6.3.3 Bearing Layer of Bridge Foundation

Foundations of the bridge can be born on the Nubian formation which is the basal rock layer around the proposed project area.

According to subsurface exploration by mechanical boring, the Nubian formation is found below the alluvial layer (clay and sand) at approximately 10 meters below the ground surface. The upper part of this is decomposed and loose for several meters. It is necessary, therefore, to found the bridge on the stiff rock zone through the decomposed part.

From the borings completed by mid July, i.e. B1 to B5, it can be judged that a bearing stratum which will sustain the loadings transmitted from the upper structures lies at an approximate depth of 15.0 m below the riverbed taking into consideration the requirements for foundation design.

6.3.4 Embankment Study

Maximum embankment height affects bridge length where abutments will be planned in a flood-prone area. Therefore, consolidation settlement and circular slip failure were examined to estimate the maximum embankment height.

Details of these calculations are described in Appendix 6.3 and the results are summarized below:

(1) Consolidation Settlement

According to the boring results, clayey layer of 4 meters depth was judged as the probable settlement layer. Calculation was made by means of Osterbery's graph on the premise that the unit weight of the embankment is 1.8 t/cu.m, and the results are summarized in Table 6.3.

Table 6.3 Consolidation Settlement by Embankment Heights

Embankment Height	Settlement
6 m	40 cm
8 m	50 cm
10 m	55 cm
12 m	70 cm

Required time for 80% achievement of the above settlement is estimated at 450 days.

(2) Circular Slip Failure

Calculation was made for the following two (2) cases:

- Embankment without counter-weight embankment
- Embankment with counter-weight embankment

As a result of iteration by division method, the following is obtained when the unit weight of the embankment is 1.8 tf/cu.m and its cohesive strength is 3.0 tf/sq.m.

A) Embankment Without Counter-weight Embankment

- Maximum Embankment Height = 8.0 m

B) With Counter-weight Embankment

- Maximum Embankment Height = 10.0 m
- Counter-weight Embankment = 0.30 tf/m^2

(3) Maximum Embankment Height to Be Recommended

Based on the discussions above in (1) and (2), the following is concluded as the recommendation for maximum embankment heights for the proposed road.

Without countermeasures against circular slips

Maximum Embankment Height = 8.0 m

With counter weight embankment

Maximum Embankment Height = 10.0 m

Although the embankment height might be theoretically possible up to 12 meters or more if the counter-weight is increased, such a high embankment is impractical since consolidation settlement of more than 70 cm would occur in two (2) years.

6.4 HYDROLOGY

6.4.1 Methodology

Hydrology covers the following study items so as to provide meteo-hydrological and river hydraulic design conditions for the New White Nile Bridge and related structures:

a) Understanding of meteo-hydrological situation at the project site

b) Provision of high and low water levels and their flow velocities

c) Estimation of probable wind velocity

d) Provision of Design wave height induced by probable wind

e) Estimation of local scouring depth around bridge pier

f) Estimation of probable rainfall intensity

The data collection and establishment of procedures are made to achieve the above purpose. Their details are mentioned in the following:

(1) Available Data

A) Meteorological Data

Meteorological observation has been carried out by the Meteorological Department, Ministry of Defense, the Republic of the Sudan. In Khartoum City, there are two observatories, namely: 1) Khartoum and 2) Shambat.

Khartoum observatory is installed at the international airport and its observation was started at the beginning of the 20th century. Shambat observatory located in Khartoum North was constructed in 1937.

In this study, meteorological data recorded at Khartoum observatory are collected because its observation period is longer than Shambat observatory.

The following data relevant to the planning and design of the New White Nile Bridge and its related facilities were collected:

a) Air temperature
b) Relative humidity
c) Evaporation
d) Wind velocity and direction
e) Rainfall

Availability of the above data are listed in Appendix 6.4 (1).

