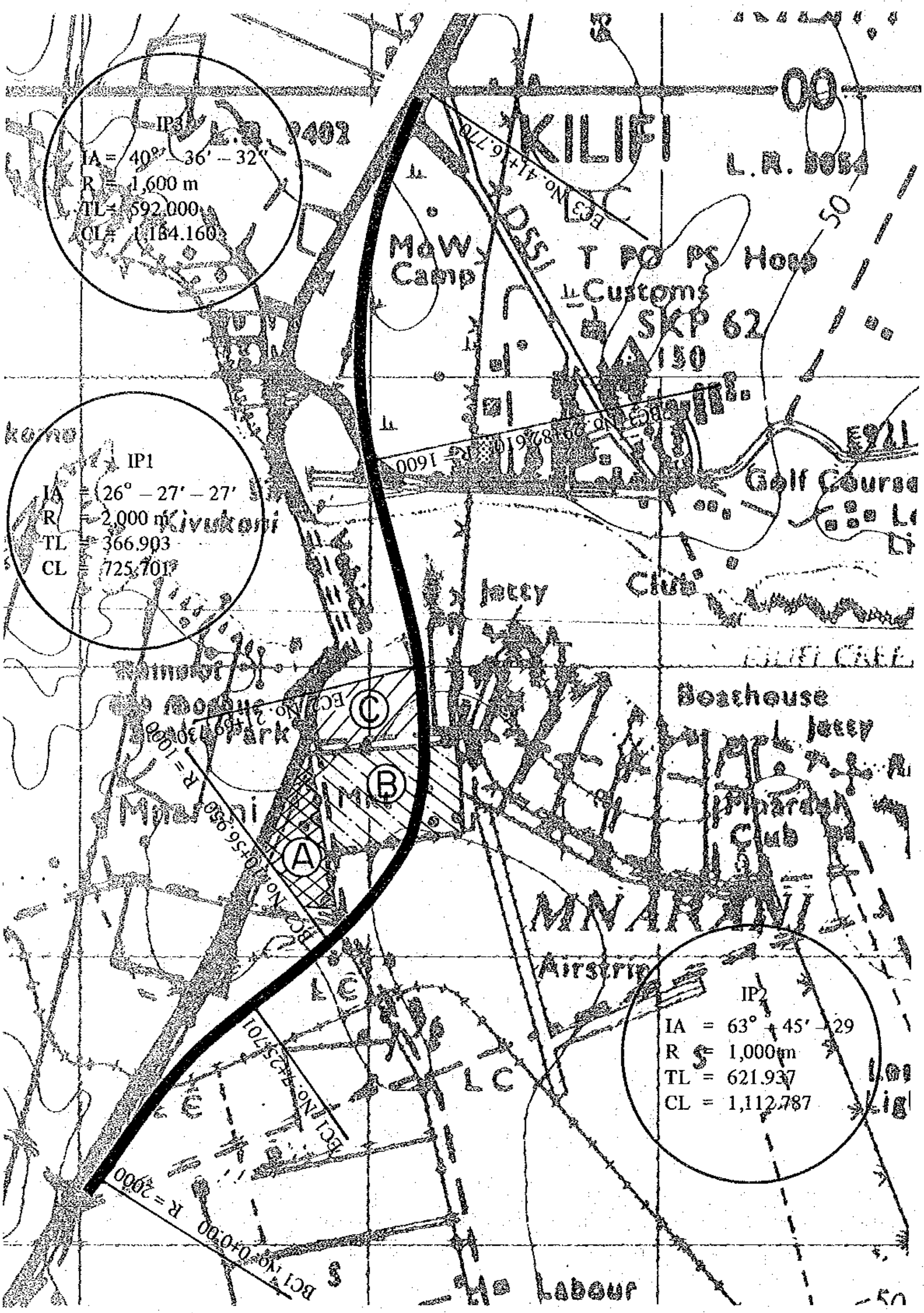


Fig. 6–1 HORIZONTAL ALIGNMENT PLAN

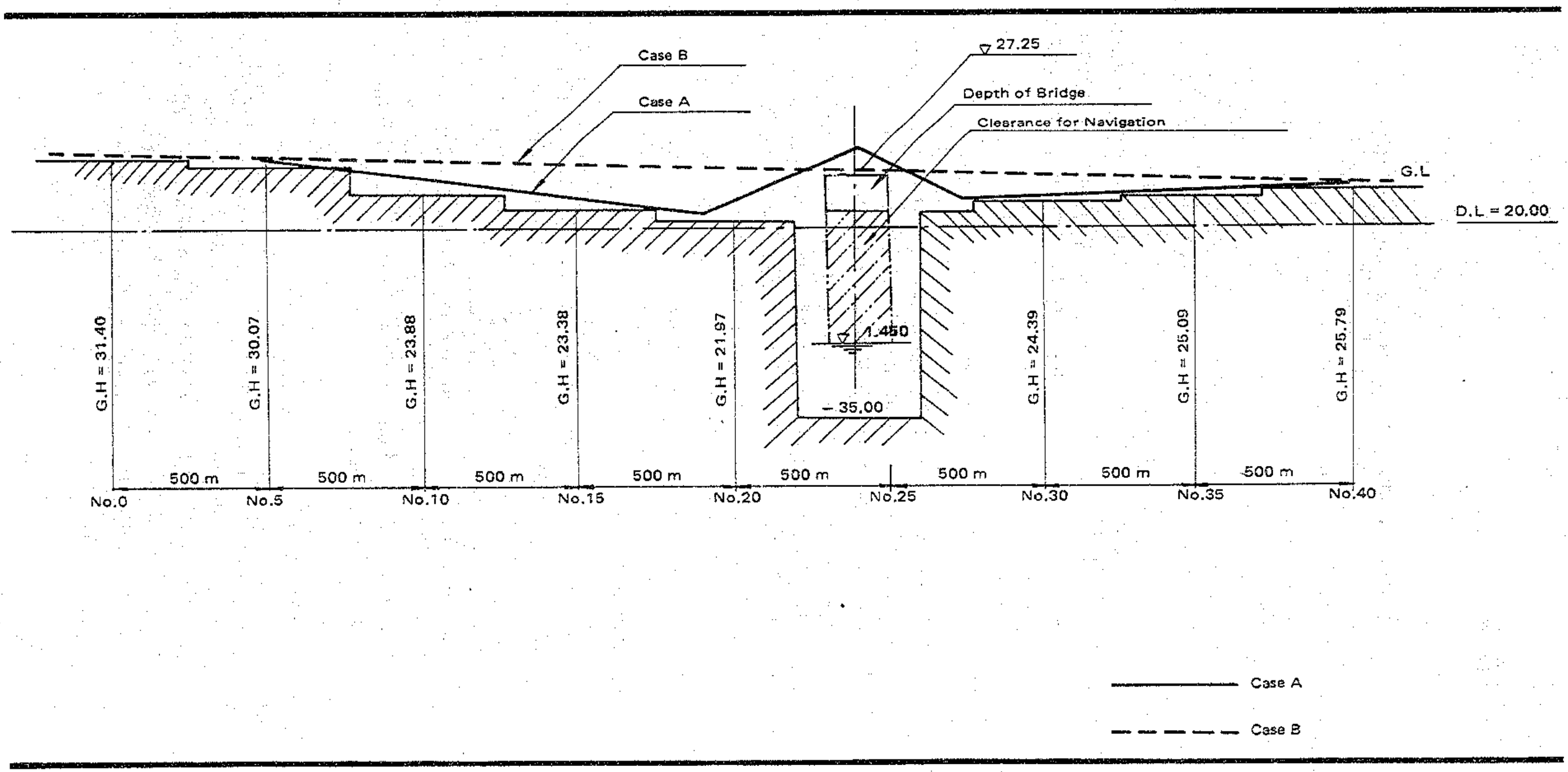


Fig. 6–2 VERTICAL ALIGNMENT PLAN

6.3 CROSS-SECTION

6.3.1 Basic Considerations

(1) Basic Standards Used:

The standard cross-section is determined on the basic of the guidelines as given in the "Road Design Manual – Part I (Geometric Design of Rural Roads)", with some elements modified.

(2) Number of Lanes

It is expected that the future traffic volume on the Kilifi bridge will swell to 7,831 (p.c.u.) V/D in the year 2010. However, the calculated traffic capacity of a two (2) lane road lies between 10,000 (p.c.u.) V/D and 15,000 (p.c.u.) V/D while that of a four (4) lane road lies between 40,000 (p.c.u.) V/D and 50,000 (p.c.u.) V/D. With such understandings as the expected future traffic volume, the future land use and development plans, a two (2) lane road is recommended for this project.

(3) Lane Width

The lane width is determined on the basis of the composition of traffic, the function of the road and the future traffic volume. The results of the traffic counting survey shown that the medium trucks, the heavy trucks and the buses, together accounted for approximately 38.6 per cent of the total traffic. This percentage is very high when compared to vehicle composition of traffic on other roads and as such a wider lane width is recommended. The "Road Design Manual" has established lane widths of 3.00 meters, 3.25 meters and 3.50 meters. The above factors necessitate a lane width of 3.5 meters (Type II) for this project.

(4) Shoulder Type

The function of a shoulder is as described in the "Road Design Manual" prepared by the Ministry of Works. In general, there are two different types of shoulders, hard and soft. Most of the roads in Kenya have soft shoulders. The road inventory survey, indicates that B–8 route in Kilifi has also soft shoulder. The traffic lanes and the shoulders of these roads are at different levels due to the wearing of the latter by heavy rains and other reasons. Such a situation is very dangerous for the road users. For this reason, hard shoulders are recommended.

(5) Shoulder Width

The project road passes through the urban area of Kilifi town. The shoulder width should therefore be determined bearing in mind the parking problems and emergency stops. Taking the width of a passenger car is between 1.7 m and 1.9 m. As the necessary lateral clearance, a 2.50 m shoulder width is adopted.

(6) Pedestrian Way

The maximum daily number of pedestrians crossing from Kilifi to Mnarani and vice versa are 235 persons and 342 persons respectively. The average pedestrians for 12 hours are 134 persons and 202 persons respectively.

The establishment of a pedestrian way be required in the urban areas of Kilifi and Mnarani. Taking into consideration the width of a person, the horizontal clearance for the hanged baggages of the persons and mixed transportation, i.e. persons and bicycles, a 2.0 meter width for the pedestrian way on both sides are established.

(7) Embankment Slope

The slope of the embankment is determined on the basis of the height of the embankment. With reference to the Road Design Manual of the Ministry of Transport and Communications (M.O.T.C.), the embankment slope is decided.

The height of the embankments of the road is less than 3 meters and for this reason, an embankment slope of 1:2.0 is adopted.

(8) Road Reserves (Right of Way Width)

A road reserve width of 60 meters is adopted due to the following reasons:

a) The MOTC's design manual, requires a road reserve width of 40 m or 60 m.

b) At this point, the percentage of land acquisition costs within the project cost is very low, so the desirable road reserve width is manageable.

c) To preserve the good environment along the Kilifi creek.

6.3.2 Typical Cross-section

(1) Road Cross-section

The determination of the road cross-section is based on the consideration of the future landuse and results of the reconnaissance survey. The cross-section so determined is shown in Fig. 6–3. Cross-section type A and type B and adopted to the urban and suburban area respectively.

(2) Bridge Cross-section

According to the standard cross-section type BR–2 given in the "Road Design Manual", the carriageway width is 7.5 meters (*0.25 m + 7.0 m + *0.25 m) (*is the width of the shoulder) but an 8.50 meter wide carriageway bridge due to the following reasons:

a) Percentage of heavy trucks is high

b) A high running speed is expected on the Kilifi bridge.

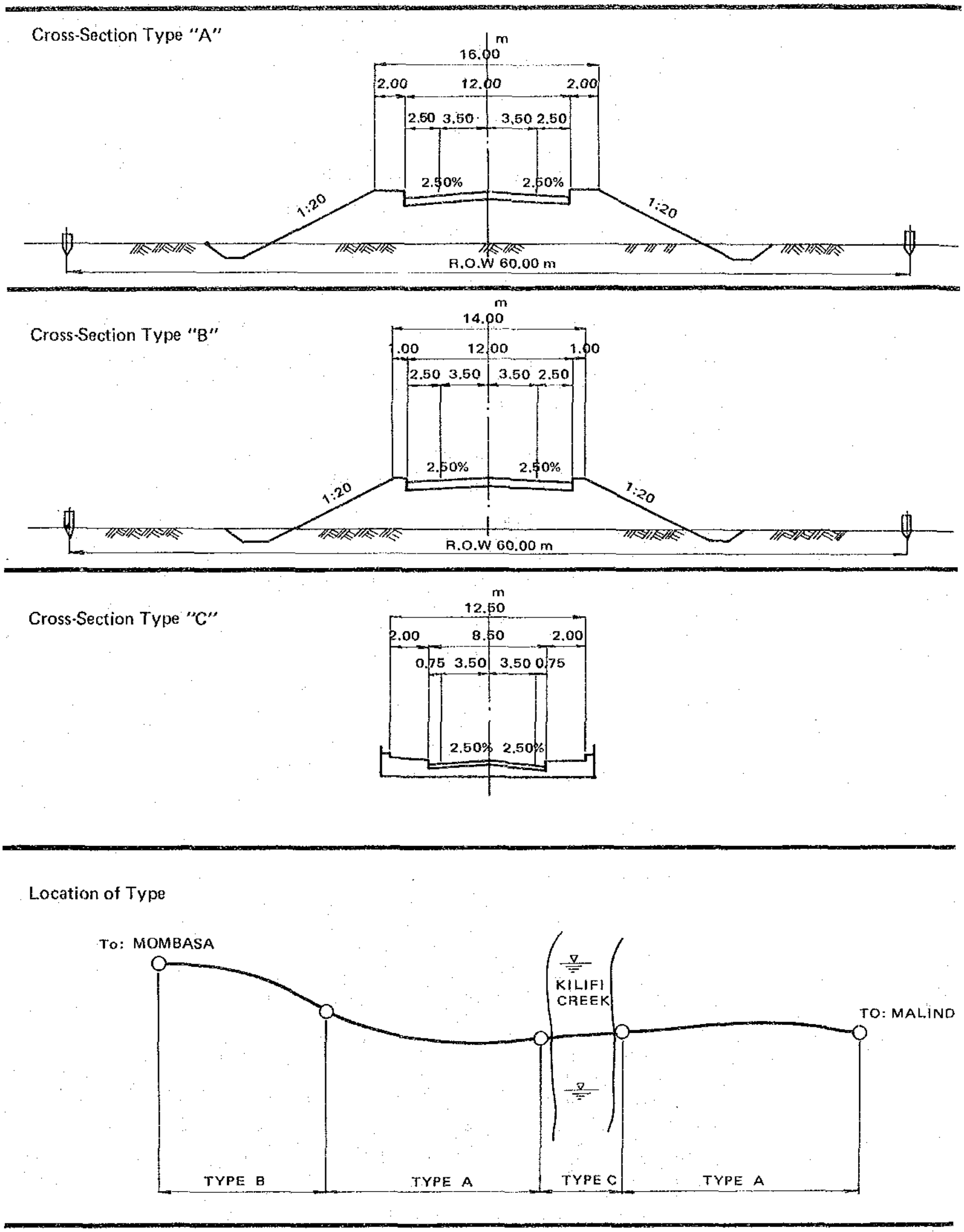


Fig. 6–3 CROSS-SECTION TYPE

6.4 PAVEMENT DESIGN

The pavement materials and pavement design is carried out on the basis of the "Road Design Manual (Materials and Pavement Design for New Road)" which was prepared by M.O.T.C.

6.4.1 Traffic Class

The cumulative number of the standard axles is estimated at 20.3 million on the basis of the projected future traffic volume. According to the classification, the traffic in this study can be classified as class T2 but on account of the project life and maintenance a class T1 has been adopted.

6.4.2 Climate

The mean annual rainfall around the Kilifi area is between 1,100 mm and 1,200 mm and according to the classification the area may be considered wet.

6.4.3 Subgrade

The soil in the study area is classified on the basis of the results of the material survey including soil testing of the embankment mateial. The detailed material survey is described in the soil technical report. According to the result of the material survey, the S3 soil class is adopted for this study.

6.4.4 Pavement Structure

The pavement structure consists of the surfacing base course and sub base course.

(1) Surfacing

There are several types of surfacing but in practice, two types of the asphalt concrete pavements namely type I and type II are the most suitable and cover virtually all the need of this country. Asphalt concrete type I in this study is adopted on the requirement of the road design manual prepared by M.O.T.C.

(2) Base Course

Considering the future traffic volume, the cumulative number of standard axles and the condition of the subgrade, the dense bitumen macadam method is recommended.

(3) Sub-base Course

With the knowledge of the soil conditions of the project site, future traffic volume and the cumulative number of standard axles, a graded crushed stone type of sub-base course is recommended.

(4) Pavement Structure

On the basis of the recommended pavement materials and in accordance with the requirements of the M.O.T.C.'s Road Design Manual, the measurement of each course of the pavement is determined, as shown in Fig. 6–4 with a Standard Pavement Structure Type 12, S–3 and T–1 for the Subbase course, base course and surfacing respectively.

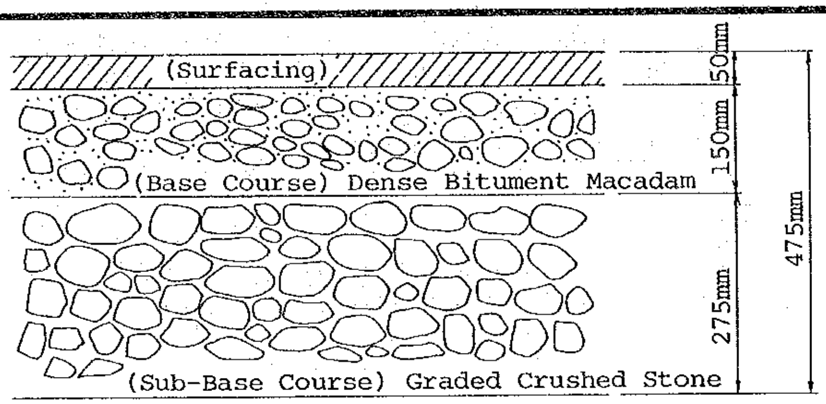


Fig. 6–4 MEASUREMENT OF PAVEMENT STRUCTURE

6.5 INTERSECTION DESIGN

6.5.1 Basic Considerations

The project road is to serve dual functions that is an arterial for both the short distance trips made by the inhabitants within the study area and the long distance trips. The intersection design should therefore considered the traffic flows, traffic accidents and accessibility of the inhabitants. The basic guidelines for the intersection design are as follows.

a) To conform to the existing road network

b) To avoid interupting the traffic flow

c) To decrease the traffic accidents

d) To ensure a good accessibility for the inhabitants

e) To conform to the existing ground condition

6.5.2 Type of Intersection

The type of intersection required depends much on the class of the existing road with which the project road intersects. As shown in Table 6–2, if the project road intersects with a local road, for example, gradeseparated intersection design is not necessary. The reconnaissance survey reveals that the intersected existing roads are but local roads. This finding hence leads to the adoption of at-grade intersection design.

Table 6–2 TYPE OF INTERSECTION

Intersection of Project Road	At-grade intersection		Grade Separation	Remarks
	Non-signalized	Signalized		
to Primary Distributor	X	X	O	
to District Distributor	X	O	X	
to Local and Access Road	O	X	X	

Note: O – to establish intersections

X – not to establish intersections

6.5.3 Intersection Interval

For the determination of intersection interval, factors such as stopping distance, design speed, deceleration distance, reaction time distance need to be carefully weighed. In a case where the design speed is 100 km/h, a reaction time of 10 sec. with a deceleration speed of 0.2 g, an intersection interval of 475 m is generally said to be sufficient. As this project is likened to the above case, intersections are planned at an interval of 500 m.

6.5.4 Description of Intersection

Having considered all the necessary factors, and intersection interval, design for each intersection is determined. The location and the conceptional traffic flow pattern of each intersections are illustrated in Fig. 6–5.

a) "A" and "E" Intersections

The project road is connected to the existing B–8 road at points "A" and "E" by a T-type at-grade intersection.

b) "B" Intersection

Intersection "B" has five (5) legs. In a 5-legs intersection, it is very difficult to control the traffic flow, as the intersection has been originally planned to be a 4-legs intersection. There are two existing roads across the proposed route along the "B" intersection. To prepare the two intersection at the each existing roads may be occured interrupted traffic flow on the proposed road. So, "B" intersection is connected to the one existing road. The connection between the existing road and the other existing road is planned by frontage road that is constructed parallel in the proposed road.

c) "C" Intersection

"C" intersection is planned as a normal 4 legs intersection.

d) "D" Intersection

The difference in level between the proposed road and the existing road is about 3.0 meters. Conveniently this intersection may be planned as a grade separated intersection. However, having concerns for the service of the Kilifi township, at-grade intersection in this area is much preferred.