B) Hydrological Data

There are four water level gauging stations around the project sites, namely: 1) Tamaniat on the River Nile, 2) Khartoum on the Blue Nile, 3) Mogran and 4) Gordon's Tree on the White Nile. Furthermore, Gebel Aulia dam was constructed in 1937 and hydrological observation has been carried out since its completion. Location and installed year are as below:

Table 6.4 Water Level Gauging Stations

Name of Station	Installed Year	Zero of Gauge (RL.m)	Distance from Confluence of the White and Blue Nile Rivers
a) River Nile			
Tamaniat	1912	361.73	40 km downstream
b) Blue Nile			
Khartoum	1904	363.00	4 km upstream
c) White Nile			
Mogran	1915	362.70	10 m upstream
Gordon's Tree	1913	363.35	7 km upstream
Gebel Aulia dam	1935		40 km upstream

Source : Ministry of Irrigation

These stations are operated and maintained by the Ministry of Irrigation, the Republic of the Sudan and the Egyptian Irrigation Department. Hydrological data such as water level, discharge and its measurement are collected so as to understand the river flow characteristics of the White Nile.

The inventory of the collected data is given in Appendix 6.6.

C) Data Relevant to the White Nile

In order to clarify historical changes of the river course of the White Nile, the aerophotographs shot in 1952, 1964, 1965, 1984 and 1988 were collected from the Survey Department, Ministry of Defense.

(2) General Procedure

In the study, the following procedures were established to understand the meteo-hydrological conditions through site reconnaissance, data collection and compilation:

A) Provision of High and Low Water Levels and Their Flow Velocity

Water level gauging stations provided in Khartoum city have long-term observation periods of more than 70 years. Therefore, design water levels of the neighboring bridges, the existing White Nile Bridge, Shambat Bridge, Blue Nile Bridge and Burri Bridge, are on the basis of the recorded highest water level at each site.

In this study, the annual highest water levels at Mogran, which has an observation period of 74 years and is located close to the project site, are available. Based on these water level records, design high water level is determined at the highest one recorded at Mogran.

The same method is applied to determination of design low water level by using the annual lowest water level at Mogran.

B) Estimation of Probable Wind Velocity

The Magnitude of the wind velocity for the design of bridge and river related structures will be determined for a 100-year wind velocity.

Wind data for 51 years from 1938 to 1988 are available in this study and it is judged that frequency analysis is applicable in estimating probable wind velocity.

C) Provision of Design Wave Height Induced by Probable Wind

The White Nile near the project site behaves as a natural retarding basin during the wet season due to backwater from the Blue Nile. Also, gusts of wind called the "Habub" blow along the White Nile from the south.

This Habub will cause wind current waves facing the bridge piers, abutments, and embankment structures crossing the White Nile.

Characteristics of wind current waves are examined by the Shallow Wave Theory based on the estimated wind velocity and topographical conditions surveyed in this study.

D) Estimation of Local Scouring Depth around Bridge Pier

Local scouring depth around bridge piers is examined by using empirical formulae taking into account scouring conditions at the existing White Nile Bridge so as to plan footing protection works.

E) Estimation of Probable Rainfall Intensity

This is necessary for determining the scale of drainage facility required for the road structure and to convert this rainfall in the catchment area of a drain to discharge.

Frequency analysis is applied to estimate probable rainfall amount and its intensity for several duration times and return periods and then a probable rainfall intensity-duration equation is established by using the result of a frequency analysis.

6.4.2 Climate

Meteorological features based on the records at Khartoum observatory are summarized as below (Refer to Appendix 6.4 (3)):

(1) Air Temperature

The monthly mean air temperature at Khartoum gradually rises from 23.2° C in January to 33.7° C in June and then falls until the next January. The annual mean temperature is 28.8° C.

The absolute maximum temperature at Khartoum over the last 90 years was 47.7° C recorded in June 1940 and the minimum was 6.0° C in January 1903.

(2) Relative Humidity

Since Khartoum city including the project site belongs to the arid zone, the annual mean relative humidity is 30 % and rather low. The yearly variation ranges from 20-30 % in the dry season from October to May up to 40-50 % in the wet season from June to September.

(3) Pan Evaporation

The monthly mean evaporation observed by pan ranges from 13 mm/day to 20 mm/day and the annual mean is 16.1 mm/day corresponding to an annual evaporation amount of 5,900 mm.

(4) Wind Velocity

The annual mean wind velocity is 4 m/sec and the yearly variation of monthly average is small. The monthly highest wind velocity for 10 minutes duration, however, ranges from 10 m/sec to more than 30 m/sec and especially in the wet season, gusts of wind having velocities of more than 20 m/sec occur frequently. An instantaneous maximum wind velocity of about 40 m/sec was recorded in August 1947.

As for wind direction, northern wind prevails in the dry season, and in the wet season it mainly blows from the south.

(5) Rainfall

Khartoum has an annual rainfall of 150 mm and more than 90 % of this falls during the four months from June to September.

The rainy days with rainfall of more than 0.1 mm are only 22 days in a year.

Khartoum City was damaged by flood in 1988 and one day rainfall amounted to 200 mm, which is the maximum ever recorded for about 40 years from 1950 to 1988, was recorded during the flood.

6.4.3 Present River Conditions

Areas considered in route alternatives in Chapter V are located between the confluence with the Blue Nile and 2 km upstream of Um Shugiera Island.

The river profile and cross-section and flood flow conditions along the above river stretch of about 6.5 km are described below:

(1) River Profile and Cross-section

A) River Profile

The White Nile flows down in the river channel having a gentle riverbed slope of 1:10,000 between Gebel Aulia dam and Mogran gauge site. According to the river sounding survey carried out by the Study Team, however, the slope rapidly changes to 1:400 near the project site.

In the river reach considered in the route study, there are two islands, namely Wad Dakien Island at the existing White Nile Bridge and Um Shugiera Island located at 700 m upstream from Wad Dakien Island. These are found in the dry season and under water during the wet season.

B) River Cross-section

The White Nile has a compound river section which consists of low and high water channels and has a width in the range of 700 to 4,000 m.

The low water channel is divided into two streams by the Wad Dakien and Um Shugiera islands in the dry season. The river widths of left and right side streams at Wad Dakien Island are about 80 m and 130 m and those at Um Shugiera Island are about 120 m and 500 m respectively. The river channel between the two islands has a single low water section of width of 500 m. The water depth is 4-5 m at the Wad Dakien Island section and about 2 m at other sections during the dry season.

The high water channel, which is defined as the area inundated during the wet season every year, has an extensive width ranging from 1.5 to 2.0 km on both the left and right sides.

(2) River Related Structures around the Existing White Nile Bridge

There are several river related structures along the river stretch up to the upstream end of Um Shugiera Island. These are 1) the existing bridge piers, 2) protection work for embankment slopes of the approach roads to the bridge, 3) small scale movable pumps for irrigation with capacities generally less than 50 liters/minute, and 4) a drainage pump utilized in the wet season.

The present conditions of facilities related to this project are described below:

A) Bank Slope Protection Work

Stone masonry with mortar riprap is provided for protecting the embankment slope of 5 m height and it has passed about 70 years after completion of the work. This facility is in good condition generally, but the last flood in 1988 left several partial scars due to the collision of a ship.

B) Bridge Pier and Abutment Structures

Occurrence of local scouring around piers in the low water channel was reported in "Assessment, Inspection and Rehabilitation of The White Nile Khartoum-Omdurman Bridge". Therefore, the Study Team took measurements using an echosounder.

According to the results of the river cross-section survey at the existing bridge site, the maximum scouring depth was measured at about 3 m around the pier in the low water channel.

Other piers and abutments on the high water channel and Wad Dakien Island do not cause scouring around piers and erosion at pile caps, since these are inundated in remarkably still river water during the wet season.

(3) River Flow Condition

A) Annual Fluctuation of River Flow Discharge

Monthly discharges at Gebel Aulia dam in the White Nile and at Khartoum water level gauging station in the Blue Nile are shown in Table 6.5:

Table 6.5 Monthly Discharges at Khartoum and Gebel Aulia Dam

Month	Blue Nile at Khartoum (cu.m/sec)	White Nile at G/A dam (cu.m/sec)	Khartoum/ G/A dam
January	259	950	0.27
February	178	845	0.21
March	148	884	0.17
April	149	1,046	0.14
May	183	881	0.21
June	419	723	0.58
July	1,956	465	4.21
August	5,787	516	11.22
September	5,208	709	7.35
October	2,708	1,031	2.63
November	909	1,043	0.87
December	442	1,031	0.43
Mean	1,539	844	1.82

Note : The above figures indicate mean monthly values for the period of 1912-1982 at Khartoum and 1943-1982 at Gebel Aulia dam.