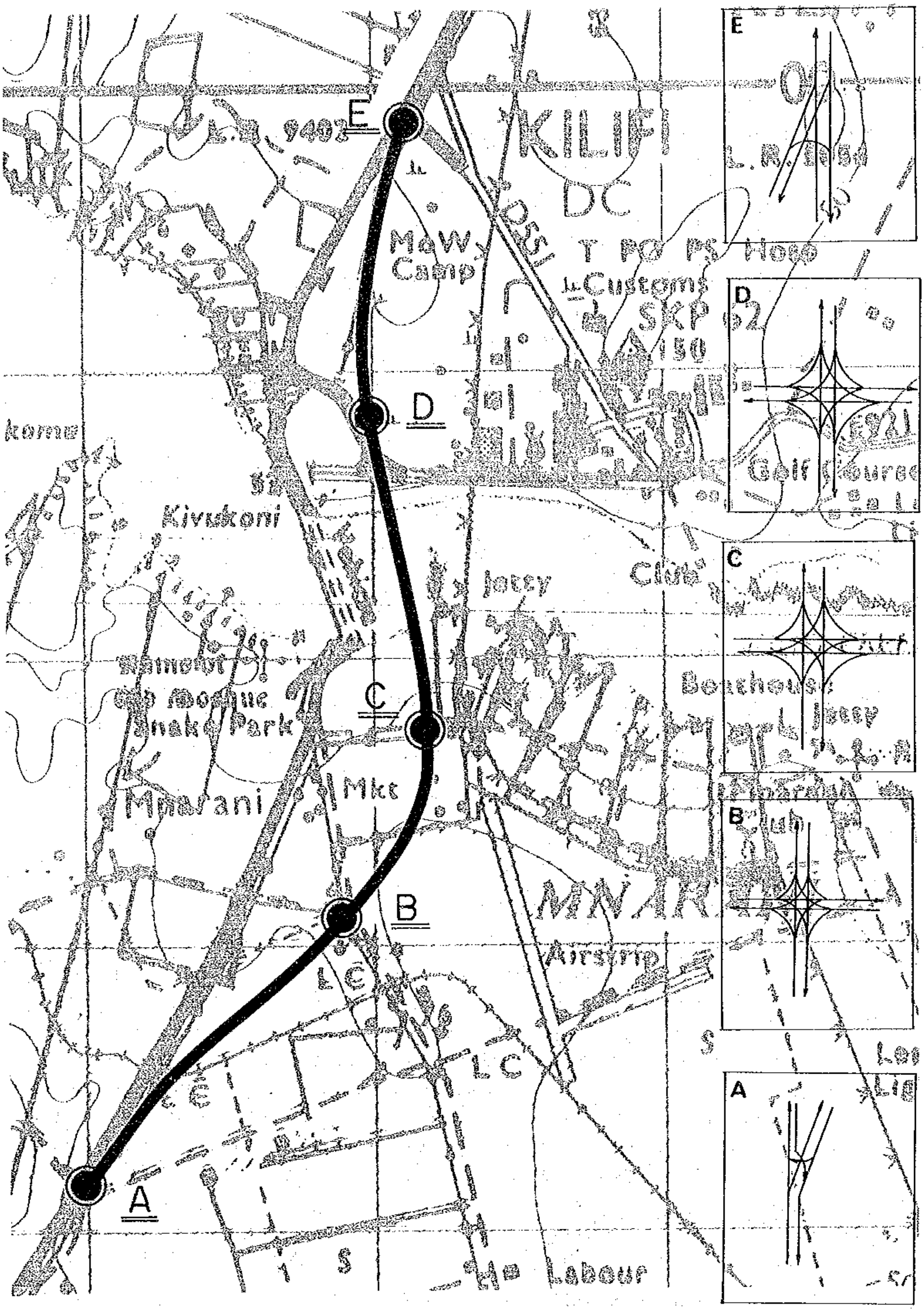


Fig. 6–5 LOCATION OF INTERSECTION

Therefore, the intersection is shifted about 100 meters to the north from the originally planned location.

6.6 RESTING AREAS

The study area is endowed with a good environment with flush greenery, and a great view of the sea. Some resting areas are deemed appropriate and would be an invaluable asset.

The resting areas are planned at the Kilifi side on both road side considering the environmental aspects. The general drawing of the resting areas are illustrated in the Preliminary Engineering Plans.

7 . PRELIMINARY BRIDGE DESIGN

7.1 INTRODUCTION

In this chapter, a more detailed study is carried out on the design of a cable stayed bridge with the central span of 250 m and a total length of 420 m which is proposed as the result of the alternative bridge study. In accordance with the latter study, the foundation for the abutments at both end of the bridge is of the direct spread foundation design, but for the main towers the reinforced concrete pile foundation is employed. These foundations piles are made of cast-in-place concrete that fills the holes bored by the reverse circulation method.

7.2 POSITIONING OF PIERS AND ABUTMENTS

7.2.1 Positioning of Piers

Considering the results of the topographic survey and the subsurface investigation, some adjustments on the positioning of the piers and abutments are made. The following factors are considered;

a) The piers shall be so located that the depths of water at both piers will take nearly equal values.

b) The piers shall be so located as to avoid the steep slope of the seabed.

c) In the seabed beneath the surface layer of a loose silty sand layer there exists a dense silty sand layer which is expected to bear foundations. There are no specific geological faults at which the positioning of foundations should avoid.

The position of piers in this study is as follows.

Mnarani side	Station No. 23 + 24 m
Kilifi side	Station No. 25 + 74 m

The pier on Mnarani side is located on a generous slope of about 10 degree, however, the pier on Kilifi side is located inevitably on a comparatively steep slope of about 35 degree. The profile and subsurface condition of the seabed around the piers are shown in Fig. 7–1.

7.2.2 Positioning of Abutments

The position of abutments is determined following the below considerations,

a) The abutments shall be so located that the height of the abutments would take less than 12 m and the height of the embankments would take less than 10 m.

b) The abutments shall be so located as to avoid the slope at the water edge which is covered with a weathered layer.

c) The abutments shall be so located as to make the superstructure symmetric.

The position of abutments in this study is as follows.

Mnarani side	Station No. 22 + 39 m
Kilifi side	Station No. 26 + 59 m

The profile and the subsurface condition of seabed around the abutments are shown in Fig. 7–2.

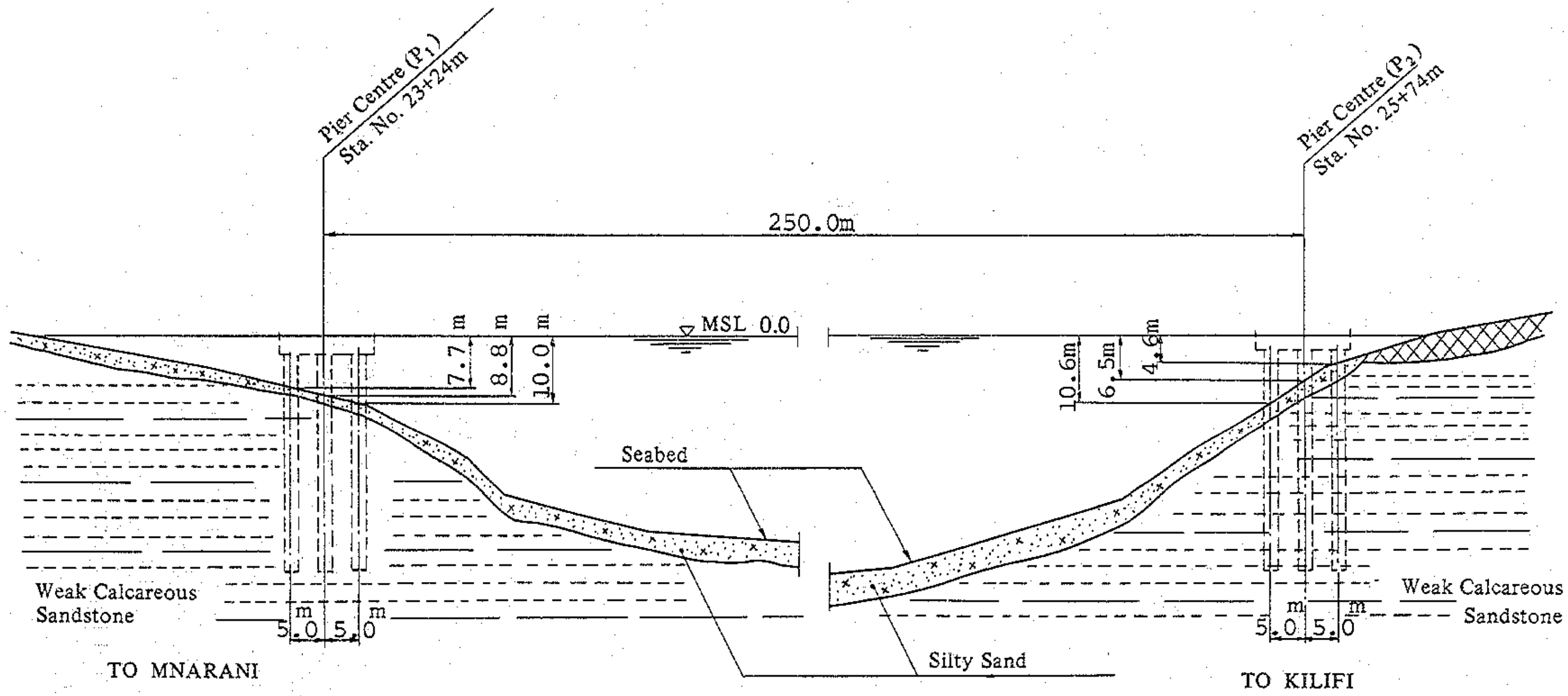


Fig. 7–1 PROFILE AND SUBSURFACE CONDITION OF SEABED AROUND PIERS

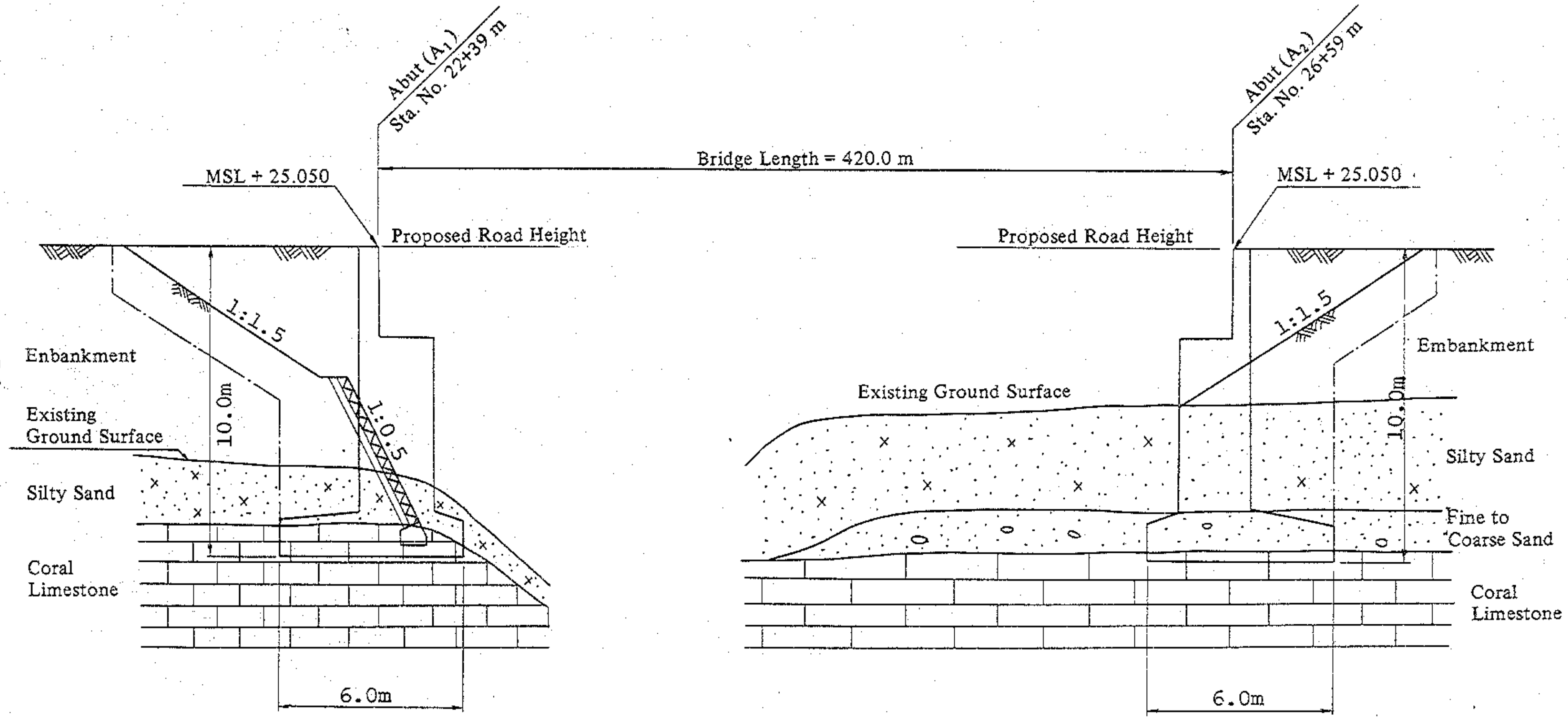


Fig. 7–2 PROFILE AND SUBSURFACE CONDITION AROUND ABUTMENTS

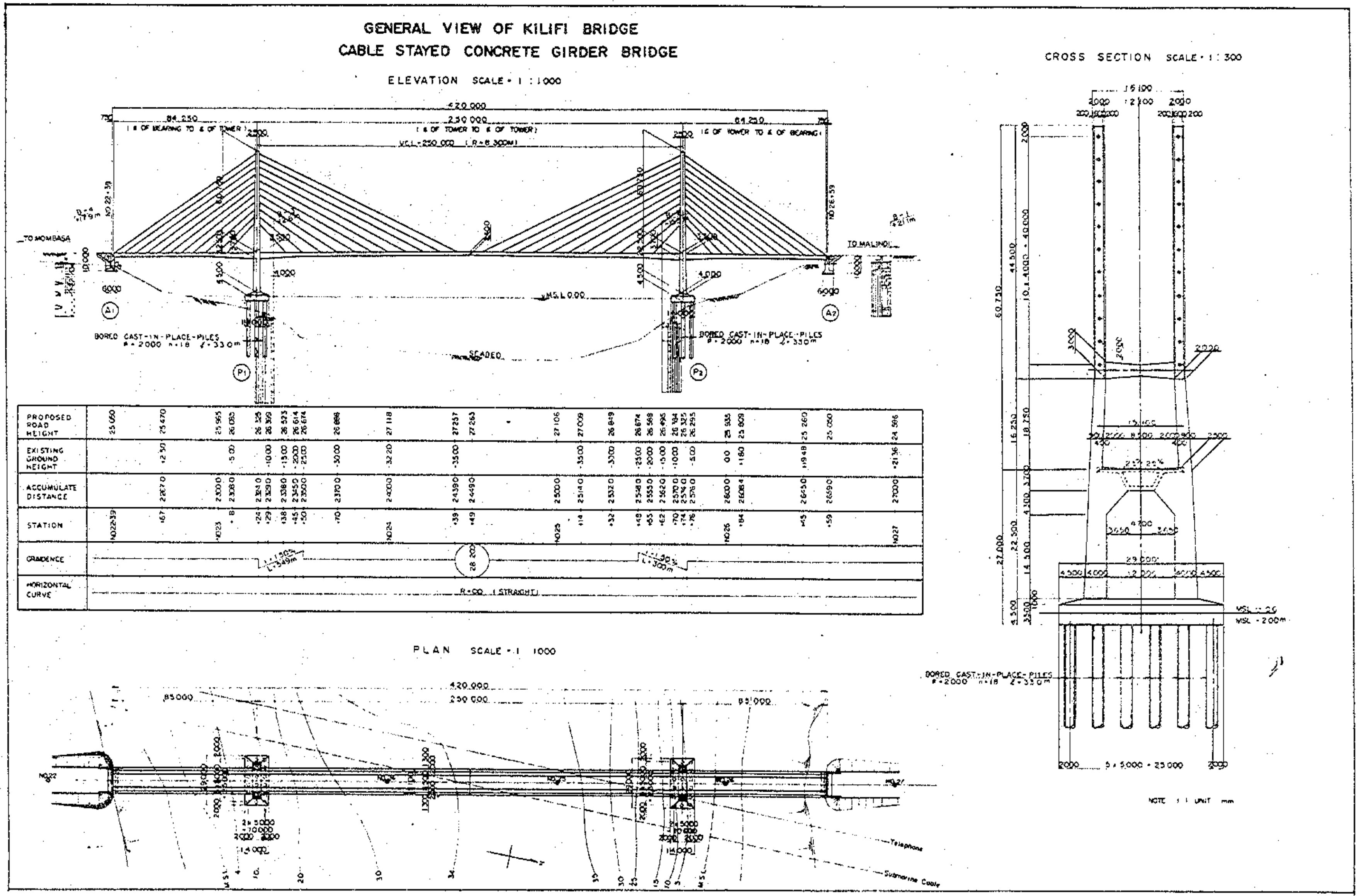


Fig. 7–3 GENERAL VIEW OF KILIFI BRIDGE
(PRELIMINARY BRIDGE DESIGN)

7.3 SUPERSTRUCTURE DESIGN

7.3.1 Design Criteria

The study of the superstructure which will be carried out hereafter is in accordance with the following design criteria.

(1) Dimension

a) Design of bridge : Cable stayed prestressed concrete girder

b) Length of bridge : 420 m
(St. No. 22+39 m ∿ St. No. 26+59 m)

c) Length of spans : 85 m + 250 m + 85 m

d) Width of bridge (Refer to Fig. 7–4)

Total width : 12.5 m
Width of carriageway : 8.5 m
Width of sidewalk : 2 x 2.0 m

e) Horizontal road alignment : Straight

f) Angle of bridge : Rectangular (90°)

g) Head clearance

Carriageway : 5.25 m above the top of the road surface

Sidewalk : 2.50 m above the top of the road surface

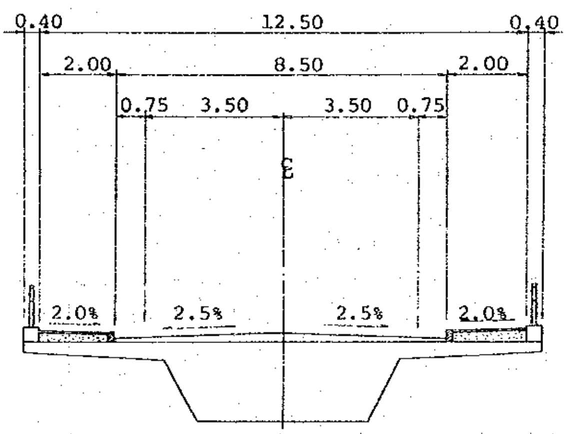


Fig. 7–4 WIDTH OF BRIDGE (Unit: m)

(2) Design Loads

a) Dead load

The unit weights as given in Table 7–1 are used in computing the dead load. However, any actual weight shall be used if it is known.

Table 7–1 UNIT WEIGHTS OF MATERIALS

Material	Unit weight
Steel, Cast steel and Forged steel	7850 kg/m^3
Cast iron	7250
Alminum alloys	2800
Reinforced concrete and Prestressed concrete	2500
Plain concrete	2350
Cement mortar	2150
Timber	800
Crushed Stone (Under Sidewalk)	1900
Asphalt pavement	2300

b) Live load

The live load shall consist of the moving load of trucks (the T-loadings and the L-loadings), and the sidewalk loading.

The design loading for the main bridge and other structures shall be in accordance with the "Specifications for Highway Bridges" by the Japan Road Association to carry TL-20 Loading. A simple comparison of this TL-20 Loading with HA and HB Loading in B.S. is carried in Appendix.

Live load for slabs and floor systems

Slabs and floor systems shall be designed for the following live loads:

i) On the carriageway, the T-loadings shown in Table 7–2 and Fig. 7–5 shall be placed. In the longitudinal direction of a bridge, generally only one T-loading shall be placed, and in the transverse direction, arbitary number of T-loadings shall be placed so as to produce the maximum stress in the member to be designed.

Table 7–2 T-LOADINGS

Class of bridge	Loading	Gross weight W (ton)	Weight of a front wheel 0.1W (kg)	Weight of a rear wheel 0.4W (kg)	Weight of a front wheel b_1 (cm)	Weight of a rear wheel b_2 (cm)	Length of contact area of a wheel on the roadsurface a (cm)
1st	T–20	20	2,000	8,000	12.5	50	20

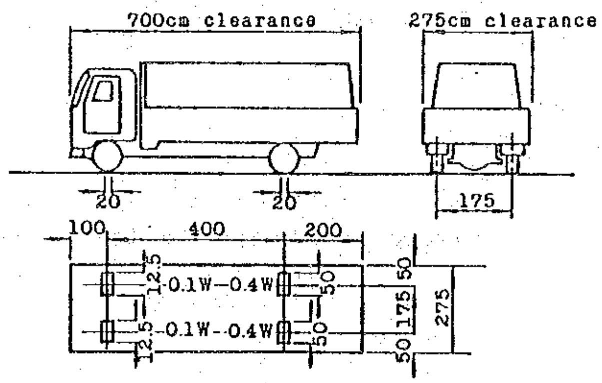


Fig. 7–5 T-LOADINGS

ii) On sidewalks, an uniform live load of 500 kilograms per square meter of sidewalk area shall be applied.

Live load for main girders

The main girders shall be designed for the following live loads:

i) On the carriageway, the L-loadings, consist of the line load, P, and the uniform load, p, defined as "main loads" in Table 7–3, shall be placed on the area up to 5.5 meters in width of the roadway, while P/2 and p/2, defined as "sub-loads" in Table 7–3, shall be placed on the remaining area of the roadway, as illustrated in Fig. 7–6, so as to produce the maximum stress in the member to be designed.

Table 7–3 L-LOADINGS

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	5000	350	430 – L but not less than 300	50% of main loads

where, L = span length in meters

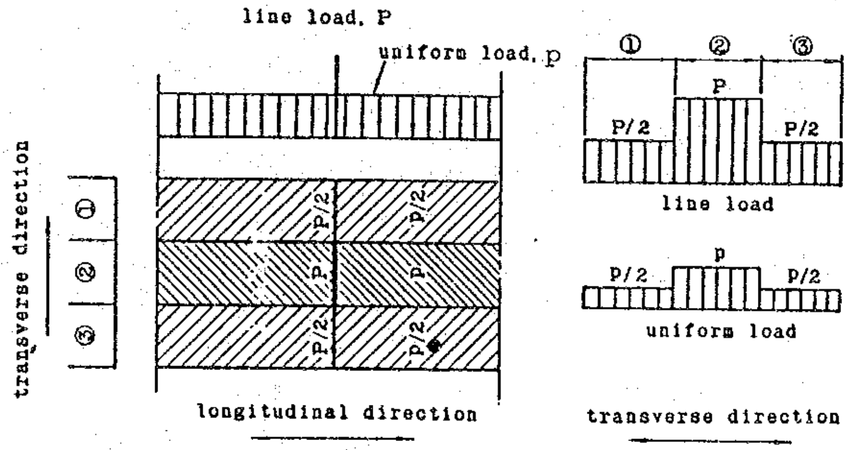


Fig. 7–6 L-LOADINGS

ii) On sidewalks, a uniform live load given in Table 7–4 shall be applied.

Table 7–4 UNIFORM LOAD FOR SIDEWALKS

Span length, L (m)	L≦ 80	80 < L ≦ 130	L > 130
Uniform loads (kg/m²)	350	430 – L	300

c) Impact

Live load stresses shall be increased for impact effects, but impact shall not be applied to the sidewalk loadings.