Source : Ministry of Irrigation

In the White Nile basin, there are large lakes, Sudd extensive swamps and Gebel Aulia reservoir which are considered to have vast retardation and regulation effects against flow discharges. As indicated in the foregoing table, therefore, the flow discharges downstream from Gebel Aulia dam are rather stable. In contrast, the flow discharge of the Blue Nile fluctuates remarkably from 148 cu.m/sec to 5,787 cu.m/sec and the ratio reaches about 40.

The White and Blue Niles join at Khartoum near the existing bridge site. The flow discharge ratio changes from 0.1 in a dry season to 11 times in a wet season. The White Nile prevails through the dry season in the discharge contribution to the River Nile. The Blue Nile, however, dominates in the wet season.

B) Flood Flow Condition

In 1988, a large scale flood occurred and caused damage in the Khartoum city area. According to an aerophotograph during the flood as shown in Figure 6.6, it was found that the river water in the White Nile was disturbed to join to the River Nile by the backwater effect of the Blue Nile.

The maximum water levels recorded at the neighboring water level gauge during the flood are as follows:

Table 6.6 Flood Water Levels in 1988

Name of Gauge	Highest Water Level (RL.m)	
	Date	Water Level
a) River Nile		
Tamaniat	Aug.27	378.69
b) Blue Nile		
Khartoum	Aug.28	379.94
c) White Nile		
Mogran	Aug.28	379.75
Gordon's Tree	Aug.27	379.62
Downstream of Gebel Aulia dam	Aug.26	379.70

Source : Ministry of Irrigation

As shown in the table, the water level between Mogran and Gebel Aulia dam was almost horizontal. At this time, Gebel Aulia dam released a discharge of 800 cu.m/sec because the reservoir water level reached nearly to its high water level.

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 6.6	RIVER FLOW CONDITION DURING 1988 FLOOD
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THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 6.6	RIVER FLOW CONDITION DURING 1988 FLOOD
	JAPAN INTERNATIONAL COOPERATION AGENCY	

Flow discharge and velocity at Khartoum in the Blue Nile were estimated at about 9,000 cu.m/sec and 2 m/sec respectively.

According to the water level records at Mogran, the highest water level ever recorded is RL+379.96 m in August 1946 and the 1988 flood is the second largest flood (Refer to Appendix 6.9).

6.4.4 Hydrology

The hydraulic study was undertaken by using the surveyed river cross-sections of route B-1 in the route selection and the result is applied to all the alternatives.

(1) Provision of High and Low Water Levels and their Flow Velocity

A) Design High Water Level

There are no river improvement plans related to the White Nile near Khartoum City and flood water levels and discharges relevant to the White Nile are not yet provided.

Design water levels of the existing bridges, the White Nile Bridge, Shambat Bridge, Blue Nile Bridge and Burri Bridge, are set at the highest recorded water level at each site.

Water level records in the White, Blue and River Niles around the project site were collected and analyzed for historical annual maximum flood water levels.

Mogran gauge was installed in 1915 on the White Nile 10 m upstream from the confluence with the Blue Nile. Since the bridge sites are located between the upstream end of Um Shugiera Island and the Mogran gauge and flood water levels at the alternative bridge sites and Mogran gauge are considered to be almost the same as stated in the previous paragraph 6.4.3 (3), the recorded highest water level of RL+379.96 m at Mogran for 74 years is applied for the design high water level.

B) Design Low Water Level

Design low water level is set by converting from the minimum discharge in a dry season by means of the Manning formula since the water surface slope is estimated to be steeper than that during flood, taking into account the high flow velocity in a dry season.

Manning formula $Q = A R^{2/3} i^{1/2} / n$

Where,

- Q : Discharge in cu.m/sec
- A : Flow area in sq.m
- R : Hydraulic radius in m
- i : Average riverbed slope
- n : Roughness coefficient.

The minimum discharge at the bridge site in a dry season is considered to correspond to the outflow discharge from Gebel Aulia dam in the case of the lowest water level at Mogran.

The recorded lowest water level at Mogran and flow discharge from Gebel Aulia dam are RL+372.62 and 370 cu.m/sec in May 1945 (Refer to Appendix 6.4 (5)).