The impact fraction for the prestressed concrete bridge shall be determined by the following formulae.

For T-Loading, $I = \frac{20}{50 + L}$

For L-Loading, $I = \frac{10}{25 + L}$

where, I = Impact fraction
L = Loaded length for the member under consideration

d) Longitudinal load

A longitudinal load of 70 t resulting from traction or braking of vehicles shall be applied at the road surface and parallel to the derection of the carriageway.

e) Wind load

The wind load shall be a horizontally moving load which impacts most severely on the structure. The design wind gust speed which causes the wind load shall be 30 m/sec. The ground is described in Appendix 1.

f) Effect of temperature

The range of the effective bridge temperature shall be ±5°C and the effective temperature difference within the superstructures shall be 5°C. The co-efficient of linear expansion of concrete is taken to be 10×10^{-6} and that of stayed cables, 12×10^{-6}.

Note 1) The values of the range of effective bridge temperature and the temperature difference are determined in accordance with the specifications for Highway Bridge by Japan Road Association using the data recorded at three stations in Malindi, Kilifi and Mombasa. More detail is described in Appendix 1.

g) Effect of earthquake

Little earthquake is expected in this region, and the effect of earthquake on this bridge deisgn is small. Therefore, in this study, the verification on the effect of earthquake is omitted.

(3) Material and Allowable Stress

a) Concrete (For the main girder and tower)

Nominal design strength	$\sigma ck = 350$ kg/cm²
Allowable compressive strength	$\sigma ca = 125$ kg/cm²
Modulus of elasticity	$Ec = 3.25 \times 10^5$ kg/cm²

b) Reinforcement

Table 7–5 REINFORCEMENT

Standard	Yield Point	Modulus of Elasticity	Allowable Tensile Stress	Similar Standard
JIS G3112 SD30	> 30 kg/mm²	2.1 x 10⁶ kg/cm²	1,800 kg/cm²	BS 4461 cold worked high yield

c) Prestressing bar

Table 7–6 PRESTRESSING BAR

Standard	Yield Point	Tensile Strength	Modulus of Elasticity	Allowable Tensile Stress*	Similar Standard
JIS G3109 SBPD 95/110	> 95 kg/mm²	>110 kg/mm²	2.0 x 10⁶ kg/cm²	66.0 kg/mm²	BS 4486 Prestressing Bars
JIS G3109 SBPR 95/120	>95 kg/mm²	>120 kg/mm²	2.0 x 10⁶ kg/cm²	71.2 kg/mm²	BS 4486 Prestressing Bars

* for permanent design loads

d) Cable

Table 7–7 CABLE

Standard	Sign	Nominal size of Strand	Tensile Strength	Bearing Stress *
JIS G3536	SWPR 7A	15.2 mm Composed 7-wires	>165 kg/mm²	>140 kg/mm²

* The stress for 0.2% permanent elongation

7.3.2 Skeleton

(1) Cable Arrangement

a) Multi-cable design

A multi-cable design for the cable stayed concrete girder is selected. The multi-cable design has many cables which are stretched between the tower and main girder and are arranged with relatively small intermediate supporting points. This design is recommendable for such a bridge that has a relatively long span and must be constructed by cantilevering method.

The reasons are as follows.

Firstly, after the completion of bridge construction, because the main girder is sustained by many cables, the bending moment that acts on the main girder is very small.

On the other hand, during the construction stage, the main girder is cantilevered and sustained by cables stretched from the tower, and the length of cantilevered girder outside the supported point is very short. Thus the bending moment of the main girder during the construction is smaller than that after the completion.

Because of the small size of bending moment, the main girder can be very shallow in depth and accordingly can have very small dead weight.

Secondly, because the main girder is sustained by many cables with different lengths and oscillation frequencies, the oscillation characteristic and the aerodynamic stability of the structure are remarkably improved.

b) Harp-shaped pattern

Usually, there are two manners to connect the cables to the tower, one is to connect the cables concentrating them at the top of the tower, and the other is to connect them along the upper portion of the tower at suitable intervals.

The former is usually called the radiating pattern and the latter is called the harp-shaped pattern.

In this study, the harp-shaped pattern is selected on the basis of the following reasons.

Firstly, in the case of the radiating pattern, the concentrated cables at the top of the tower make the detail of the anchoraged complicated.

Secondly, in the case of the radiating pattern, all the cables must be connected after the completion of the tower. In the case of the harp-shaped pattern, because the tower can be constructed simultaneously with main girder, the efficiency of the work is superior to that of the radiating pattern.

c) Intervals of cables

Usually, the intervals of points which the cables are connected on the girder are so determined that the length between the points of connection has equal or doubling length of the block constructed in each shift of the cantilever carriage. In this study, the length of a block constructed at each shift of the cantilever carriage is assumed to be 5.0 m, and the connecting points are placed at intervals of 10 m. The connecting points of cables to the top of the tower is placed at intervals of 4.0 m. This is determined to be as short as possible keeping the space for anchorages. The number of cables stretched toward the central span and the flanking span is eleven each.

d) Cable planes

Generally, there are two kinds of cable plane arrangement in the cable stayed girder bridge, single plane and double planes.

In this study, the single plane arrangement is eleminated on the basis of the following reasons.

Firstly, the cable stayed girder bridge having the cables at a single plane is inferior to that of the double planes in terms of aerodynamic stability.

Secondly, in the cable stayed girder bridge having the cables at a single plane, the plane of cables is located at the center of the carriageway and this divides the carriageway into two.

This would spoil the profitable employment of the road space in such a relatively narrow bridge as this design.

In the cable stayed girder bridge having double planes, there are two ways of positioning the cable planes. (Refer to Fig. 7–7) One is to have the cable planes located at the far edges of the sidewalk and the other is to locate them between the sidewalk and the carriageway. The former is adopted in this study because the latter has the following demerits.

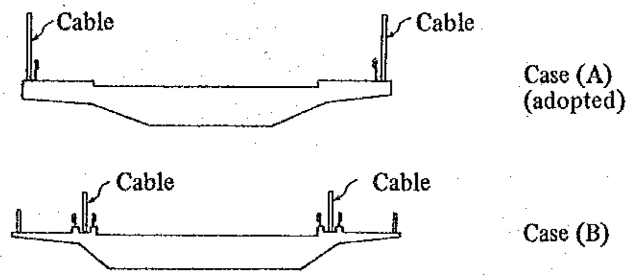


Fig. 7–7 POSITIONING OF CABLE PLANES

Firstly, the narrow room between cables would put a visual pressure upon the dirvers.

Secondly, to protect the cables from any car collision and walkers' mischief, a guardrail and a handrail would be necessary, and accordingly a wider bridge floor is required. This increases the bridge cost.

(2) Tower Height

Generally, a cable works more efficiently if the angle between the cable and the main girder is placed at a wider angle. However, the cables placed at wider angles are always not cost-savers, because of the longer cables and higher towers.

Some literature concerning this matter show that from the economic view point the ratio the height of the tower to the length of the central span should be taken as 1:4. In other words, the top cable of forestay should be placed nearly at an angle of 25° to the main girder.

In this study, the height of tower above the road surface is determined to be 60.75 m. The ratio of the height of the tower to the central span is 1:4.1 and the angle of top cable in the central span is placed at 27°.

(3) Mechanical Configuration

Fig. 7–8' shows the various mechanical configurations which are usually employed.

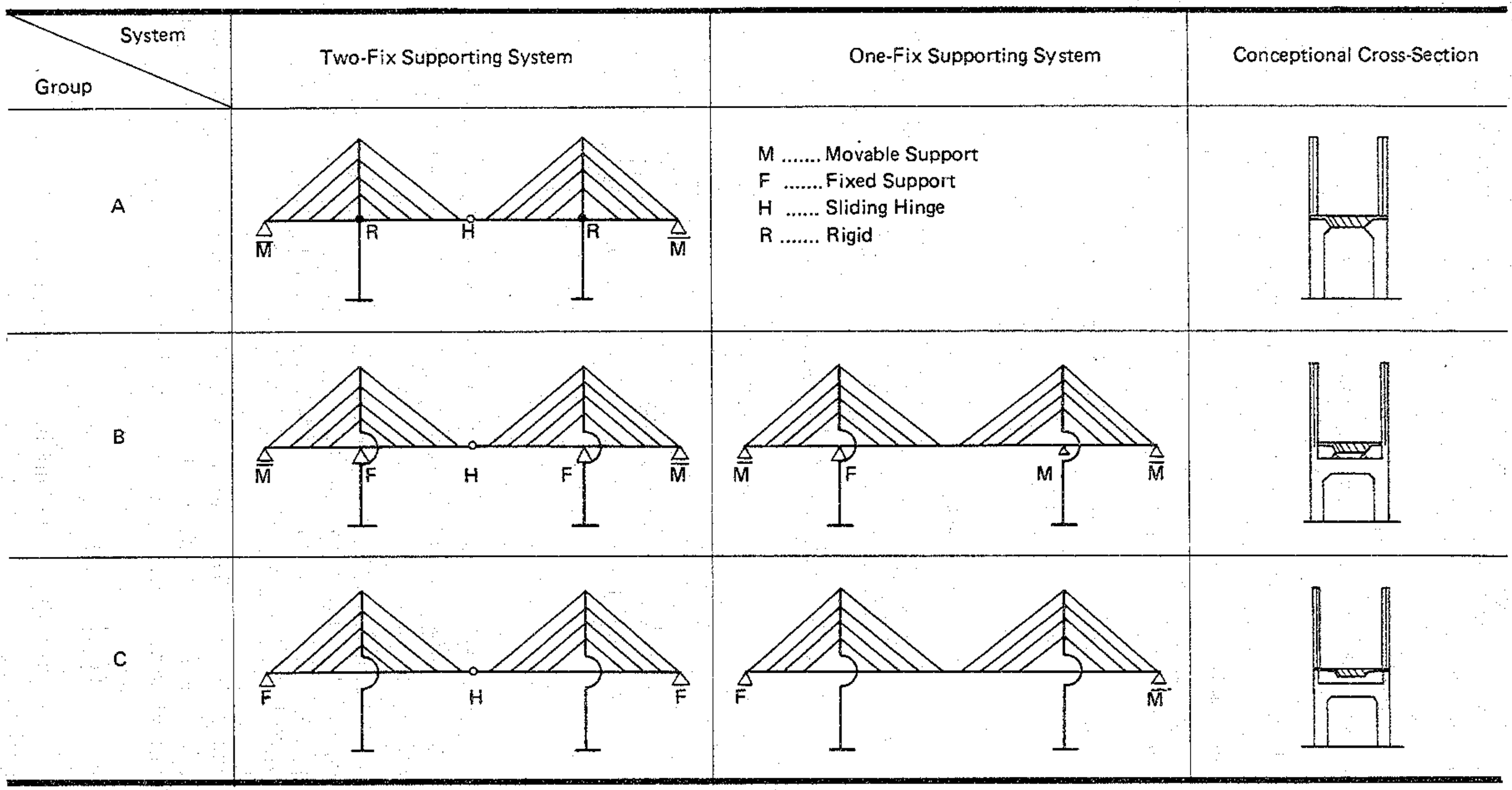


Fig. 7–8 MECHANICAL CONFIGURATION

These are classified into three groups, i.e. A, B and C from the view point of the connection between the tower and the main girder. Further, each group except group A is devided into two from the view point of the continuity of the main girder.

In group A, the tower which stands up on the foundation and the main girder are monolithically built and rigidly connected. In group B and C, the tower which stands up on the foundation is separated from the main girder.

In this study, the mechanical configuration of group A is adopted on the basis of the following reasons.

Firstly, this mechanical configuration needs no bearing shoes which must sustain the large quantity of reaction and accordingly saves the bridge construction cost.

Secondly, during the early stage of construction of the main girder, it is cantilevered out toward the central span and the flanking span simultaneously, balancing each other. In this stage of construction, the fact that the main girder is being connected rigidly with the tower makes the construction easy, safe and economical.

As the main girder is connected rigidly with the tower in this design, a hinge is required at the centre of the central span to release the expansion and shrinkage caused by the temperature change.

This hinge brakes the continuity of the main girder, reduces the comfortability of a running car and slightly decreases the rigidity of the main girder.

On the other hand, in this design, the scale of movement of bearings on abutments caused by the running cars and temperature change is very little. This makes the device against the uplift reaction at abutments simple.

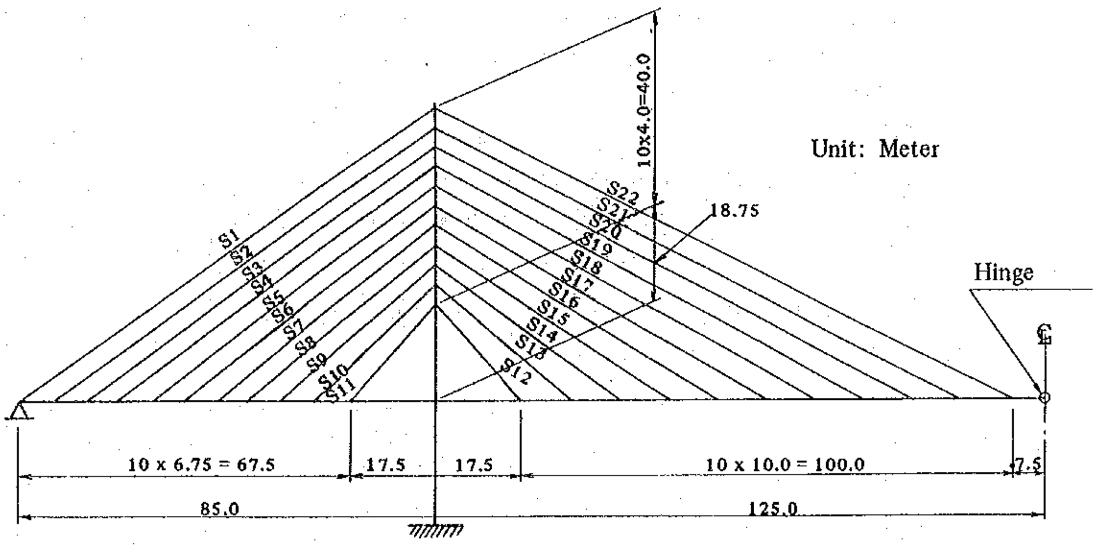
7.3.3 Design of Cables

(1) Kind of Cable

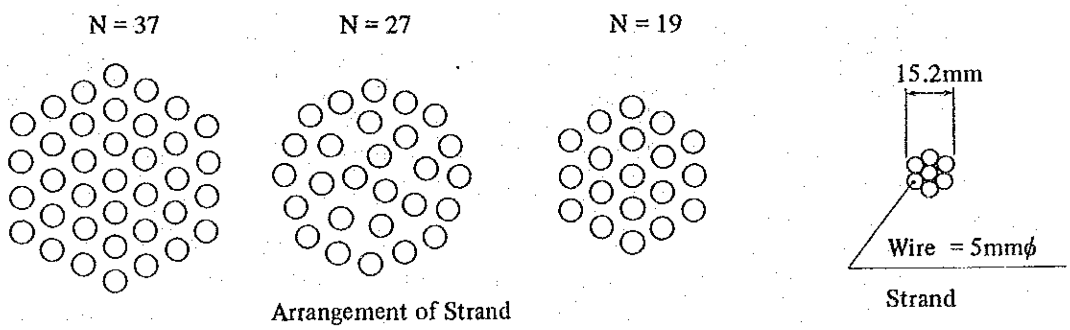
Hitherto, many kinds of cable, i.e. locked coil rope, spiral wire and parallel wire have been used for cable stayed bridges with various kinds of protection against corrosion.

Each kind of cable has its merits and demerits evaluated from its strength, durability and construction workability.

In this design, spiral wire strand is adopted considering that the kind of cable used must be a popular one and it must be protected completely against corrosion. Each strand has a diameter of 15.2 mm and is composed of seven wires each with a diameter of 5 mm. Each cable is in turn composed of 19 to 37 strands.



Cable Number	Design Tensile Strength	Number of Strand	Allowable Tensile Strength	Cable Number	Design Tensile Strength	Number of Strand	Allowable Tensile Strength
S 1	775 t	N=37	787t	S 12	125 t	N=19	404 t
S 2	763	37	787	S 13	121	19	404
S 3	750	37	787	S 14	525	27	574
S 4	636	37	787	S 15	627	37	787
S 5	622	37	787	S 16	635	37	787
S 6	605	37	787	S 17	601	37	787
S 7	592	37	787	S 18	572	37	787
S 8	590	37	787	S 19	596	37	787
S 9	574	27	574	S 20	724	37	787
S 10	506	27	574	S 21	749	37	787
S 11	366	19	404	S 22	670	37	787

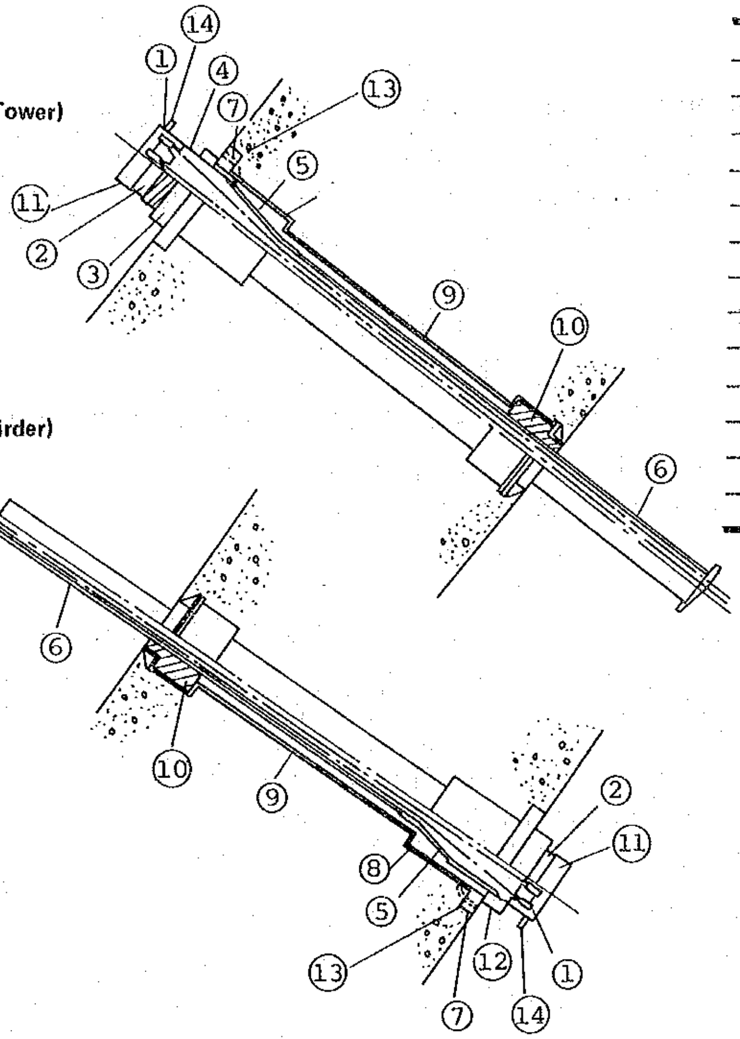


Arrangement of Strand

Fig. 7–9 DESIGN TENSILE STRENGTH AND CROSS-SECTION OF STAYED CABLE

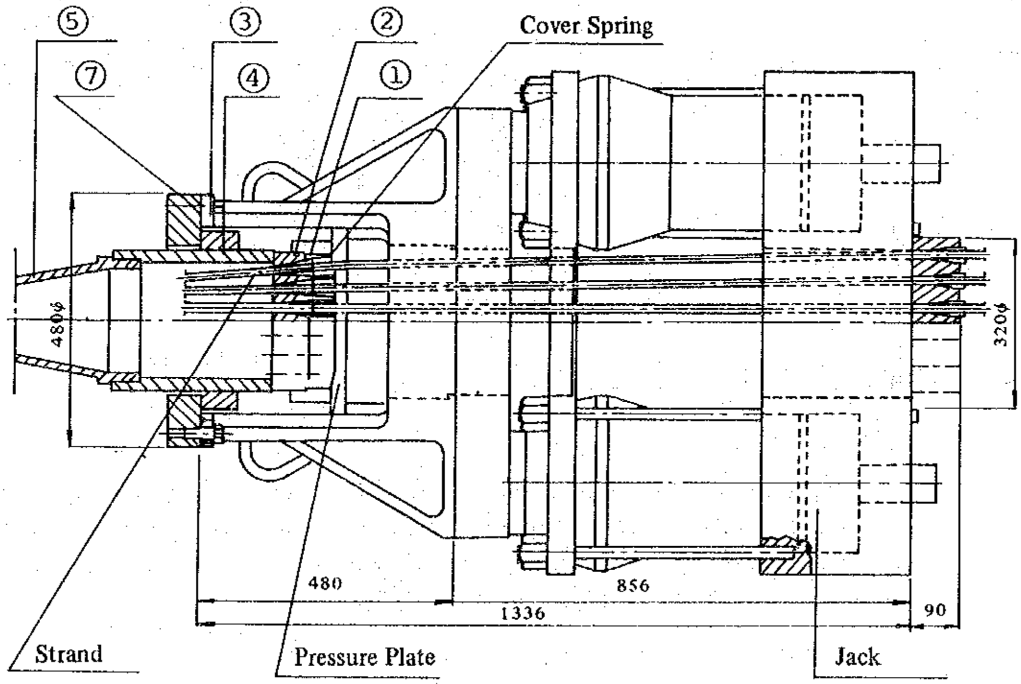
Anchorage (Tower)

Anchorage (Girder)



①	Wedge
②	Block
③	Ring Nut
④	Adjusting Tube
⑤	Trumpet
⑥	Polyethylene Tube
⑦	Bearing Plate
⑧	Tube
⑨	Tube
⑩	Damper
⑪	Cap
⑫	Anchor Ring
⑬	Grouting Hole
⑭	Grouting Hole

Jacking



(Unit: mm)

Fig. 7–10 DETAIL OF ANCHORAGE

(2) Propertioning of Section

The sections of cables, i.e. the number of strands for each cable, and the respective design strength and working force are shown in Fig. 7–9.

(3) Anchorage

For the anchorage of cables the Freyssinet anchor which is one of the more popular anchorages is applied. (Refer to Fig. 7–10)

Each cable, composed of many strands is anchored with a steel socket.

The tower sockets of forestay cables and backstay cables at the same level would take a pitch of 60 cm horizontally.

(4) Protection Against Corrosion

Each cable is inserted in a casing tube. The gap between the cable and the tube is grouted with mortar. Usually the casing tube is made of steel or alminum or polyethylene. This bridge is planned employing the polyethylene tube, because it is easy to handle and it has a high resistant characteristic against the corrosion itself. The thickness of the tubes is 5.5 mm. The diameters of tubes are varied as the diameters of cables inserted vary. (N=19 → ϕ 114mm, N=27 → ϕ 140mm, N = 37 → ϕ 165mm)

Fig. 7–11 shows the typical cross-section of the cable.

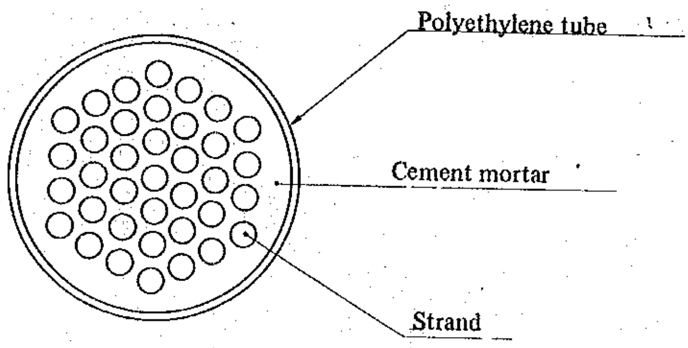


Fig. 7–11 CROSS-SECTION OF CABLE

7.3.4 Design of Main Girder

(1) Configuration

The cross-sectional configuration of the main girder is shown in Fig. 7–12. As shown in the figure, the main girder has a mono-box configuration and the web of the box form an angle of 27° with the vertical (pitch 1:2) is due consideration of the aerodynamic stability.

The depth of the maingirder is 2.40m except a protion near the tower where the maingirder has a greater depth of 3.70 m to sustain the greater bending moment.

The top deck of the box which must carry the weight of the running cars directly has a thickness of 35 cm and is prestressed with P.C. rods transversly.

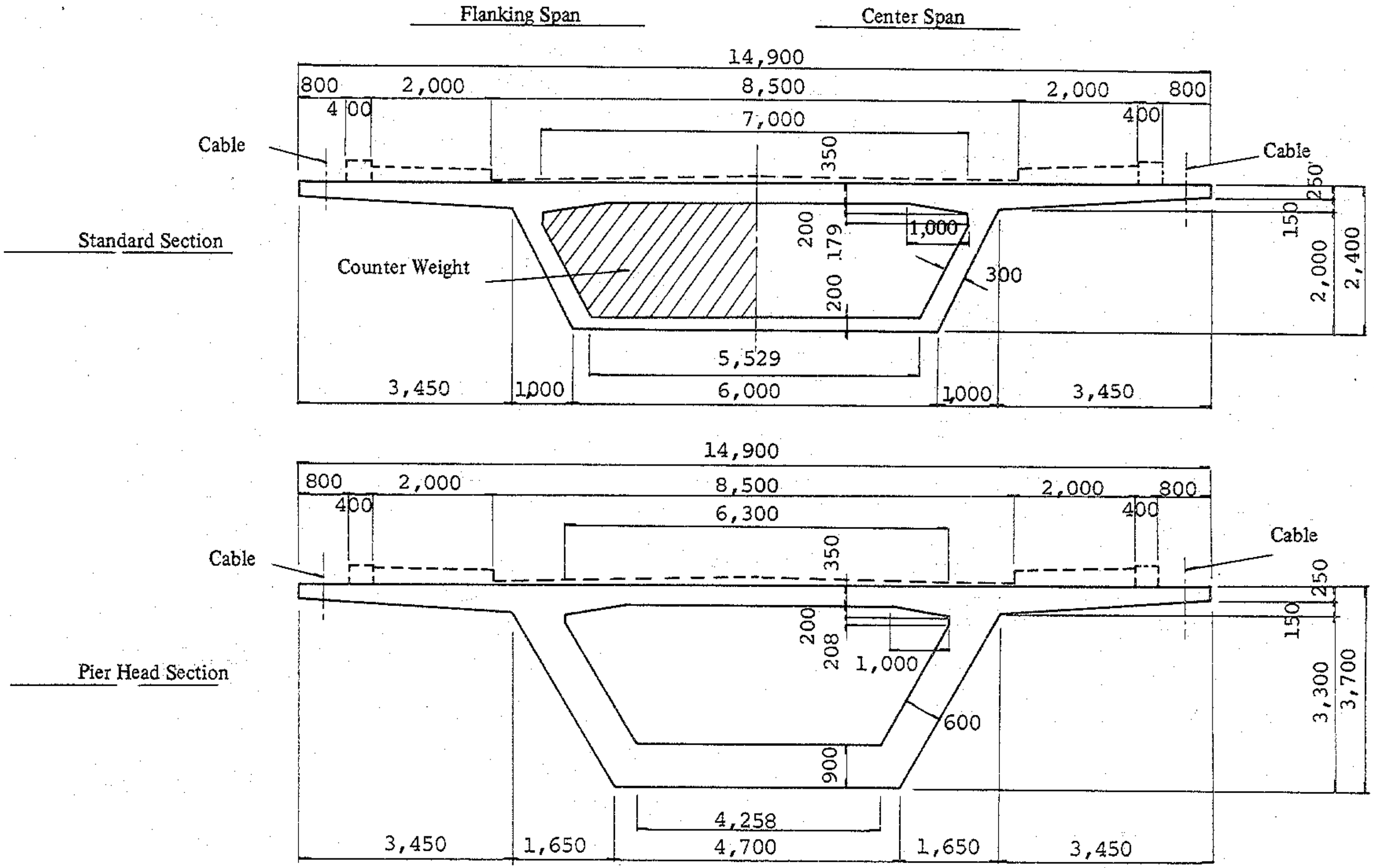
The web has a thickness of 30 cm except a portion near the tower where it has a greater thickness of 60 cm.

The bottom flange has a thickness of 20 cm except a portion near the tower where it has a greater thickness of 90 cm.

The room inside the box of the flanking span is filled with a lean mixture of concrete as a counterweight.

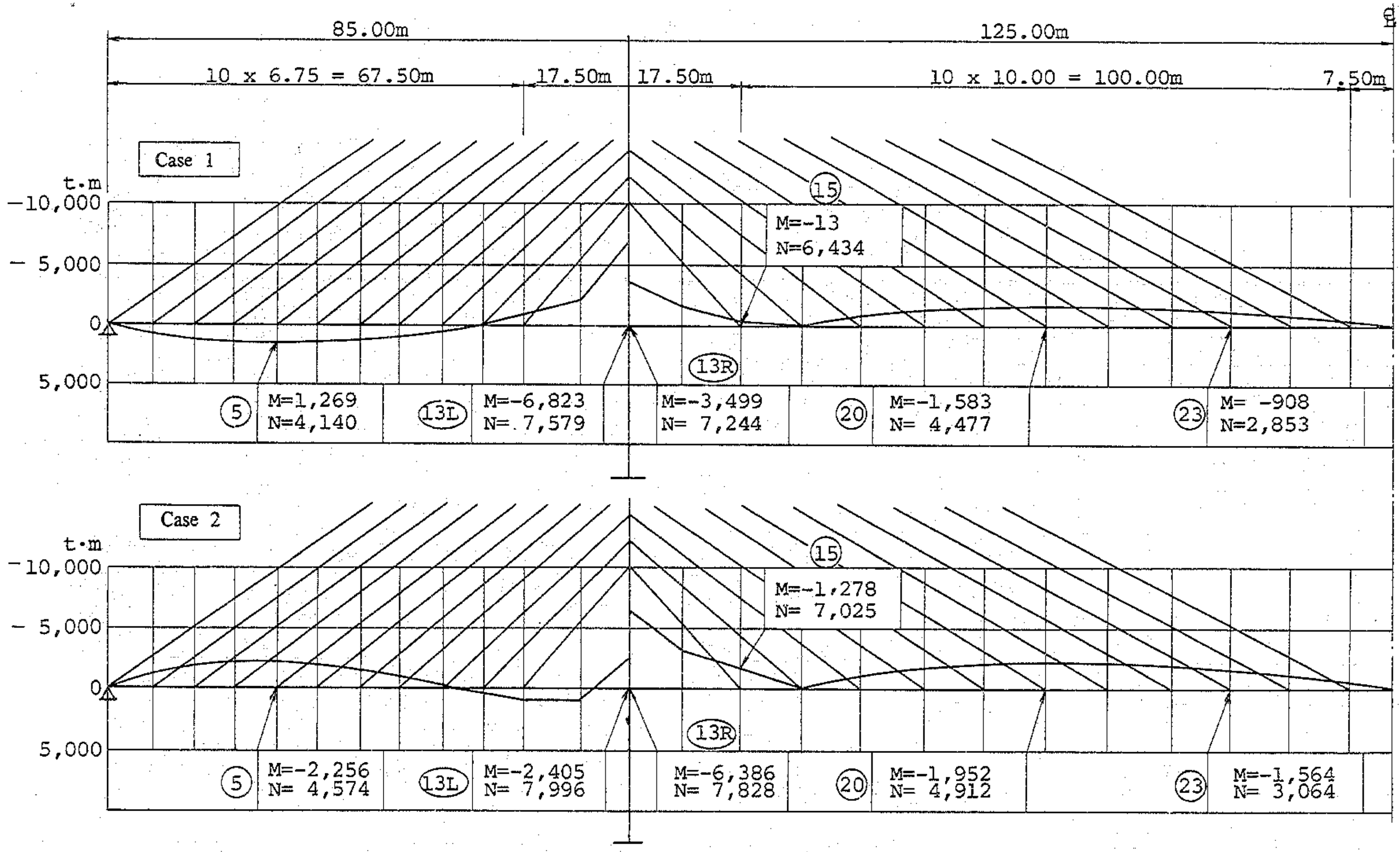
(2) Proportioning of Section

The bending moment of the main girder is shown in Fig. 7–13. The working stresses and corresponding allowable stresses of the principal points of the main girder are shown in Table 7–8. As the table shows, in most of the sections the working stresses are slightly smaller than the allowable stress and accordingly the proportioning of main girder is reasonable.



Unit: mm

Fig. 7—12 CROSS-SECTION OF MAIN GIRDER



Notes:
1) Case 1, Condition which the live loads are carried on the flanking span
2) Case 2, Condition which the live loads are carried on the center span
3) M = Bending Moment   N = Axial compressive force
4) Unit; t·m, t

Fig. 7–13 BENDING MOMENT DIAGRAM OF MAIN GIRDER

Table 7–8 DESIGN UNIT STRESS OF MAIN GIRDER

		Flanking Span		Pier Head				Center Span					
Section Number		5		13 L		13 R		15		20		23	
Section of Girder	A (m^2)	7.774		13.247				7.774					
	I (m^4)	5.372		25.112				5.372					
	Zu (m^3)	7.784		15.717				7.784					
	Zℓ (m^3)	3.142		11.945				3.142					
Axial Force (t)		4,574		7,579		7,828		7,025		4,912		3,064	
Bending Moment (t.m)		–2,256		–6,823		–6,386		–1,278		–1,952		–1,564	
		(u)	(ℓ)	(u)	(ℓ)	(u)	(ℓ)	(u)	(ℓ)	(u)	(ℓ)	(u)	(ℓ)
$\sigma N = N/A$	(t/m^2)	588	588	572	572	591	591	904	904	632	632	394	394
$\sigma M = M/Z$	(t/m^2)	–290	718	–434	571	–406	535	–164	407	–251	621	–201	498
σS	(t/m^2)	–32	–68	–39	–24	–42	–14	–25	–66	–43	–26	–49	3
$\sigma = \sigma N + \sigma M + \sigma S$		266	1,238	99	1,119	143	1,112	715	1,245	338	1,227	144	895
σa	(t/m^2)	1,250	1,250	1,250	1,250	1,250	1,250	1,250	1,250	1,250	1,250	1,250	1,250

Notes:
A = Sectional Area
I = Geometrical Moment of Inertia
Zn = Section Modulus (Upper Flange)
Zℓ = Section Modulus (Lower Flange)
σs = Unit Stress by Creep and Drying Shrinkage
(u) = Top of Upper Flange
(ℓ) = Bottom of Lower Flange

7.3.5 Design of Tower

(1) Configuration

The tower is made of reinforced concrete and the configuration is shown in Fig. 7–14.

The tower has two pillars corresponding to the cables of double planes. The bottom part of the pillar is connected directly with the footing on the foundation piles.

Two pillars are connected with thick beam which supports the main girder and strut against such horizontal thrust as wind force, and make up a rigid frame.

The pillar has a square section of 2.0 m x 2.0 m at the top. This dimension follows the space required for anchoring the cables. However, the pillar has a greater square section of 4.0 m x 4.0 m at the base tapering gradually to sustain the greater bending moment and axial force.

At the corners of the square of each pillar small flanges are made to hide the sockets of the cables.

(2) Proportioning of Section

Table 7–9 shows the design forces acting on the cross-section B–B and C–C of each pillar, and the working stresses. The arrangement of reinforcement in each cross-section is illustrated in Fig. 7–15.

As Table 7–9 shows the working stresses of the reinforcements are very small compared with the allowable stress. This is because the reinforcements are so arranged in order to cover the minimum value which the cross-sectional area of the member demands in avoiding any crackings due to shrinkage of drying concrete.

According to items in the "Specifications for Highway Bridges" by the Japan Road Association, the minimum cross-sectional area of reinforcements for the concrete pillar is 0.8% of the cross-sectional area.

The proportioned cross-sectional area of reinforcement in section B–B and C–C are 0.96% and 0.93% respectively.

Table 7–9 DESIGN FORCE AND WORKING STRESS OF TOWER

Section	Design Force		Working Stress		
				Working Stress	Allowable Stress
B–B	Axial Force	3.230 t	Concrete	68 kg/cm^2	125 kg/cm^2
	Bending Moment	1.792 t·m	Reinforcement	102 "	1,800 "
C–C	Axial Force	4.130 t	Concrete	56 "	125 "
	Bending Moment	3.453 t·m	Reinforcement	56 "	1,800 "

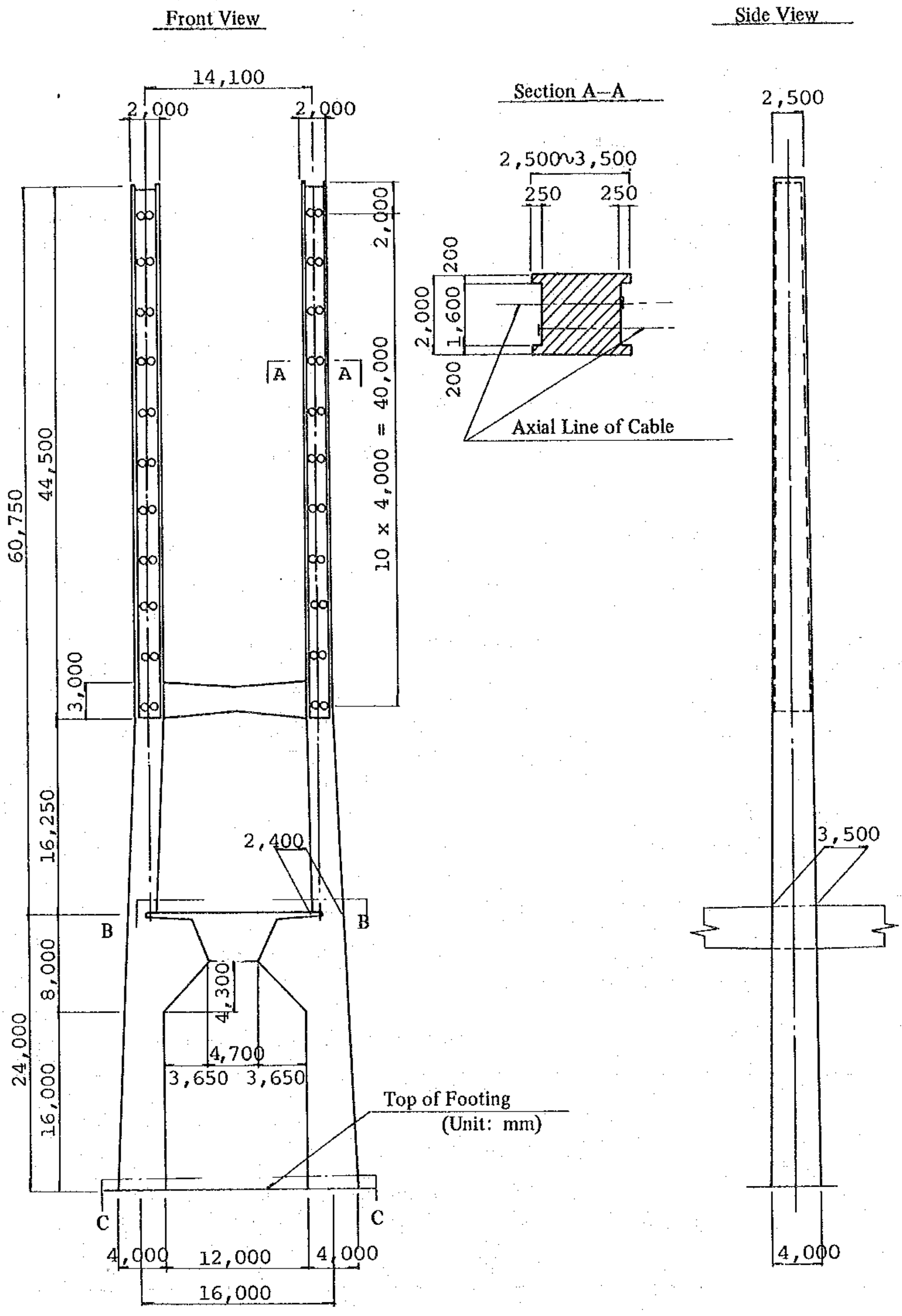
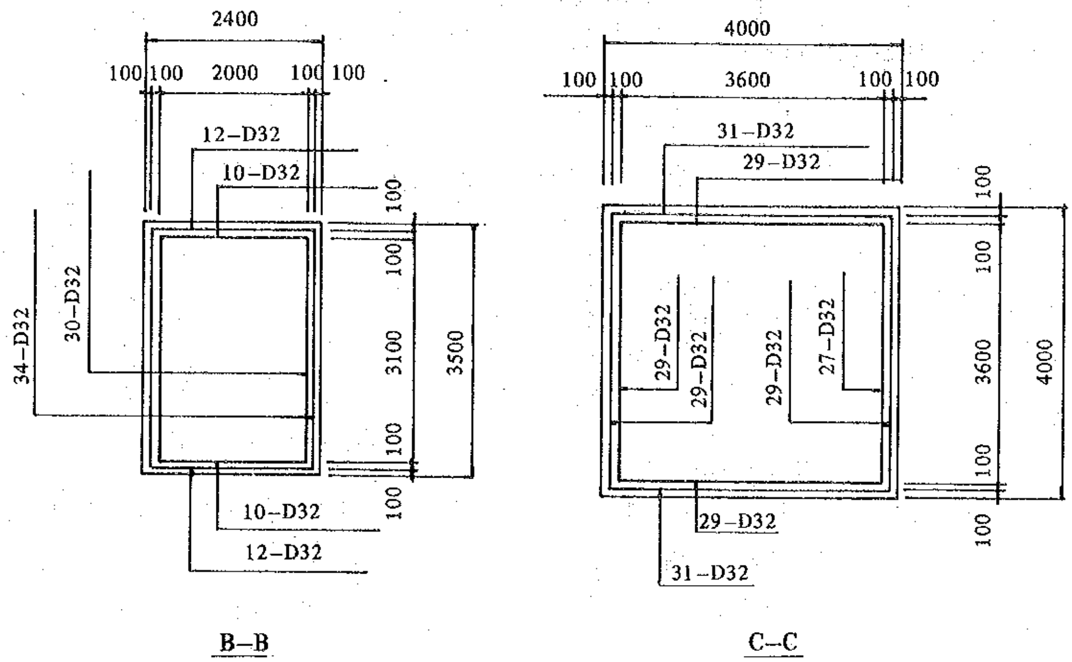


Fig. 7–14 TOWER



Note) 1. Unit: mm
2. 1–D32 = 7.942 cm²

Fig. 7–15 REINFORCEMENTS ARRANGEMENT OF A TYPICAL CROSS-SECTION OF THE TOWER

7.4 SUBSTRUCTURE DESIGN

7.4.1 Design Criteria

(1) Substructure Type

Abutment	(Wall-Type Abutment) + (Spread Footing Foundation)
Tower	(Cast-In-Place Pile Foundation)

(2) Materials and Allowable Stress

a) Concrete

For Abutment and Pier

Design Strength	$\sigma ck = 240 \ kg/cm^2$
Allowable Compressive Unit Stress	$\sigma ca = 80 \ kg/cm^2$
Young's Modulus	$Ec = 2.7 \times 10^5 \ kg/cm^2$

For Cast-In-Place Pile

Design Strength	$\sigma ck = 300 \ kg/cm^2$
Allowable Compressive Unit Stress	$\sigma ca = 80 \ kg/cm^2$
Young's Modulus	$Ec = 3.0 \times 10^5 \ kg/cm^2$

b) Reinforcement

Same to as the standard used in the superstructure.

However, allowable tensile stress of reinforcements which are used for underground or underwater structure shall be 1,600 kg/cm^2.

(3) Allowable Bearing Capacity of Foundations

a) Foundation of abutment

Allowable Bearing Capacity of Soil

$Qa = 40 \ t/m^2$

Allowable Friction Force at Bottom of Footing

$$Fa = \frac{1}{n} \cdot V \cdot \mu$$

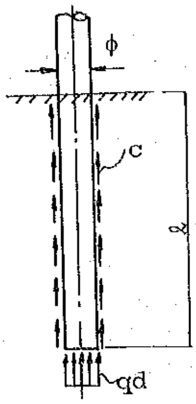
where, Fa: Allowable Friction Force
n : Safety Factor = 1.5
μ : Friction Factor = 0.6
V : Vertical Force Working at the Bottom of Footing

b) Cast-In-Place foundation

Allowable bearing capacity per pile can be represented in the following form;

$$Ra = \frac{1}{n} Ru$$

$$Ru = \pi \cdot \phi \cdot \ell \cdot c + \frac{\pi \phi^2}{4} qd$$



where, Ra : Allowable bearing capacity of a pile
Ru : Ultimate bearing capacity of a pile
n : Safety factor
π : Circular constant
ϕ : Diameter of pile
ℓ : Penetrated depth of pile into the layer
c : Average skin friction per unit area of the surface of a pile
qd : Ultimate bearing stress of the soil at the bottom of a pile

According to the Specifications for Highway Bridges by Japan Road Association, the value of n, c and qd are given as follows;

$n = 3$
$c = 15\ t/m^2$ (Sandy soil)
$qd = 300\ t/m^2$ (Sandy soil)

7.4.2 Design of Abutments

As known from the results of the geotechnical investigation, on either land side, the stiff coral lime stone which is expected to be the bedrock is encountered within 3 m to 5 m deep below the ground surface.