The lowest water level at the bridge sites are estimated at RL.+373.54 m based on a discharge of 370 cu.m/sec and Manning formula using the river cross-section surveyed, a roughness coefficient of 0.03 and an average riverbed slope of 1:400 around the bridge site.

C) Flow Velocity

Flow velocity at the above high water level is estimated assuming that the recorded maximum outflow discharge of 1,520 cu.m/sec in December 1988 flows down through the bridge section (Refer to Appendix 6.4 (6)).

As for the flow velocity at the low water level, the low water level of RL+373.54 m and discharge of 370 cu.m/sec are utilized for the derivation of velocity.

In converting from the water levels and discharges, the hydraulic continuity equation is applied:

Continuity equation $v = Q / A$

Where,

- v : Flow velocity in m/sec
- Q : Discharge in cu.m/sec
- A : Flow area in sq.m which is calculated from water level and river cross-section.

From the above, the flow velocities at the high and low water levels are estimated as follows (Refer to Appendix 6.4 (10)):

Table 6.7 Flow Velocities at High and Low Water Levels

Water level (RL.m)	Flow Discharge (cu.m/sec)	Area (sq.m)	Velocity (m/sec)
379.96	1,520	4,400	0.35
373.54	370	280	1.32

(2) Wind Velocity

Probable mean wind velocity for 10 minutes duration is examined by using the annual maximum wind velocity records from 1938 to 1988 by means of the Iwai Method, which was developed in Japan assuming samples belonged to log-normal distribution (Refer to Appendixes 6.12 and 6.13).

Estimated probable wind velocities for several return periods are as follows:

Table 6.8 Probable Wind Velocity

Return Period (year)	Wind Velocity (m/sec)	(mile/hour)
2	20.1	45
5	23.7	53
10	25.8	58
20	27.7	62
50	30.0	67
100	31.5	70

Source : The Study team

(3) Wind Current Wave

Pier and abutment structures of the New White Nile Bridge will be inundated by river water when the depth is estimated to be around 6 m below the high water level of RL+379.96 m.

In the study area, Habub frequently occurs during a wet season and is expected to make it necessary for wind current waves to be considered in designing the above structures.

Wave characteristics are defined by using Shallow Wave Theory, which is developed by Bretschneider, taking into account the water depth in the White Nile.

The following conditions are set in estimating wave characteristics:

a) Wind velocity and its direction are set at 31.5 m/sec, having 100-year probability, and the southern direction prevailing in the wet season in Khartoum City.

b) Fetch is set at about 11,000 m long from the bridge sites to Al Kalakla at which the river course of the White Nile meanders to the southwest direction.

c) Water Depth is set at 6 meter deep.

Bretschneider prepares the relation between wave height, wind velocity and fetch in shoal area based on observation records carried out by himself.

Based on this relation and the above-mentioned conditions, the following wave characteristics are estimated (Refer to Appendix 5.9):

a) Wave height	2.1	m
b) Wave period	4.3	seconds
c) Wave length	29.1	m

(4) Local Scouring Depth

Local scouring depth to be considered for bridge pier design is estimated by using the following empirical formulae:

a) Laursen formula

$$(D/h) = 5.5(Z/h)((Z/11.5/h+1)^{1.7}-1)$$

b) Lacey's regime formula

$$Z = 0.47\ (Q/f)^2$$

c) Neil formula

$$(Z/D) = 1.5\ (h/D)^{0.3}$$

Where,

D : Width and/or diameter of pier in m

h : Average depth in m

Z : Scouring depth from riverbed around pier in m

Q : Discharge in cu.m/sec

f : Lacey's silt factor.

Preliminarily, assuming 1) a pier width of 5 m, 2) an average water depth of 2.5 m, which is calculated from mean annual water level of RL+374.35 m from 1943 to 1982 after completion of Gebel Aulia dam and a riverbed elevation of RL+372.1 m at the proposed bridge site, 3) the recorded maximum discharge of 1,520 cu.m/sec from Gebel Aulia dam and 4) Lacey's silt factor of 0.5 taking into account the riverbed of silty clay in the White Nile, local scouring depths are calculated as follows:

Table 6.9 Estimated Local Scouring depth

Method	Local scouring depth (m)
Laursen	3.9
Lacey	6.8
Neil	6.1

Source : The Study Team

In this project, the maximum value estimated by the above formulae will be adopted as a design value to provide pier protection against scouring.