The foundation of the abutment shall be the spread footing type that is to rest on this coral lime stone layer.

The height of the abutment and the breadth of footing are 10 m and 6 m. These dimensions are the same on both Mnarani side and Kilifi side.

The outline of the abutment is illustrated in Fig. 7–16.

A negative force of 25 t to 429 t works from the superstructure to the abutment. In order to use the abutment as a counter weight, the main girder of superstructure is anchored to the abutment by twenty P.C. – strands.

The stability is checked in the following three cases;

Case (1) No working force from superstructure (Under construction)
Case (2) Maximum vertical force from superstructure
Case (3) Minimum vertical force from superstructure

Table 7–10 shows the results of calculation for the stability of abutment.

7.4.3 Design of Tower Foundation

(1) Diameter of Pile

The design diameter of piles shall be 2.0 m from the following reason;

The diameter generally used in the cast-in-place pile by the reverse circulation drilling method is 1.0 m to 2.0 m. In the case where the diameter of the pile is over 2.0 m, the construction cost rises radically because a special drilling machine is needed.

For diameter of 1.0 to 2.0 m, the result of comparison indicates that the diameter of 2.0 m is the most economical.

(2) Penetration Depth

As known from the results of the geotechnical investigation, the stiff silty sand is encountered within 2 m to 4 m below the seabed. Although almost of this silty sand layer is very hard and compact layer N-Value of over 60, relatively loose layer with N-Value of approximately 25 to 40 is encountered in 25 m to 32 m deep below MSL. At these points, the pile shall penetrate through this loose layer to a 35 m depth below MSL.

Fig. 7–17 shows the relation between the soil condition and penetration of piles. As described later, the piles are arranged in three rows in the direction of span and in 5 m distance between each pile center. At both piers on Mnarani side and Kilifi side, the minimum penetration depth below the sea-bed is 25 m and the maximum projecting length over the sea-bed to the bottom of footing is 8 m.

(3) Allowable Bearing Force per Pile

Based on the formula indicated in "Design criteria (3)–b)".

$$\begin{aligned} Ru &= \pi \cdot \phi \cdot \ell \cdot c + \frac{\pi \phi^2}{4} qd \\ &= 3.14 \times 2.0 \times (25.0 \times 15.0) + \tfrac{1}{4} \times 3.14 \times 2.0^2 \times 300.0 \\ &= 3{,}299 \text{ t} \\ Ra &= \tfrac{1}{3} \times 3{,}299 = 1{,}100 \text{ t} \end{aligned}$$

(4) Arrangement and Stability of Piles

Fig. 7–18 shows the arrangement of piles, and Table 7–11 a sense of stability of piles.

Number of piles needed for one pier is eighteen (18). These are arranged in three rows in the direction of span, each having six piles.

The distance between the center of each pile is kept at 5 m which is 2.5 times of the diameter of piles to avoid the decrease of efficiency for the bearing force per pile.

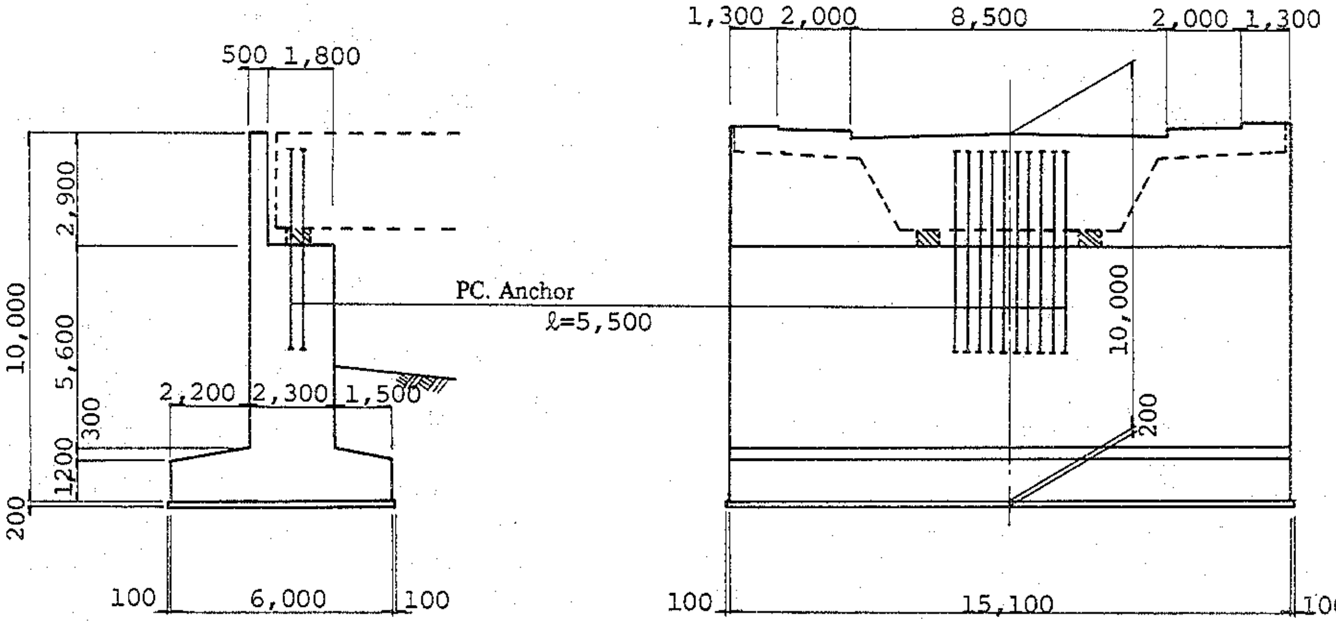


Fig. 7–16 ABUTMENT

Table 7–10 RESULT OF CALCULATION FOR STABILITY OF ABUTMENT

Loading Case	Vertical Force from Superstructure	Working Force at Bottom of Footing			Working Bearing Stress	Allowable Bearing Stress	Allowable Friction Force at Bottom of Footing
		V	H	M			
1	–	1,712 t	408 t	152 t.m	20.6 t/m^2	40 t/m^2	685 t > H
2	-25 t	1,657 t	408 t	260 t.m	21.1 t/m^2	40 t/m^2	663 t > H
3	-429 t	1,253 t	408 t	543 t.m	19.8 t/m^2	40 t/m^2	501 t > H

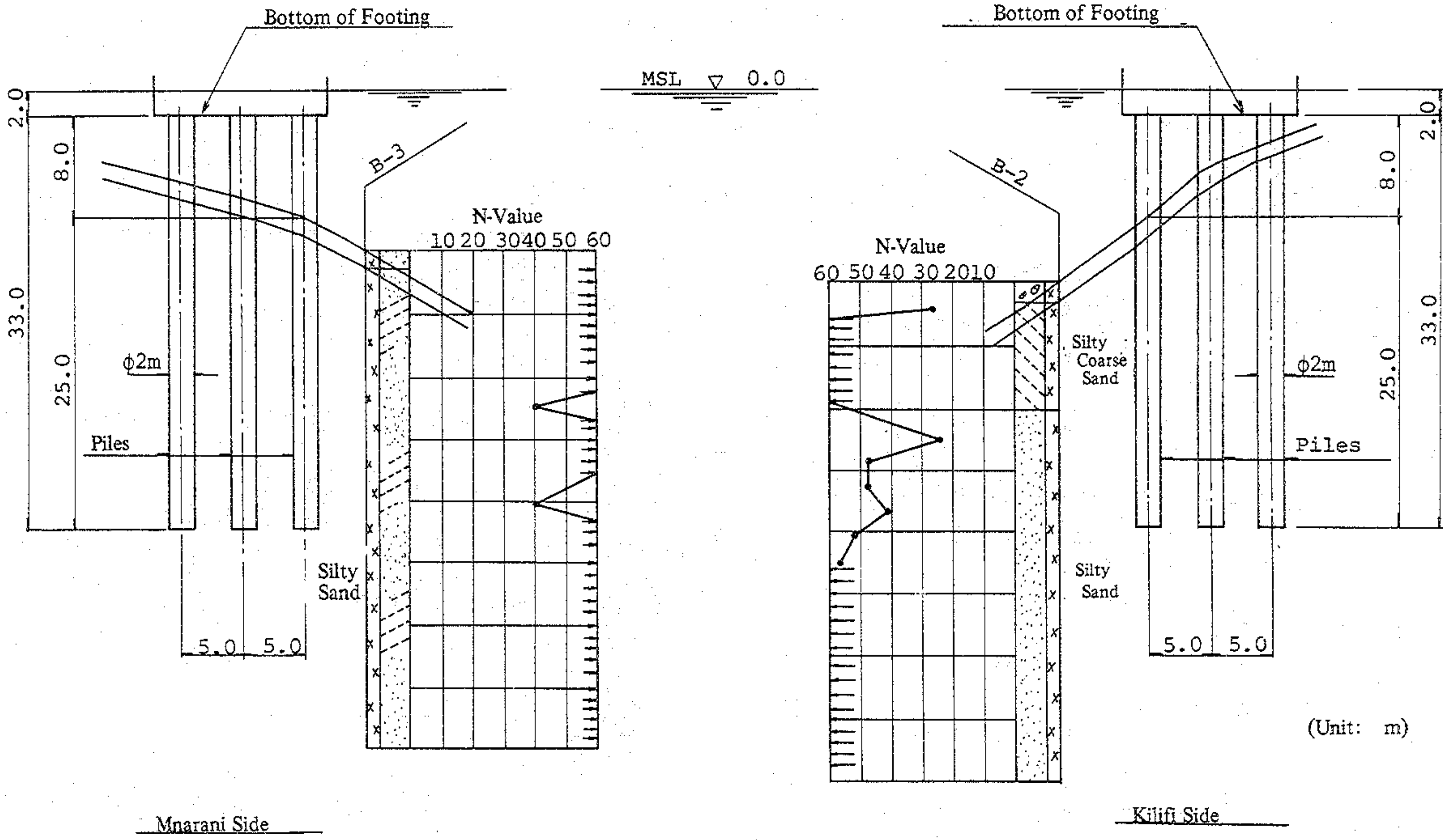


Fig. 7–17 SOIL CONDITION AND PENETRATION OF PILES

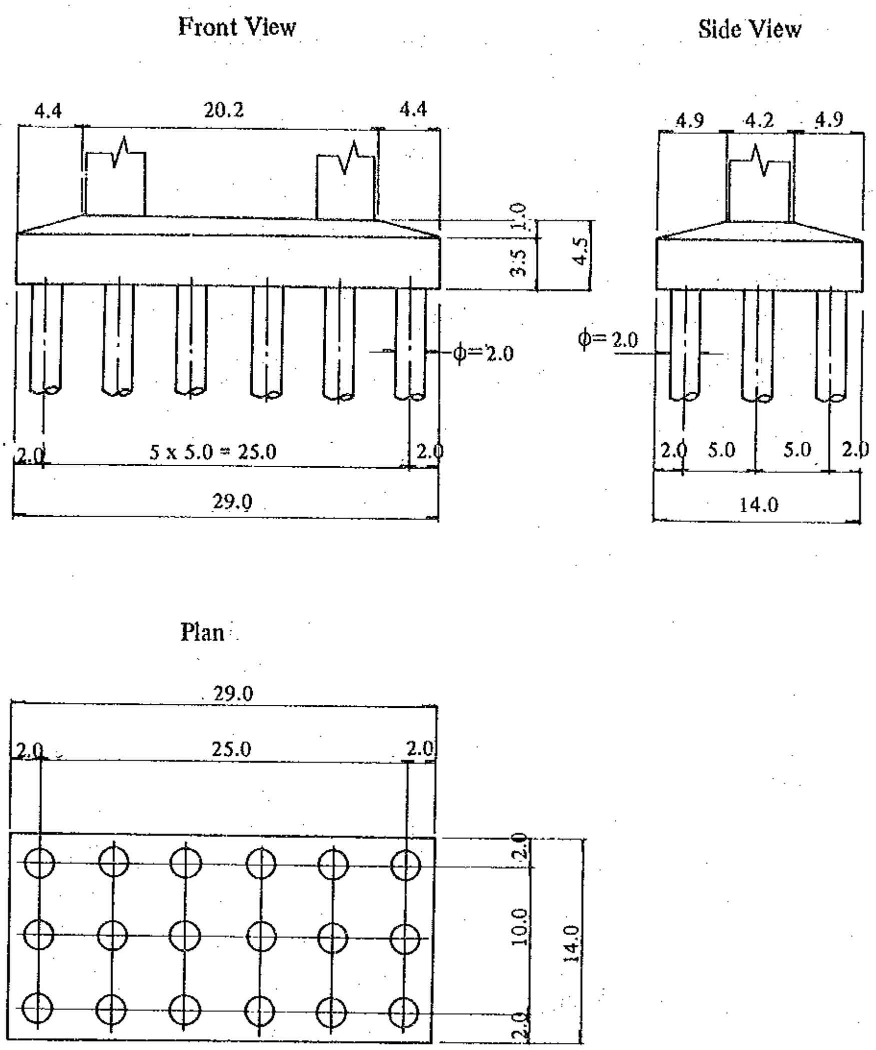
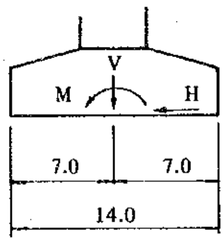


Fig. 7–18 ARRANGEMENT OF PILES

Table 7–11 STABILITY OF PILES

Working Forces at Bottom of Footing	V	16,700 t
	H	70 t
	M	6,700 t.m
Max. Working Force per Pile		1,018 + (38) = 1,056 t
Min. Working Force		818 + (38) = 856 t
Allowable Bearing Stress of Soil		1,100 t
Horizontal Displacement at Top of Pile		1.6 mm
Vertical Displacement at Top of Pile		10.3 mm

Note: (38) indicates the weight of pile itself.



7.5 EXECUTION SCHEME

7.5.1 General

The construction procedure and construction method are illustrated in Fig. 7–19 and Fig. 7–20.

The construction procedure can be divided into 4 stages, that is stage 1 to stage 4.

a) Stage 1

This involves primarily the preparation of working equipment and the construction of temporary structures.

b) Stage 2

This stage covers the construction of abutments and tower piers..

c) Stage 3

The main girders, the main tower and the cables are constructed in this stage.

However, parts of the main girder namely, those near the abutments and the midspan cannot be constructed by the cantilever carriage at this stage.

The main construction equipment such as the tower crane and the cantilever carriage are removed at the end of this stage.

d) Stage 4

The last stage consists of connecting the central section and the end section of the main girder, the adjustment of the stress in the cable. The main work of the construction would be completed. All equipment and temporary structures are finally removed.

7.5.2 Temporary Work

(1) Facility for Concrete Work

The essential facility for concrete work is to be located on Mnarani side considering the case of transporting the necessary construction equipment from Mombasa.

A batcher plant, a simple crusher, a sieving machine are to be established on the site.

The batcher plant would have a capacity of 50 m^3 per hour, determined by taking an average of 50 cm depth of the footing concrete a day.

Unsalinate water for the concrete work is to be taken from a well which is established in stage 1. The water tank would have a capacity of about 100 m^3.

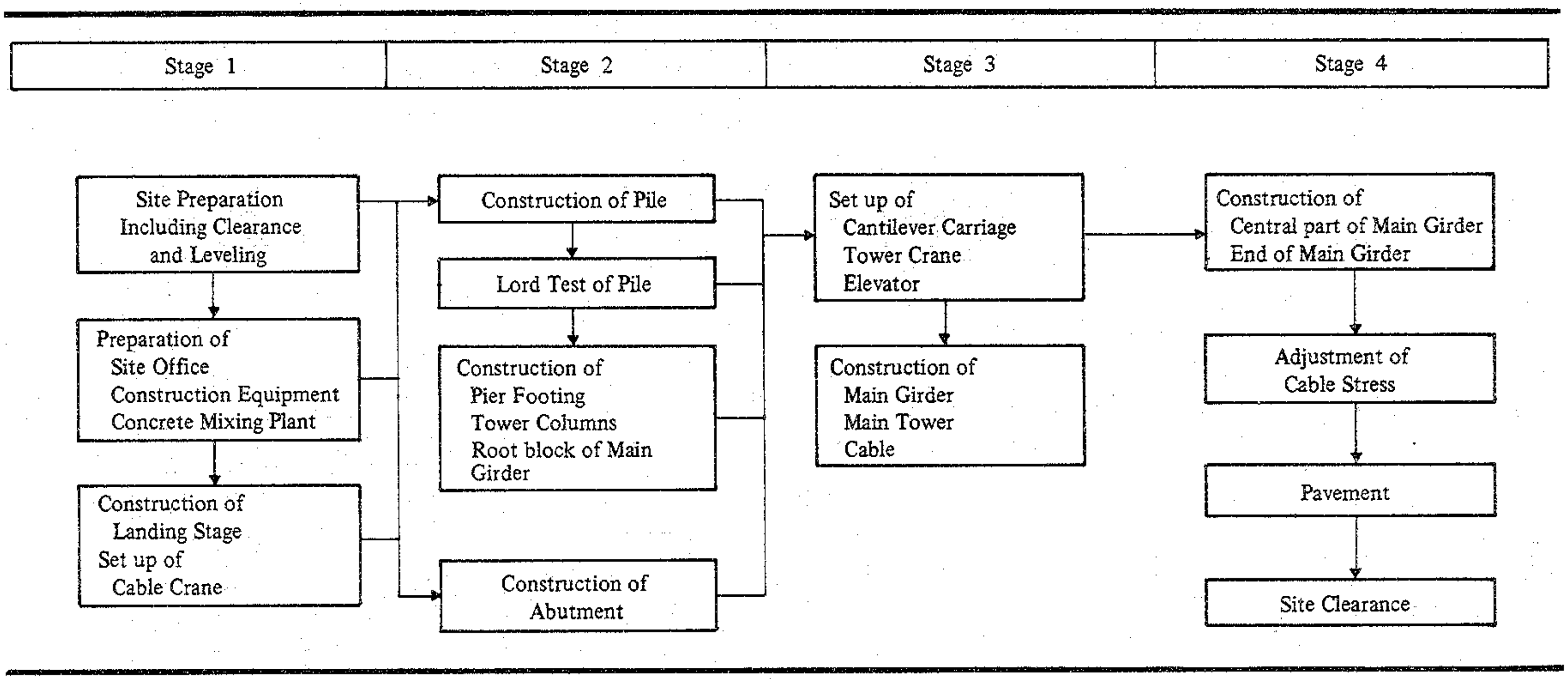


Fig. 7–19 CONSTRUCTION PROCEDURE

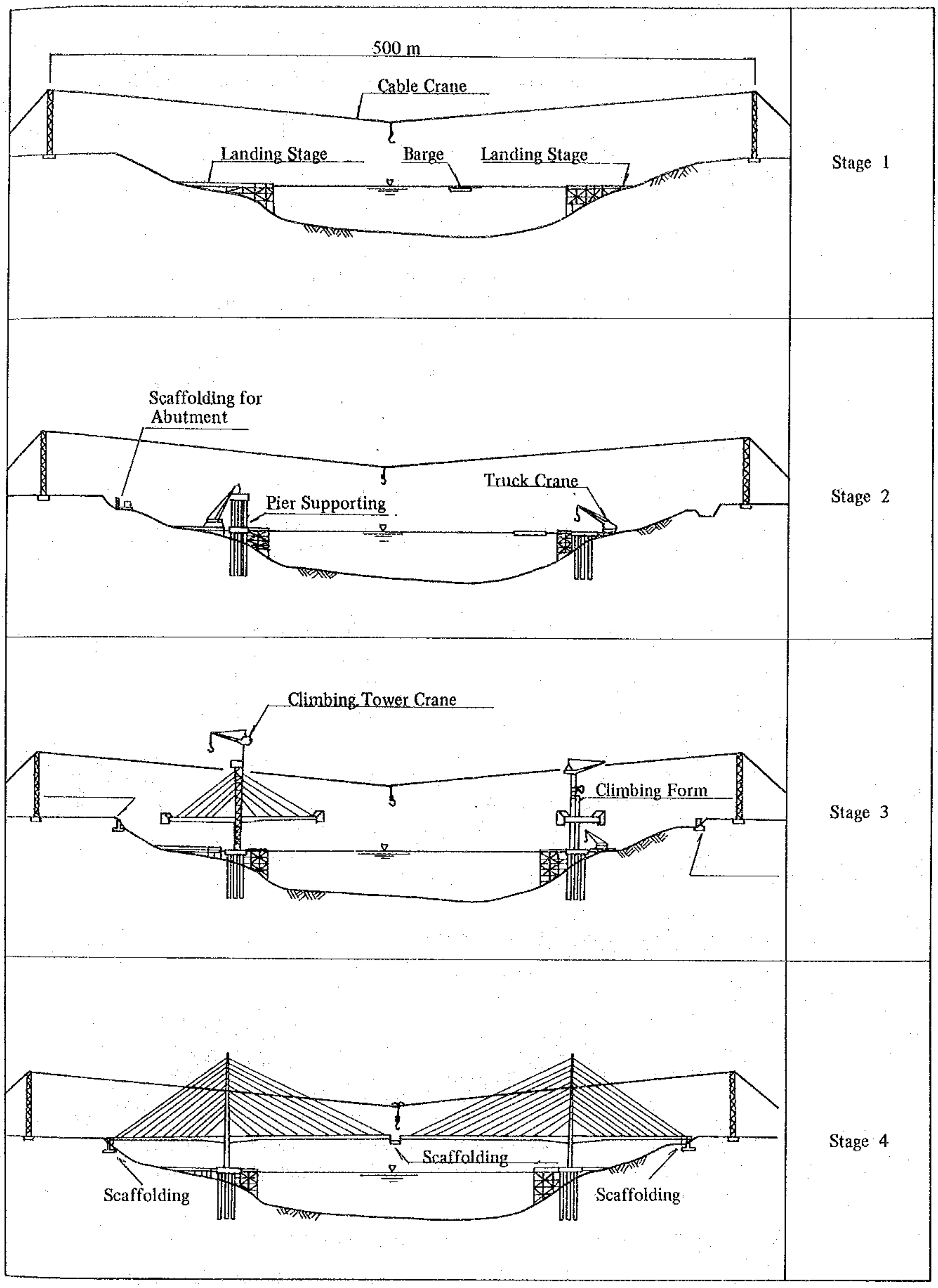


Fig. 7–20 CONSTRUCTION METHOD

The aggregate for the concrete can be transported from the following two areas:

a) Coarse aggregate : Chasimba Rock Quarry

b) Fine aggregate : Tiboni Sand and Sabaki River Sand

(2) Cable Crane

The cable crane is to be established for the purpose of transferring the construction material such as concrete, reinforcement scaffolding and P.C. bar.

A cable crane with a length of about 500 m and a height of about 50 m and a capacity of 5 ton would be established.

(3) Temporary Landing Stage

The temporary landing stages are essential for the construction of the piers.

The landing stage would have a length of 70 m on Mnarani side and 48 m on Kilifi side.

H-shaped steel with a size of 400 x 400 mm is used for the supporting colums. The girder for the platform would be made with prefabricated steel deck.

A pile length of 5 m under the seabed is required taking into account the load of the truck crane (60 ton).

The typical cross-section of the landing stage is illustrated is the Drawing.

The supporting pile for the landing stage would be constructed by vibration hammer where the water is relatively shallow. While of the water is relatively deep, it would be constructed by the vibration hammer too, after the holes for the pile are excavated by auger machine.

7.5.3 Construction of Tower

(1) Piles for Foundations

Rotary type boring equipment is used for the construction of the foundation piles. In the water, the piles are protected from water by the 2.2 m diameter steel casing pipe.

The casing pipe is driven by the vibro-hammer to a depth of 2 ∿ 3 m below the seabed. A guide jacket which is fixed to the supporting piles of the landing stage is established to reduce any horizontal shake during the construction of the casing pipe.

Bentnite water would be used for the protection of the boring hole.

The pile must be constructed at over 25 m under the seabed which is composed of a hard layer of compact sand with N value of over 60.

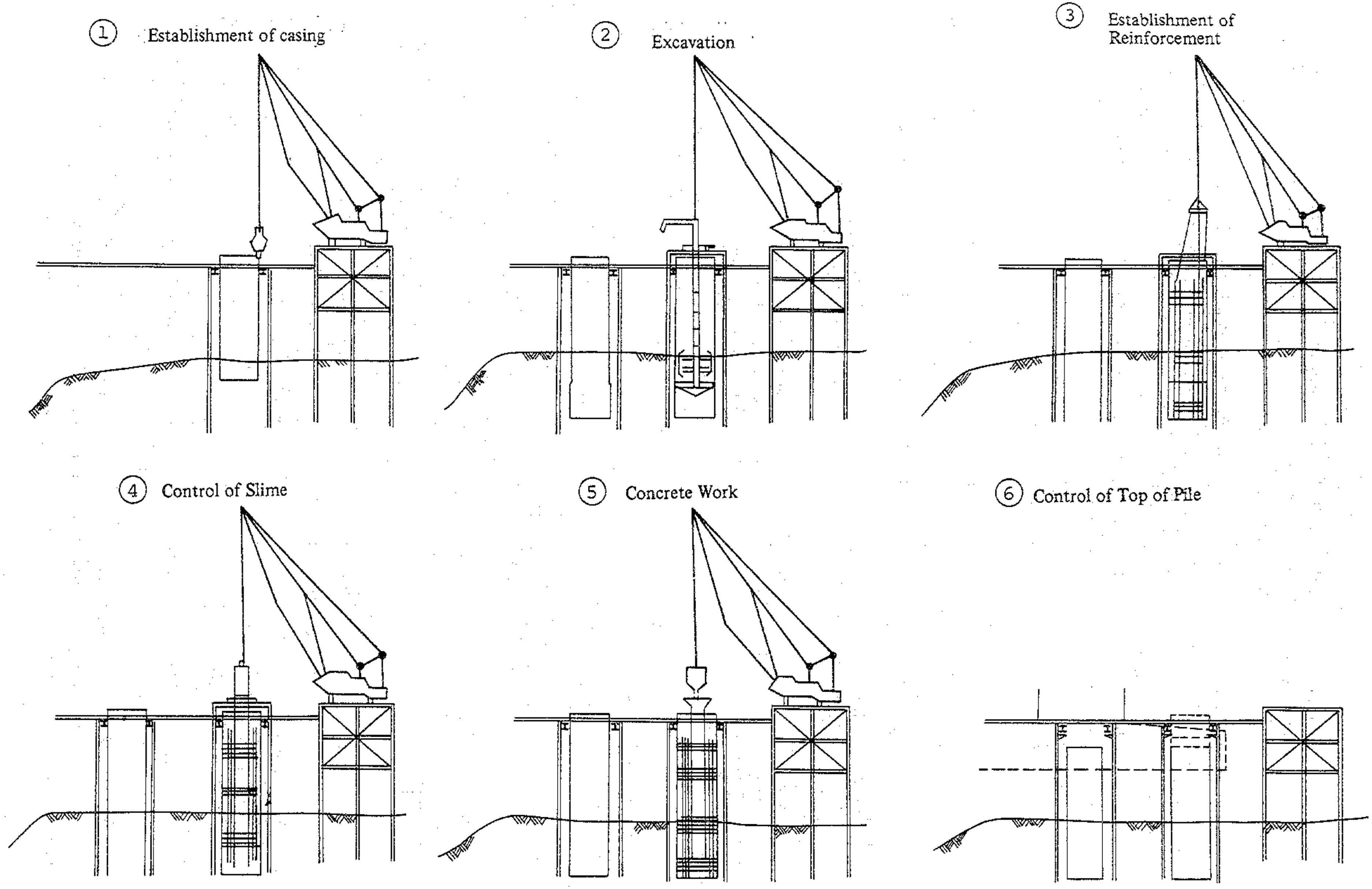


Fig. 7–21 CONCEPTIONAL PLAN OF PILE CONSTRUCTION

The excavation speed for the construction of the pile is estimated at about 0.5 $\sim$ 0.6 m per hour. Subsequently, the working hours for the boring of one pile is estimated at about 48 hours.

The concrete work for the piles of the Mnarani side would be carried out by concrete backets mounted on trucks running on the landing stage. On the otherhand, the Kilifi side would be carried out by a combination of cable crane and grand hopper mounted on the barge. A 15 m^3 per hour working capacity is required. A conceptional working plan for the construction of the piles is illustrated in Fig. 7–21.

(2) Footing

The bottom scaffolding for the footing is hunged from the temporary latticed girders which are connected to the top of the piles that have already being established.

The pre-cast concrete plates are used as the scaffolding for the lateral faces of the footing.

A conceptional working plan for the construction of the footing is illustrated in Fig. 7–22.

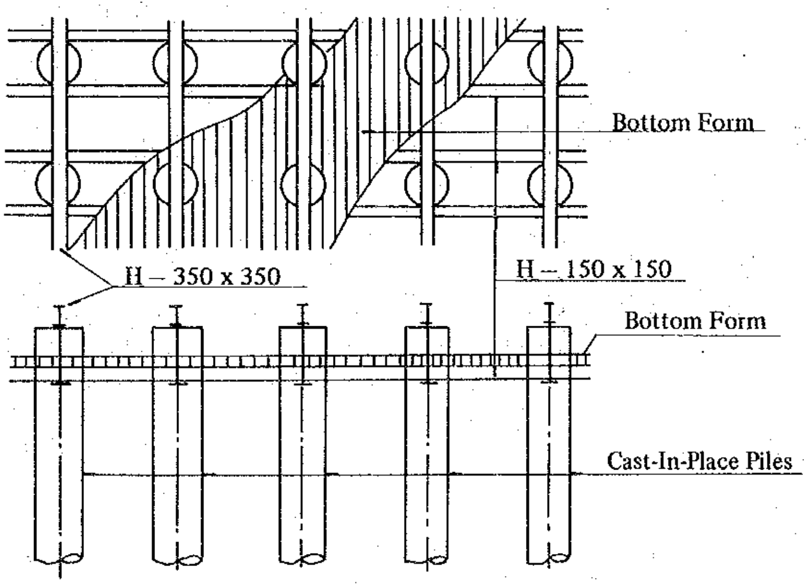


Fig. 7–22 BOTTOM SCAFFOLDING FOR FOOTING

7.5.4 Construction of Superstructure

(1) Main Girder

After the construction of the tower legs, the first block of the main girder is built.

A comparatively large cantilever carriage (350 t capacity, block length of 5 m) is employed for the construction of the main girder.

As the cable interval on the main girder is about 10 m wide following the construction of the two (2) blocks, the cahle is stretched and connected.

The main tower is extended upward gradually stage by stage so is the main girder.

A conceptional working plan for the main girder and the cantilever carriage are illustrated in Fig. 7–23 and Fig. 7–24 respectively.

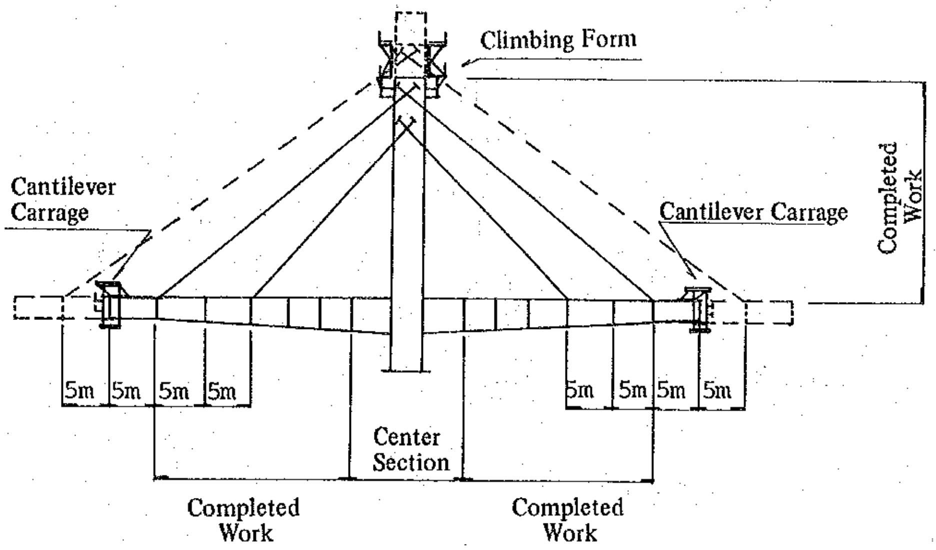


Fig. 7–23 CONCEPTIONAL WORKING PLAN OF MAIN GIRDER

Two cantilever carriage equipment are erected on both the tower of Kilifi and Mnarani sides. The main towers and main girders are constructed simultaneously.

Lifting and the assembling of the cantilever carriages are carried out with climbing tower crane which is located on the footing of the pier.

The assembling of the cantilever carriage structure needs a period of 15 days.

The concrete volume placed by the cable and tower crane with every shift of the cantilever carriage is about 40 m³.

The time needed for the concrete placing by the cantilever carriage and the stretching of the cable are 10 and 5 days respectively. It follows that the cycle time of the block between the cables is 25 days.

The concrete for the end sections of the main girder, each with a length of 2.25 m near the abutment and a central section, 6.5 m long at the midspan is placed and supported with suspended scaffolding without a cantilever carriage.

(2) Main Tower

The construction of a high structure such as the main tower for the cable stayed bridge, the slip form method or climbing form method are generally

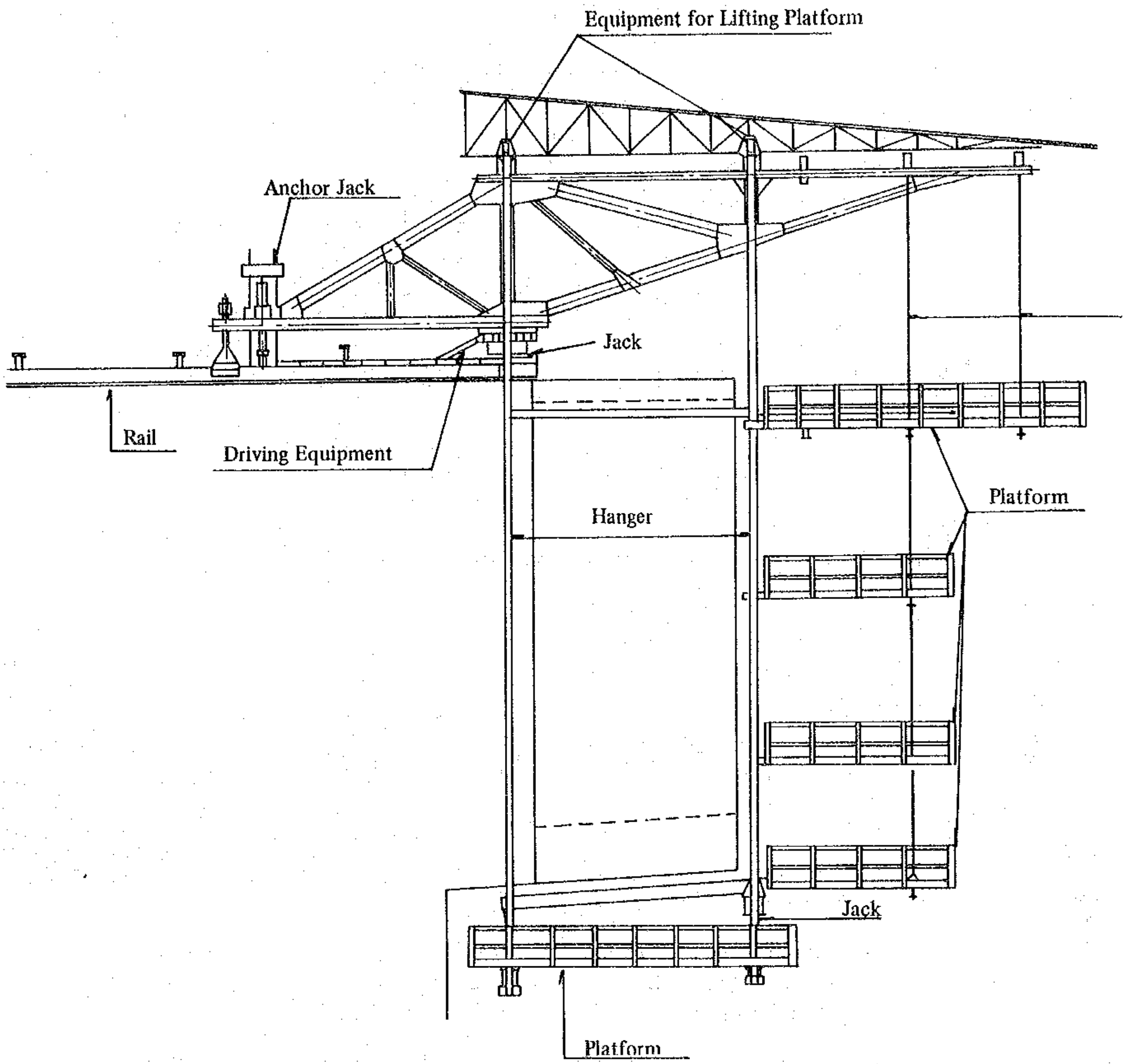


Fig. 7–24 CONCEPTIONAL PLAN OF CANTILEVER CARRIAGE

recommended on account of their economic merit and safety.

In the slip form method, the form slides continuously, while in the climbing form method it shifts in stages each with a length of a block (2 $\sim$ 4 m).

The climbing form is adopted in this project.

The main tower consists of number 18 blocks each with a length of 3.5 m. The concrete volume of a block is about 50 m^3 and the construction period of a block is 10 days.

(3) Cable

The order of the construction of the cables is as follows:

a) The sockets for anchoring the cables are installed first, on the tower and the main girder.

b) Polyethylene tubes for covering the cables are welded on the bridge floor at the required length.

c) Each of these polyethylene tube is lifted subsequently to its desired position.

d) A strand is inserted in the tube, and stretched toward the lower end by a winch to minimize its sag.

e) All the remainder strands are inserted into the tube from the tower side by using the pushing machine.

f) The cable is tensioned in a prescribed force and anchored.

g) After the construction of all the members of the bridge including the filling of the gaps of the main girder, the tensile forces of all the cables are adjusted.

h) The gap between the strands and the tube is grouted with cement mortar.

7.5.5 Working Process and Main Equipments

Table 7–12 WORKING PROCESS OF CONSTRUCTION OF BRIDGE

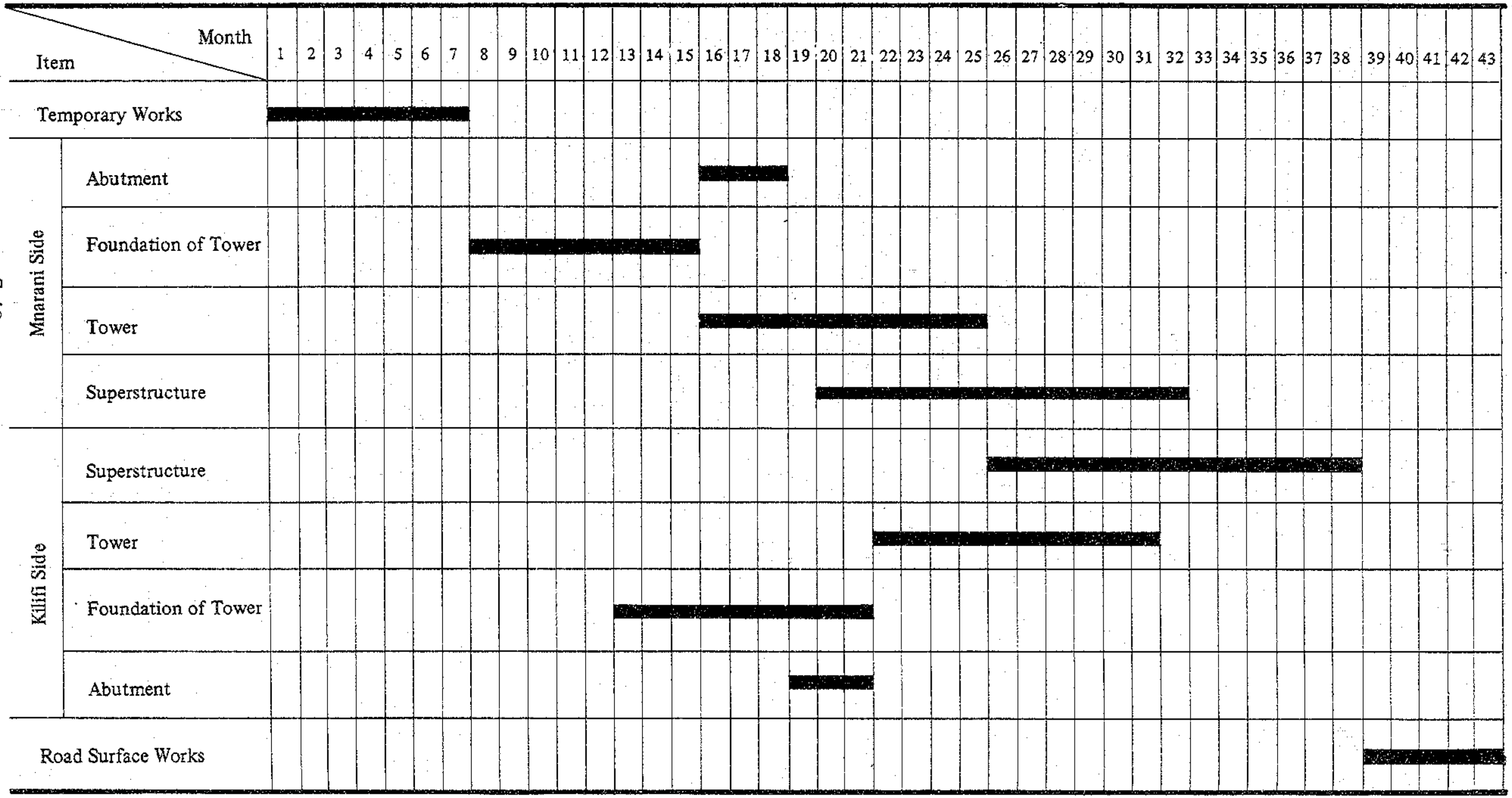
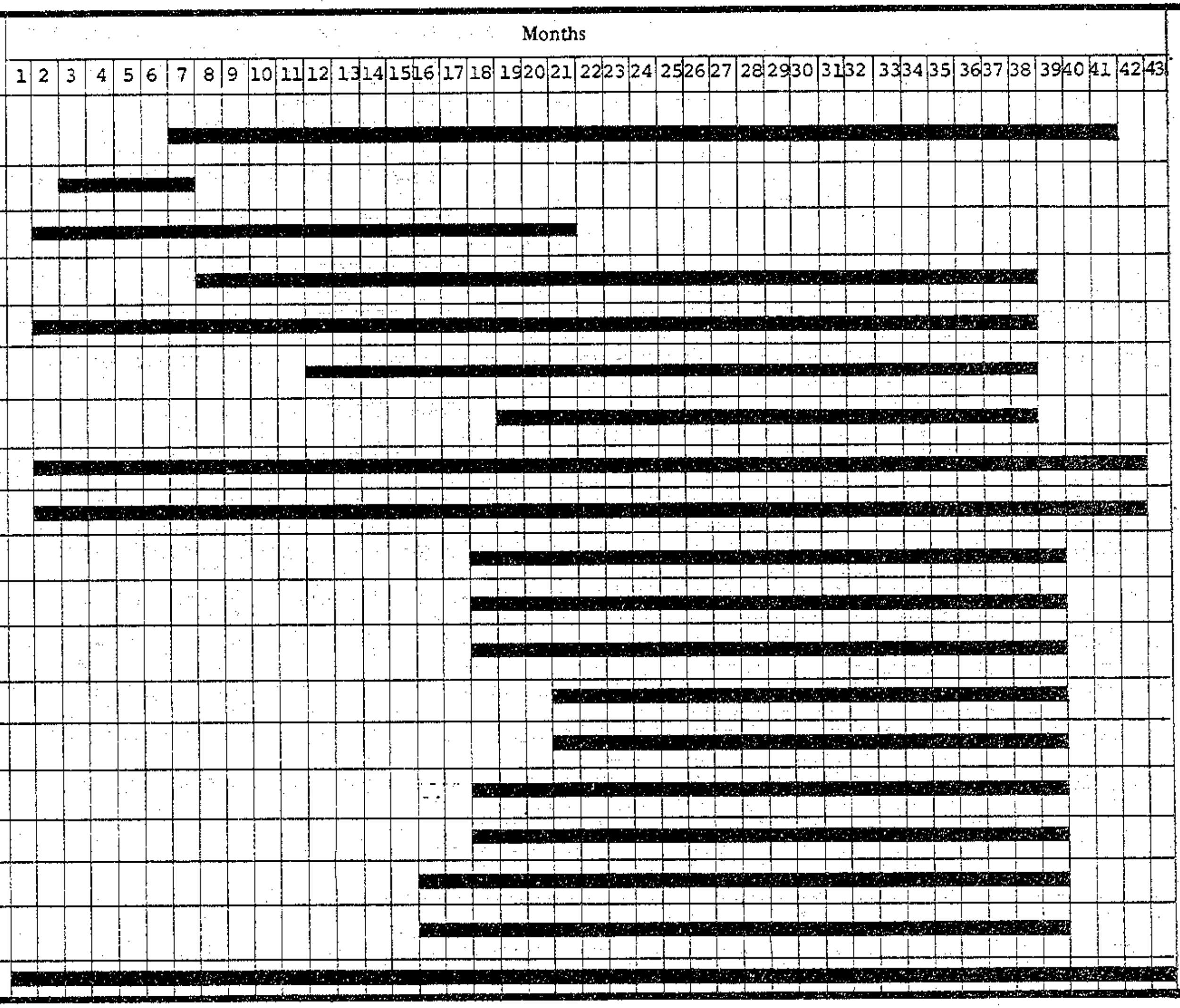


Table 7–13 MAIN EQUIPMENTS FOR BRIDGE WORKS AND THEIR PERIOD FOR USE (1)

Item	Capacity	Number	Months (1–43)
Deck Barge	500 t	2	
Cable Crane	5t ℓ=500m	1	
Crawler Crane	80 t	1	
Truck Crane	35 t	1	
Truck Crane	20 t	1	
Concrete Plant	50 m^3/h	1	
Cone Crusher	45 KW	1	
Dynamo Electric Machine	300 KVA	2	
Dynamo Electric Machine	175 KVA	3	
Dynamo Electric Machine	100 KVA	5	
Dynamo Electric Machine	75 KVA	3	
Reverse Circulation Drill	ϕ3m 35KW	1	
Vibro Hammer	150 KW	1	
Clamshell Backet	2 m	1	
Three Wing Bit	ϕ = 2∿2.4m	1	
Muddy Water Tank	30m^3 x 2	6	
Welder	500 A	2	
Vibro Hammer	60 KW	1	
Compressor	10.5 m^3	1	

Table 7–14 MAIN EQUIPMENTS FOR BRIDGE WORKS AND THEIR PERIOD FOR USE (2)

Item	Capacity	Number
Compressor	$5.0 m^3$	1
Earth Auger	22 KW	1
Back & Forth	$0.3 m^3$	1
Tyredozer	19 t	1 -
Winch	35 KW	2
Climing Tower Crane	45 PS 100 tm	2
Cantilever Carriage	350 tm	4
Concrete Backet	$1.5 m^3$	4
Vibrator	ϕ60	12
Dywidag Jack	70 t	4
Dywidag Jack	50 t	3
Electric Pump for Dywidag Jack		5
Grout Mixer		1
Grout Pump		2
Dynamometer	100 t	1
Bar Bender		1
Freyssinet Jack	S_6 Type	8
Electric Pump for Freyssinet Jack	LET Type	8
Truck	3.5 t	1



8 . ESTIMATION OF PROJECT COST

8.1 GENERAL

The project cost is calculated on the basis of the same frameworks as in the Interim Report which was conducted in Aug. 1983. (Feacibility study on Kilifi Bridge Construction Project; Interim Report).

The project cost presented in this report is expressed in July, 1983 prices, and price contingencies such as domestic inflation, inflation in countries of likely foreign suppliers, sectoral price trends and project impact on local prices are not considered.

8.1.1 Cost Estimation Method

The cost estimation process is shown in Fig. 8–1.

8.1.2 Conditions

a) The project cost is presented in Kenya Shillings (K.Shs.)

b) The project cost estimate is carried out based on 1st July,1983 prices.

c) The exchange rate of the US dollar is 13.06 K.Shs. at this point.

d) The project cost is divided into foreign currency and local currency.

e) All imported construction equipment and materials are duty free.

8.1.3 Basic Cost

The basic cost consists of:

a) The cost of labour

b) The equipment and material for construction

c) Other necessary items.

8.1.4 Construction Cost

The components of the construction cost are as follows:

a) Basic cost

b) Overhead cost

c) Profit of contractor

d) Tax.

8.1.5 Foreign Currency

The components of the foreign currency are as follows:

a) Costs of imported equipment and materials such as steel products and others (CIF price).

b) A portion of the material cost for cement, asphalt and fuel.

c) A portion of labour cost.

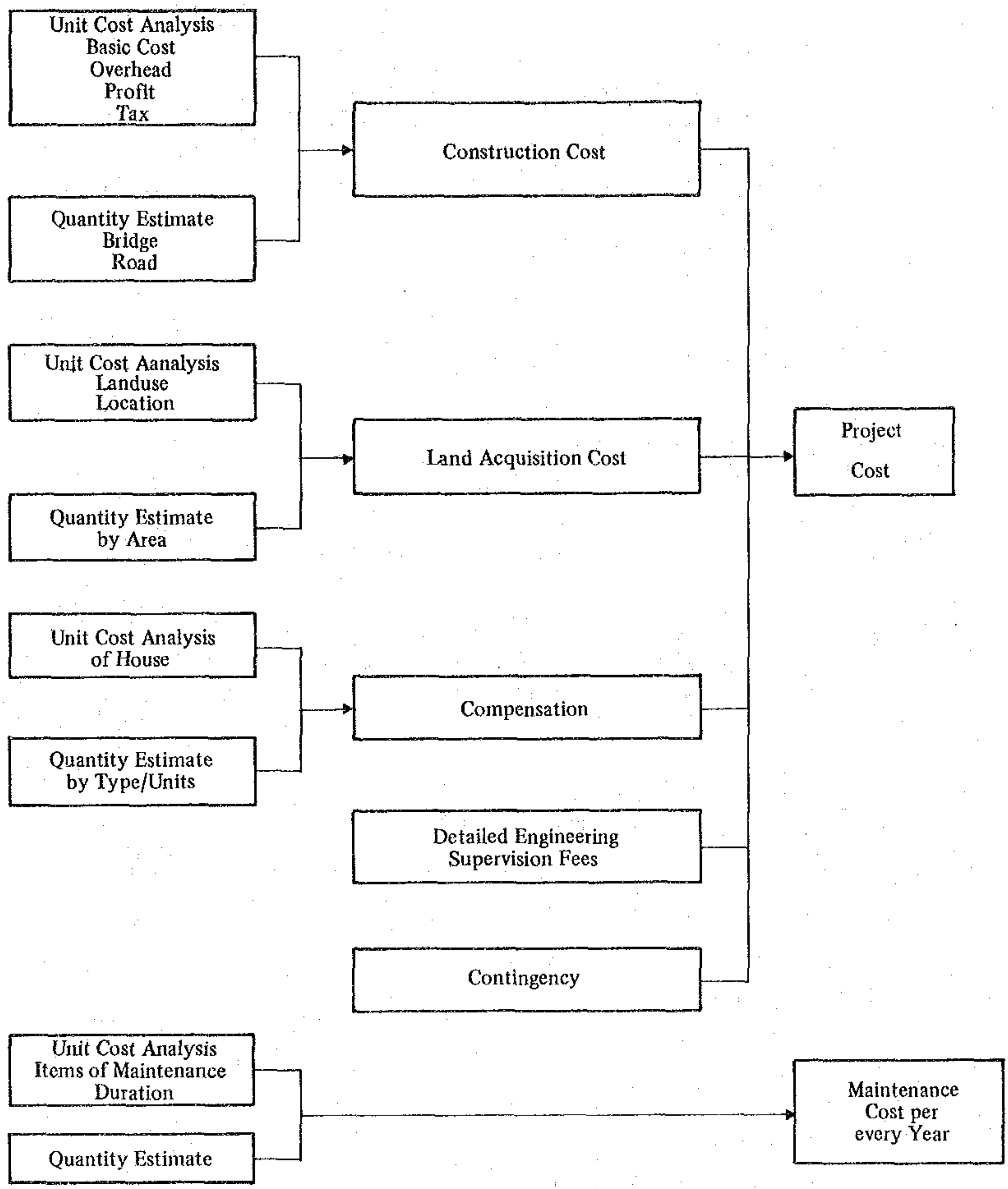


Fig. 8–1 FLOW OF COST ESTIMATION PROCESS

d) A portion of the detailed engineering and supervision service fees.

e) A portion of the overhead, profit and contingency.

8.1.6 Local Currency

The components of the local currency are as follows:

a) Purchase Cost of domestic products such as crushed stone, sand, paint etc.

b) Transport cost in Kenya.

c) A portion of the material cost for cement, asphalt and fuel.

d) A portion of labour cost.

e) A portion of the detailed engineering and supervision service fees.

f) A portion of the overhead, profit and contingency.

g) Cost of land acquisition and compensation.

h) Tax.

8.2 UNIT COST ANALYSIS

The unit cost analysis is studied for the main construction work items on the basis of the Analysis of Contract Rates, 1980 and the other related data. However, unit cost for items such as bored cast-in-place concrete pile, erection of superstructure and others, are individually decided by each particular estimation with reference to comparable domestic and foreign constructions.

The basic guidelines for the unit cost analysis are as follows:

a) Construction materials available in this country.

b) Market prices of the construciton materials and equipment.

c) Labour cost, power and capability in this country.

d) Construction efficiency and capacity in this country.

e) Construction experience in this country.

Bearing the above guidelines in mind, the unit cost analysis is carried out.

8.2.1 Components of Unit Cost

The unit cost itself is also divided into two components, i.e. foreign and local currency.

8.2.2 Cost of Construction Materials and Equipment

The cost of major construction materials, which are produced in this country, is derived after discussion with the related offices and a reference to the pricelists of private companies. The market price list of the major domestic materials is shown in Table 8–1.

The cost of construction materials and equipment, which are not produced in this country, is referred to the foreign market prices. The cost list of the foreign products is shown in Table 8–2.

8.2.3 Labour Cost

The labour cost is estimated on the basis of the collected data. The cost list of labour is shown in Table 8–3.

Table 8–1 MARKET PRICE OF MAJOR DOMESTIC MATERIALS

(In K.Shs.)

Material	Description	Unit	Market Price	Remarks
Sand		M^3	58	
Crushed Stone	Crusher Dust	"	31	
	1/4"	"	93	
	1/2", 3/4", 1", 2"	"	97	
	Limestone	"	350	delivered Kilifi
Stone Boulders	6" x 9"	"	70	
Cement	Standard	T	1,143	1,648 K.Shs./m^3
	Portland	"	1,213	1,749 K.Shs./m^3
	Sulphate Resisitng	"	12 85/30	1,853 K.Shs./m^3
Gas/Oil	Gasolene	L	7/50	Regular
	Diesoline	"	5/50	
	Engine Oil	"	21/50	Regular
Concrete Block	140mm x 190 x 390	No	5/75	
	240mm x 190 x 390	"	8/97	
Timber	150mm x 25 Cedar	M	6/42	
	– do – Cypress	"	5/80	
	– do – Hardwood	"	23/84	
	300mm x 500mm – do –	"	95/35	
Paint	Plastic Emulsion	L	114/20	ex Factory
	Enamel Paint	"	71/60	– do –
Cut-back Bitumen	Grade MC30	"	4/10	delivered Kilifi
	Grade 800/1400	"	5/30	– do –

Table 8–2 COST OF FOREIGN PRODUCTS (CIF PRICE)

(In K.Shs.)

Item	Class	Unit	Cost	Remarks
Steel Deformed Bar	JIS G3112 SD30	T	4,150	
H-Shape	JIS G3101 SS41	T	6,200	
	JIS G3106 SM50Y	T	6,900	
Plate	JIS G3101 SS41	T	6,300	
	JIS G3106 SM50Y	T	7,000	
P.C. Rod	JIS G3109 SBPR 95/120	T	17,600	
P.C. Cable	JIS G3536 SWPR 7A	T	18,700	
Shoe	Cast Steel	T	47,000	
	Gum Pad t = 70 mm	M	20,000	
Expansion Joint	Rubber Δ = 150 mm	M	16,500	
Guard Rail	Steel	M	268	
Handrail	Alumi. h = 1.2 m	M	2,400	
Lighting (Na. Lamp)	Steel Pole h = 10 m	No.	11,000	
	Alumi. Pole h = 10 m	No.	27,000	
Traffic Sign	Alumi. $A = 2\ m^2$	No.	18,000	
				Depreciation rate
Cone Crusher	45 kw	No.	1,145,000	200×10^{-6}/Hour
Asphalt Plant	30 t/h 101 kW	No.	2,390,000	370×10^{-6}/Hour
Concrete Plant	$50\ m^3/h$ 74 kW	No.	2,845,000	299×10^{-6}/Hour
R.C. Drill	ϕ3.0 m 35 kW	No.	1,222,000	400×10^{-6}/Hour
Vibro Hammer	60 kW	No.	474,000	474×10^{-6}/Hour
	150 kW	No.	1,216,000	545×10^{-6}/Hour
Earth Auger	22 kW	No.	291,000	445×10^{-6}/Hour
Compresser	$5\ m^3$ 52 PS	No.	159,000	$2,500 \times 10^{-6}$/Day
	$10.5\ m^3$ 105 PS	No.	308,000	$2,500 \times 10^{-6}$/Day
Cement Silo	100 t	No.	729,000	775×10^{-6}/Day
Climing Tower Crane	45 PS 100 tm h = 96 m	No.	2,732,000	$1,227 \times 10^{-6}$/Day
Muddy Water Tank	$30\ m^3$	No.	111,000	$1,778 \times 10^{-6}$/Day
Deck Barge	500 t	No.	1,486,000	$1,067 \times 10^{-6}$/Day
Cantilever Carriage	350 t-m 83 t	No.	2,276,000	$1,510 \times 10^{-6}$/Day
Vibrator	60 ϕ 1.2 kW	No.	8,400	$3,889 \times 10^{-6}$/Day
Winch	35 kW	No.	105,000	$3,187 \times 10^{-6}$/Day
Roof, Floor & Others	for Cantilever Carriage	No.	226,000	
Dywidag Jack	70t	No.	46,500	
	50t	No.	39,600	
Elec. Pump for D.J.		No.	39,500	
Freyssinet Jack	S6 Type	No.	98,400	
Elec. Pump for F.J.	LEP Type	No.	50,100	
Bar Bender		No.	49,100	
Grout Mixer		No.	21,800	
Grout Pump		No.	48,300	
Dynamometer	100 t	No.	44,800	
Three-Wing Bit	2.0 ∿ 2.4 m	No.	57,300	
Concrete Backet	$1.5\ m^3$	No.	39,800	
Welder	500 A	No.	168,000	

Note: Refer to Table 8–4 for the purchase price of other equipments.

Table 8–3 LABOUR COST

(In K.Shs. at 1983 Price)

Item	Unit cost per 8 hours at day
1. General Labourer	45.5
2. Steel Bender and Fixer	68
3. Mason	68
4. Truck Driver	68
5. Operator	100
6. Foreman	100

8.2.4 Hourly Rate of Construction Equipment

To estimate the unit cost of the main working items of the bridge and approach road, it is necessary to know the costs which are involved in the use of construction equipment. For this reason the hourly or dayly rates of the construction equipment are calculated, including the operators and its helpers wages. This result is shown in Table 8–4 for schedule of dayworks.

8.2.5 Result of Unit Cost Analysis

The result of unit cost analysis is shown in Table 8–5 and 8–6. This unit cost consists of base cost, overhead, profits of contractor and tax.

Table 8—4(A) SCHEDULE OF DAYWORKS

Equipment		Bulldozer					Dump Truck		Excavator (Back and Forth)		
Type		12 t	21 t	21 t (With ripper)	39 t	39 t (With ripper)	8 t	11 t	0.35m³	0.6m³	1.0m³
Engine power (PS)		106	203	203	339	339	244	312	84	102	175
Weight (T)		12.0	20.9	23.8	38.7	43.2	7.1	9.3	10.9	18.5	71.7
Lifetime (Years)		6	6	6	6	6	4	4	5	5	5
Workingtime/Year (Hours)		1,100	1,100	1,100	1,200	1,200	1,600	1,700	1,200	1,300	1,300
Workingday/Year (Days)		165	165	165	180	180	215	225	180	195	195
Depreciation rate/Hour ($\times 10^{-6}$)		352	352	359	329	357	328	309	325	308	308
Purchase price (1,000 K.Shs.)		912	1,671	1,879	3,356	3,660	327	521	694	1,069	1,937
F.C. (K.Shs.)	Depreciation	321.0	588.2	674.6	1,104.1	1,306.6	122.0	161.0	225.6	329.3	596.6
	Consumption	–	–	11.6	–	14.0	19.1	25.0	–	–	–
	Fuel	29.0	55.5	64.7	92.6	108.1	21.6	27.6	23.6	28.7	49.1
	Lubricants	5.8	11.1	12.9	18.5	21.6	4.3	5.5	4.7	5.8	9.8
	Total/Hour	355.8	654.8	763.8	1,215.2	1,450.3	167.0	219.1	253.9	363.8	655.6
L.C. (K.Shs.)	Fuel	34.0	65.1	76.0	108.8	126.8	25.4	32.5	27.7	33.6	57.7
	Lubricants	6.8	13.0	15.2	21.8	25.4	5.1	6.5	5.6	6.7	11.6
	Wages	22.4	22.4	22.4	22.4	22.4	10.7	10.7	22.4	22.4	22.4
	Total/Hour	63.2	100.5	113.6	153.0	174.6	41.2	49.7	55.7	62.7	91.7
Total Currency (K.Shs.)/Hour		419.0	753.3	877.4	1,368.2	1,624.9	208.2	268.8	309.6	426.5	747.2

Equipment		Excavator (Clamsheel Bucket)			Macadam Roller		Mortorized Grade		Tyre Roller		Road Stabilizer
Type		0.6m³	1.0m³	2.0m³	10.5 t	12 t	3.1 m	3.7 m	7.85 t	15.5 t	2.3m x 0.4m
Engine power (PS)		105	170	217	58	89	110	126	44	85	185
Weight (T)		29.9	33.1	69.5	8.1	10.1	9.3	11.5	6.1	8.6	8.5
Lifetime (Years)		7	7	7	7	7	6	6	7	7	5
Workingtime/Year (Hours)		1,000	1,200	1,300	900	900	1,000	1,000	900	900	600
Workingday/Year (Days)		150	150	195	150	150	150	150	150	150	120
Depreciation rate/Hour ($\times 10^{-6}$)		334	315	279	348	348	353	353	348	348	667
Purchase price (1,000 K.Shs.)		1,711	1,938	3,566	384	412	700	780	364	466	1,237
F.C. (K.Shs.)	Depreciation	571.5	610.5	994.9	133.6	143.4	247.1	275.3	126.7	162.2	825.1
	Consumption	–	–	–	–	–	12.6	14.8	–	–	165.2
	Fuel	29.5	47.7	61.0	9.1	13.9	18.7	21.3	6.3	12.1	36.5
	Lubricants	5.9	9.6	12.2	1.8	2.8	3.7	4.3	1.2	2.4	7.3
	Total/Hour	606.9	667.8	1,068.1	144.5	160.1	282.1	315.7	134.2	176.7	1,034.1
L.C. (K.Shs.)	Fuel	34.6	56.1	71.5	10.7	16.4	21.8	25.1	7.3	14.1	42.8
	Lubricants	6.9	11.2	14.3	2.2	3.3	4.4	5.0	1.5	2.8	8.6
	Wages	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	26.9
	Total/Hour	63.9	89.7	108.2	35.3	42.1	48.6	52.5	31.2	39.3	78.3
Total Currency (K.Shs.)/Hour		670.8	757.5	1,176.3	179.8	202.2	330.7	368.2	165.4	216.0	1,112.4

Table 8–4(B) SCHEDULE OF DAYWORKS

Equipment		Vibrating Roller	Road Sprinkler	Tyredozer	Dis-tributor	Dump Truck		Cable Crane	Truck Crane		
Type		2.5 t	1,800 ℓ	19 t	300 ℓ	8 t	11 t	5 t, 500 m	4.8 t	20 t	35 t
Engine power (PS)		12.4	85	200	10.7	244	312	22 kW	125	175	185
Weight (T)		2.9	4.4	19.2	5.4	7.1	9.3	91.2	7.7	19.9	36.2
Lifetime (Years)		6	5	5	6	4	4	7	6	6	6
Workingtime/Year (Hours)		750	1,200	1,200	600	1,600	1,700	–	1,000	1,100	1,100
Workingday/Day (Days)		150	195	170	150	215	225	–	145	160	160
Depreciation rate/Hour ($x10^{-6}$)		471	333	358	533	328	309	154.3	303	276	276
Purchase price (1,000 K.Shs.)		182	143	1,409	540	372	521	4,610	550	1,323	2,347
F.C. (K.Shs.)	Depreciation	85.7	47.6	504.4	287.8	122.0	161.0	711.3	166.7	365.1	647.8
	Consumption	–	–	243	–	19.1	25.0	–	–	–	–
	Fuel	3.4	7.7	54.6	2.3	21.6	27.6	–	12.7	17.7	18.7
	Lubricants	0.7	1.6	10.9	0.5	4.3	5.5	–	2.5	3.5	3.7
	Total/Hour	89.8	56.9	594.2	290.6	167.0	219.1	711.3	181.9	386.3	670.2
L.C. (K.Shs.)	Fuel	4.0	9.1	64.2	2.8	25.4	32.5	–	14.8	20.8	22.0
	Lubricants	0.8	1.8	12.8	0.5	5.1	6.5	–	3.0	4.2	4.4
	Wages	9.1	10.7	22.4	16.0	10.7	10.7	22.4	22.4	22.4	22.4
	Total/Hour	13.9	21.6	99.4	19.3	41.2	49.7	22.4	40.2	47.4	48.8
Total Currency (K.Shs.)/Hour		103.7	78.5	693.6	309.9	208.2	268.8	733.7	222.1	433.7	719.0

Equipment		Crawler Crane	Truck Mixer		Concrete Pump-Car	Truck
Type		80 t	3.2 m^3	4.3 m^3	80 m^3/h	3.5 t
Engine power (PS)		180	195	270	190	98
Weight (T)		49.6	7.5	10.0	12.6	2.5
Lifetime (Years)		6	5	5	4	4
Workingtime/Year (Hours)		1,200	1,000	1,000	1,300	1,150
Workingday/Year (Days)		180	165	165	190	165
Depreciation rate/Hour ($x10^{-6}$)		301	360	360	419	435
Purchase price (1,000 K.Shs.)		3,281	468	578	1,711	109
F.C. (K.Shs.)	Depreciation	987.6	168.5	208.1	716.9	47.4
	Consumption	–	–	–	–	–
	Fuel	26.4	19.2	26.6	34.1	8.9
	Lubricants	5.3	3.8	5.3	6.8	1.8
	Total/Hour	1,019.3	191.5	240.0	757.8	58.1
L.C. (K.Shs.)	Fuel	31.0	22.6	31.3	40.1	10.5
	Lubricants	6.2	4.5	6.3	4.8	2.1
	Wages	22.4	10.7	10.7	22.4	10.7
	Total/Hour	59.6	37.8	48.3	67.3	23.3
Total Currency (K.Shs.)/Hour		1,078.9	229.3	288.3	825.1	81.4

Table 8–4(C) SCHEDULE OF DAYWORKS

Equipment		Dynamo Electric Machine					Asphalt Sprayer	Tamper
Type		75kVA	100kVA	125kVA	175kVA	300kVA	200ℓ	80 kg
Engine power (PS)		93	120	153	210	370	4	4
Weight (T)		2.2	2.5	2.5	4.0	6.1	0.15	0.08
Lifetime (Years)		6	6	6	7	7	3	3
Workingtime/Year (Hours)		–	–	–	–	–	–	–
Workingday/Year (Days)		140	140	140	140	140	125	140
Depreciation rate/Day (x 10^{-6})		1,905	1,905	1,905	1,735	1,735	620	4,167
Purchase price (1,000 K.Shs.)		192	237	287	389	679	164	14.5
F.C. (K.Shs.)	Depreciation	365.8	451.5	546.7	674.9	1,178.1	101.7	60.4
	Consumption	–	–	–	–	–	–	–
	Fuel	206.4	266.4	339.7	466.2	821.4	12.4	12.4
	Lubricants	41.3	53.3	67.9	93.2	164.3	2.5	2.5
	Total/Day	613.5	771.2	954.3	1,234.3	2,163.8	116.6	75.3
L.C. (K.Shs.)	Fuel	242.4	312.8	398.7	547.3	964.3	14.6	14.6
	Lubricants	48.5	62.5	79.8	109.5	192.8	2.9	2.9
	Wages	–	–	–	–	–	–	45.5
	Total/Day	290.9	375.3	478.5	656.8	1,157.1	17.5	63.0
Total Currency (K.Shs.)/Day		904.4	1,146.5	1,432.8	1,891.1	3,320.9	134.1	138.3

Table 8–5 UNIT COST FOR BRIDGE

(In K.Shs.)

Item		Sub-Item	Class	Unit	Unit Cost		
					F.C.	L.C.	Total
Superstructure Works	Main Girder	Concrete	σ_{ck} = 350 kg/cm^2	M^3	880	820	1,700
			σ_{ck} = 210 kg/cm^2	M^3	700	750	1,450
		Form	Steel	M^2	230	55	285
		P.C. Rod	SBPD 95/100 SBPR 95/120	T	59,000	6,500	65,500
		P.C. Cable	SWPR 7A	T	100,000	11,000	111,000
		Reinforcement	SD30	T	12,500	1,800	14,300
	Shoe	Gum Pad	700 x 800 x 20	No.	13,800	1,200	15,000
		P.C. Cable	SWPR 7A	T	100,000	11,000	111,000
		Hinge	Cast Steel	T	97,000	8,000	105,000
	Timbering	Girder	Crib Type	M^3	200	50	250
			Hanging Type	No.	12,900	1,100	14,000
	Stayed Cable	P.C. Cable	SWPR 7A	T	133,000	13,000	146,000
	Erection & Equipment	–	–	Lump Sum	14,500,000	1,700,000	16,200,000
Tower Works	Tower	Concrete	σ_{ck} = 350 kg/cm^2	M^3	880	820	1,700
			σ_{ck} = 240 kg/cm^2	M^3	770	780	1,550
		Form	Steel	M^2	255	60	315
		Reinforcement	SD30	T	12,500	1,800	14,300
	Timbering	–	Bracket Type	Lump Sum	230,000	20,000	250,000
	Erection & Equipment	–	–	Lump Sum	4,900,000	600,000	5,500,000
Abutment & Foundation Works	Abutment & Footing	Concrete	σ_{ck} = 240 kg/cm^2	M^3	770	780	1,550
		Form	Steel	M^2	290	80	370
		Reinforcement	SD30	T	12,500	1,800	14,300
		Excavating & Filling	Soil	M^3	245	55	300
	Pile	Cast-in-Place Pile	R.C.D. ϕ 2.0 m	M	41,000	8,000	49,000
Surface & Facility Works	Cast-in-Place Kerb	Concrete	σ_{ck} = 240 kg/cm^2	M^3	770	780	1,550
		Form	Steel	M^2	235	55	290
		Reinforcement	SD30	T	12,500	1,800	14,300
	Pavement	Carriageway	Asphalt t = 50 mm	M^2	102	80	182
		Concrete	σ_{ck} = 210 kg/cm^2	M^3	700	750	1,450
		Pedestrianway	Asphalt t=30 mm	M^2	73	59	132
		Crush Stone	–	M^3	30	140	170
	Kerb	Concrete	–	M	20	80	100
	Handrail	Alumi.	H = 1.2 m	M	3,100	700	3,800
	Drain	Cast Iron	–	No.	1,850	150	2,000
	Lighting	Alumi. Pole	H = 10 m	No.	35,000	8,000	43,000
	Expansion Joint	Rubber		M	21,000	3,000	24,000
Temporary & Other Works		–	–	Lump Sum	66,100,000	13,900,000	80,000,000

Table 8-6 UNIT COST FOR APPROACH ROAD

(In K.Shs.)

Item	Sub-Item	Class	Unit	Unit Cost		
				F.C.	L.C.	Total
Site Clearance	Common Field		Ha	8,220	2,180	10,400
	Dense Bush		Ha	11,470	3,030	14,500
Demolish	House		No.	380	110	490
	Fence		M	15	4	19
Strip Top Soil	Common	t = 0.2 m	M^3	20	5	25
Disposal	Common	10 km	M^3	23	7	30
Excavation	Soil	Class 2	M^3	28	8	36
Embankment	Soil	Class 2	M^3	55	14	69
Slope Protection	–	Grass	M^2	2	8	10
Drainage	Earth Drain	0.5 m x 0.5 m	M	20	5	25
	Pipe Culvert	ϕ 600 m	M	250	350	600
Pavement	Carriageway	t = 475 mm	M^2	151	199	350
	Shoulder	t = 300 mm	M^2	108	92	200
	Pedestrianway	t = 50 mm	M^2	41	59	100
	Over Lay	t = 50 mm	M^2	78	62	140
Facility	Kerb	Concrete	M	19	81	100
	Guard Rail	Steel	M	342	58	400
	Lane Marking	W = 15 cm	M	5	20	25
	Traffic Sign	2 m^2 Alumi.	No.	22,900	2,100	25,000
	Lighting	H = 10 m	No.	14,200	1,800	16,000
Access Road	Asphalt Pavement	W = 5.0 m	M	900	1,100	2,000
	Soil	W = 4.0 m	M	90	110	200
Removal	Telephone Line	–	Lump Sum	125,000	35,000	160,000
	Power Line	Low Voltage	M	50	80	130

8.3 CONSTRUCTION QUANTITIES ESTIMATE

8.3.1 General

The construction quantities are divided into two working packages the bridge and the approach road.

The bridge construction quantities are estimated based on the preliminary design that has looked into the structural analysis, design of main section, scheme of construction and others. The approach road construction quantities are estimated based on the preliminary design which was designed on a map scale 1:500. The map was drawn up for this study.

8.3.2 Construction Quantities

The construction quantities for the bridge and the approach road are shown in Table 8–7 and 8–8.

8.4 CONSTRUCTION COST ESTIMATE

8.4.1 Bridge Construction Cost

The bridge construction cost is estimated employing the above mentioned unit cost and quantities.

This cost consists of five parts namely superstructure works, tower works, abutment and foundation works, surface and facility works and temporary and other works. The bridge construction cost is shown in Table 8–9.

8.4.2 Approach Road Construction Cost

The approach road construction cost is shown in Table 8–10.

This cost includes costs for earthwork, pavement, drainage, facilities and access roads.

Table 8–7 BRIDGE CONSTRUCTION QUANTITIES

	Item	Sub-Item	Class	Unit	Quantity
Superstructure Works	Main Girder	Concrete	σ_{ck} = 350 kg/cm^2	M^3	4,042
			σ_{ck} = 210 kg/cm^2	M^3	1,838
		Form	Steel	M^2	13,908
		P.C. Rod	SBPD 95/100 SBPR 95/120	T	171
		P.C. Cable	SWPR 7A	T	62
		Reinforcement	SD30	T	405
	Shoe	Gum Pad	700 x 800 x 70	No.	4
		P.C. Cable	SWPR 7A	T	0.9
		Hinge	Cast Steel	T	4
	Timbering	Girder	Crib Type	M^3	400
			Hanging Type	No.	1
	Stayed Cable	P.C. Cable	SWPR 7A	T	254
	Erection & Equipment	–	–	Lump Sum	1
Tower Works	Tower	Concrete	σ_{ck} = 350 kg/cm^2	M^3	1,623
			σ_{ck} = 240 kg/cm	M^3	1,440
		Form	Steel	M^2	4,018
		Reinforcement	SD30	T	331
	Timbering	–	–	Lump Sum	1
	Erection & Equipment	–	–	Lump Sum	1
Abutment & Foundation Works	Abutment & Footing	Concrete	σ_{ck} = 240 kg/cm^2	M^3	4,116
		Form	Steel	M^2	2,278
		Reinforcement	SD30	T	336
		Excavating & Filling	Soil	M^3	2,872
	Pile	Cast-in-Place Pile	R.C.D. ϕ2.0 m	M	1,188
Surface & Facility Works	Cast-in-Place Kerb	Concrete	σ_{ck} = 240 kg/cm^2	M^3	137
		Form	Steel	M^2	683
		Reinforcement	SD30	T	6
	Pavement	Carriageway	Asphalt t = 50 mm	M^2	3,570
		Concrete	σ_{ck} = 210 kg/cm^2	M^3	95
		Pedestrianway	Asphalt t = 30 mm	M^2	1,512
		Crush Stone	–	M^3	390
	Kerb	Concrete	–	M	840
	Handrail	Alumi.	H = 1.2 m	M	840
	Drain	Cast Iron	–	No.	44
	Lighting	Alumi. Pole	H = 10 m	No.	15
	Expansion Joint	Rubber	–	M	37.5
Temporary & Other Works		–	–	Lump Sum	1

Table 8–8 APPROACH ROAD QUANTITIES

Item	Sub-Item	Class	Unit	Quantity
Site Clearance	Common Field	–	Ha	1.70
	Dense Bush	–	Ha	5.75
Demolish	House	–	No.	37
	Fence	–	M	1,070
Strip Top Soil	Common	t = 0.2 m	M^3	14,885
Disposal	Common	10 km	M^3	14,885
Excavation	Soil	Class 2	M^3	660
Embankment	Soil	Class 2	M^3	87,417
Slope Protection	–	Grass	M^2	19,810
Drainage	Earth Drain	0.5 m x 0.5 m	M	7,392
	Pipe Culvert	ϕ 600 mm	M	343
Pavement	Carriageway	t = 475 mm	M^2	29,389
	Shoulder	t = 300 mm	M^2	16,262
	Pedestrianway	t = 50 mm	M^2	10,120
	Over Lay	t = 50 mm	M^2	1,300
Facility	Kerb	Concrete	M	7,392
	Guard Rail	Steel	M	352
	Lane Marking	W = 15 cm	M	7,392
	Traffic Sign	2 m^2 Alumi.	No.	2
	Lighting	H = 10 m	No.	30
Access Road	Asphalt Pavement	W = 5.0 m	M	110
	Soil	W = 4.0 m	M	300
Removal	Telephone Line	–	M	500
	Power Line	Low Voltage	M	760

Table 8–9 BRIDGE CONSTRUCTION COST

(In 1,000 K.Shs.)

	Item	Sub-Item	Class	F.C.	L.C.	Total
Superstructure Works	Main Girder	Concrete	σ_{ck} = 350 kg/cm^2	3,557	3,314	6,871
			σ_{ck} = 210 kg/cm^2	1,287	1,378	2,665
		Form	Steel	3,199	765	3,964
		P.C. Rod	SBPD 95/100 SBPR 95/120	10,089	1,112	11,201
		P.C. Cable	SWPR 7A	6,200	682	6,882
		Reinforcement	SD30	5,063	729	5,792
	Shoe	Gum Pad	700 x 800 x 70	55	5	60
		P.C. Cable	SWPR 7A	90	10	100
		Hinge	Cast Steel	388	32	420
	Timbering	Girder	Crib Type	80	20	100
			Hanging Type	13	1	14
	Stayed Cable	P.C. Cable	SWPR 7A	33,782	3,302	37,084
	Erection & Equipment	–	–	14,500	1,700	16,200
Tower Works	Tower	Concrete	σ_{ck} = 350 kg/cm^2	1,428	1,331	2,759
			σ_{ck} = 240 kg/cm^2	1,109	1,123	2,232
		Form	Steel	1,024	241	1,265
		Reinforcement	SD30	4,137	596	4,733
	Timbering	–	Bracket Type	230	20	250
	Erection & Equipment	–	–	4,900	600	5,500
Abutment & Foundation Works	Abutment & Footing	Concrete	σ_{ck} = 240 kg/cm^2	3,169	3,210	6,379
		Form	Steel	661	182	843
		Reinforcement	SD30	4,200	605	4,805
		Excavating & Filling	Soil	704	158	862
	Pile	Cast-in-Place Pile	R.C.D. ϕ 2.0 m	48,708	9,504	58,212
Surface & Facility Works	Cast-in-Place Kerb	Concrete	σ_{ck} = 240 kg/cm^2	105	107	212
		Form	Steel	161	38	199
		Reinforcement	SD30	75	11	86
	Pavement	Carriageway	Asphalt t = 50 mm	364	286	650
		Concrete	σ_{ck} = 210 kg/cm^2	67	71	138
		Pedestrianway	Asphalt t = 30 mm	110	89	199
		Crush Stone	–	12	55	67
	Kerb	Concrete	–	17	67	84
	Handrail	Alumi.	H = 1.2 m	2,604	588	3,192
	Drain	Cast Iron	–	81	7	88
	Lighting	Alumi. Pole	H = 10 m	525	120	645
	Expansion Joint	Rubber		788	112	900
Temporary & Other Works		–	–	66,100	13,900	80,000
			Total	219,582	46,071	265,653

Table 8–10 APPROACH ROAD CONSTRUCTION COST

(In 1,000 K.Shs.)

Item	Sub-Item	Class	F.C.	L.C.	Total
Site Clearance	Common Field	–	14	4	18
	Dense Bush	–	66	17	83
Demolish	House	–	14	4	18
	Fence	–	16	4	20
Strip Top Soil	Common	t = 0.2 m	298	74	372
Disposal	Common	10 km	342	104	446
Excavation	Soil	Class 2	18	5	23
Embankment	Soil	Class 2	4,808	1,224	6,032
Slope Protection	–	Grass	40	158	198
Drainage	Earth Drain	0.5 m x 0.5 m	148	37	185
	Pipe Culvert	ϕ600 mm	86	120	206
Pavement	Carriage Way	t = 475 mm	4,438	5,848	10,286
	Shoulder	t = 300 mm	1,756	1,496	3,252
	Pedestrianway	t = 50 mm	415	597	1,012
	Over Lay	t = 50 mm	101	81	182
Facility	Kerb	Concrete	140	599	739
	Guard Rail	Steel	120	21	141
	Lane Marking	W = 15 cm	37	148	185
	Traffic Sign	$2\ m^2$ Alumi.	46	4	50
	Lighting	H = 10 m	426	54	480
Access Road	Asphalt Pavement	W = 5.0 m	99	121	220
	Soil	W = 4.0 m	27	33	60
Removal	Telephone Line	–	125	35	160
	Power Line	Low Voltage	38	61	99
		Total	13,618	10,849	24,467

8.5 LAND ACQUISITION COST AND COMPENSATION

8.5.1 Land Acquisition Cost

(1) General

The land parcels which need to be acquired for the construction of the proposed road are of the following categories;

a) Private land area

b) Creek and shore area

c) Government reservation land area

d) Existing road area

The existing road between its junction point with the propoed road and the ferry jetty is not necessary as a trunk road, after the completion of the new road. However, it will remain as a local road for the residents. Therefore, the proceeds from sale of existing road land is not calculated.

(2) Unit Cost of Land Acquisition

a) Private Land Area

The unit cost of private land acquisition is also examined in the same matter as the unit cost of construction, after discussions with the agencies concerned. The unit cost of private land acquisition is estimated at 12 K.Shs. per square meter.

b) Creek and Shore Area

The land value of the shore (above high water level) land that is utilised as workable ground with fishing ground, etc., is always considered to have some value. However, it is found that the shore area in Kilifi creek is too small to be of any use at high tide, and the space that is needed for the bridge is very small. Therefore, land acquisition cost need not be considered.

The land in the creek (below high water level) is usually assumed to be of lower value than that on the land. In this study, no land value is estimated for the land in the creek.

c) Government Reservation Land Area

For the land which has been already reserved for the new crossing, the land acquisition cost is not included in the calculation.

(3) Land Acquisition Cost

It is leaned that the road area of Kilifi side has been already reserved by the government. Accordingly, the road area of Mnarani side must be acquired for the project, and this amounts to an area of 134,340 m^2.

Land acquisition cost is obtained from the unit cost multipled by the road area as follows:

Unit Cost		Road Area		Land Acquisition Cost
12 K.Shs./m²	x	134,340 m²	=	1,612,000 K.Shs

8.5.2 Compensation

(1) General

A topographic survey is carried out and a map of 1:500 is drawn. The number of houses affected along the approach road of Kilifi Bridge are counted on this map.

(2) Compensation

The unit cost of compensation is estimated at 10,000 K.Shs. per unit and amount of compensation is estimated as follows:

Unit Cost		Number of Houses		Compensation
10,000 K.Shs	x	37	=	370,000 K.Shs.

8.6 PROJECT COST ESTIMATE

(1) Project Cost

The project cost is calculated on the basis of the financial cost which includes construction cost, engineering fee, land acquisition cost, compensation and contingency.

The engineering fee is estimated at 12% of the construction cost, and the contingency is expected at 10% of the sub-total which consists of the construction cost, engineering fee, land acquisition cost and compensation.

The project cost is as shown in Table 8–11. In this table, the price contingencies in the investment are not considered.

Table 8–11 PROJECT COST

(In 1,000 K.Shs.)

Item \ Currency	F.C.	L.C.	T.C.
Bridge	219,582	46,071	265,653
Approach Road	13,618	10,849	24,467
Construction Cost	233,200	56,920	290,120
Engineering Fee	27,851	6,963	34,814
Land Aquisition	–	1,612	1,612
Compensation	–	370	370
Sub-Total	261,051	65,865	326,916
Contingency	26,105	6,587	32,692
Total	287,156	72,452	359,608

Note: 1) The project cost is expressed in July 1983 prices.
2) The price contingency is not considered.

(2) Project Cost by Year

The detailed construction schedule has already been examined in the previous section. The result of which, together with the main working construction schedule are shown in Table 8–12.

The project cost is subdivided in accordance with the above construction schedule as shown in Table 8–13.

Table 8–12 CONSTRUCTION SCHEDULE

Working Item			1984	1985	1986	1987	1988	1989
Engineering and Evaluation								
Land Acquisition and Compensation								
Temporary & Other Works								
Bridge	Foundations	P_1						
		P_2						
	Abutments	A_1						
		A_2						
	Towers	P_1						
		P_2						
	Superstructure							
	Surface and Facilities							
Roads	Approach Roads							
	Access Roads							

Table 8–13 PROJECT COST BY YEAR

(Unit: 1,000 K.Shs., 1983 price)

			1984	1985	1986	1987	1988	1989
Bridge	F/C	219,582	0	0	33,050	49,880	86,370	50,282
	L/C	46,071	0	0	6,950	11,617	17,670	9,834
Approach Road	F/C	13,618	0	0	0	8,995	0	4,623
	L/C	10,849	0	0	0	7,130	0	3,719
Construction Cost	F/C	233,200	0	0	33,050	58,875	86,370	54,905
	L/C	56,920	0	0	6,950	18,747	17,670	13,553
	T	290,120	0	0	40,000	77,622	104,040	68,458
Engineering Fee	F/C	27,851	4,642	9,284	3,482	2,481	3,481	3,481
	L/C	6,963	1,160	2,323	870	870	870	870
Land Acquisition	F/C	0	0	0	0	0	0	0
	L/C	1,612	0	0	1,612	0	0	0
Compensation	F/C	0	0	0	0	0	0	0
	L/C	370	0	0	370	0	0	0
Contingency	F/C	26,105	464	928	3,653	6,236	8,985	5,839
	L/C	6,587	116	232	980	1,962	1,854	1,443
Total	F/C	287,156	5,106	10,212	40,185	68,592	98,836	64,225
	L/C	72,452	1,276	2,555	10,782	21,579	20,394	15,866
	T	359,608	6,382	12,767	50,967	90,171	119,230	80,091

F/C: Foreign Portion L/C: Local Portion T: Total