(5) Relation between Rainfall Intensity and Duration

Rainfall depth-duration analysis is carried out for establishing probable rainfall-intensity duration equation at Khartoum using the annual maximum rainfall records from 1951 to 1981 by means of the Iwai method according to the following procedure:

A) Probable Rainfall

Probable rainfall amounts for several different durations, 10, 20, 30, 60 and 120 minutes, are estimated using the annual maximum rainfall records. (Refer to Appendixes 6.4 (11) and 6.4 (12))

Table 6.10 Probable Rainfall Amount and Duration

Return Period (year)	Probable Rainfall Amount (mm) Duration (min.) 10	20	30	60	120
2	11	16	19	25	28
3	14	20	23	30	34
5	17	24	29	38	43
10	21	29	37	49	55
20	25	34	47	62	68

Source : The Study Team

B) Probable Rainfall Intensity

The previous probable rainfall amounts are converted to rainfall intensities by multiplying a ratio worked out by dividing 60 minutes by the rainfall duration.

Table 6.11 Probable Rainfall Intensity and Duration

Return Period (year)	Probable Rainfall Intensity (mm/hour) Duration (min.)				
	10	20	30	60	120
2	66	48	38	25	14
3	84	60	46	30	17
5	102	72	58	38	22
10	126	87	74	49	28
20	150	102	94	62	34

Source : The Study Team

C) Rainfall Intensity Duration Equation

This is needed to estimate a discharge for a different lag-time of the drainage area of the proposed facility. Therefore, the following equation is applied:

$$I = a / (t^n + b)$$

Where, I : Rainfall intensity in mm/hour
t : Lag-time in minute
a, b and n : Constants.

Constants a and b are derived from the Least Square Method using rainfall intensities and durations in Table 6.11 and n is defined as a value which minimizes the standard deviation between the rainfall intensities estimated by the equation and those in Table 6.11.

Finally, the constants of probable rainfall intensity-duration equations are established as follows (Refer to Appendix 6.16 to 6.18):

Table 6.12 Constants of Rainfall Intensity-Duration Equation

Return Period (year)	Constant a	b	n
2	276	0.057	0.60
3	336	0.000	0.60
5	420	-0.028	0.55
10	442	-0.014	0.55

Source : The Study Team

6.5 SOIL AND ROCK MATERIALS

6.5.1 Geological Origin

As a result of a geological survey in the field, the following three (3) soil and rock types were identified as the prospective construction materials for the Project. They are (1) Nile River Deposit, (2) Desert Soil and (3) Rock Hill.

(1) Nile River Deposit

This is mostly composed of silty clay which has a soft consistency and a low density. The very fine sand has poor grain size distribution in localized deposits of small area. Gravels are not found. Some of the clay can be used for low embankments.

(2) Desert Soil

In the desert region various kinds of soil are found. These are produced by weathering of sedimentary rock layers over a long time. Actual soil types are as follows:

a) Clay with angular gravels (weathered, decomposed sandstone and mudstone), which will be useful for embankments.

b) Gravels (weathered decomposed conglomerate), which are composed of chert and quart-origin stone, very hard and useful for concrete aggregates and pavement base-course materials.

c) Sand (weathered, decomposed sandstone), which is categorized as fine grain rich in grain size distribution and capable of being used as concrete fine aggregates.

(3) Rock Hill

Rock hills of basalt and granite are found sporadically. These rocks are sufficiently hard for use as concrete aggregates.

6.5.2 Characteristics of Each Material

The characteristics and technical conditions of each material are summarized in Table 6.13. The locations of borrow areas and quarry sites are shown in Figures 6.7 (A) and 6.7 (B).

Key points are itemized as follows:

a) Basalt is promising for concrete aggregates as well as for asphalt aggregates and riprap materials. As the volume to be produced is expected to be approximately 100,000 cubic meters, an area of 30,000 square meters (150 m x 200 m) is required.

b) The locations of a granite quarry and a borrow area for gravels are rather far from the proposed bridge site, approximately 40 km or more.

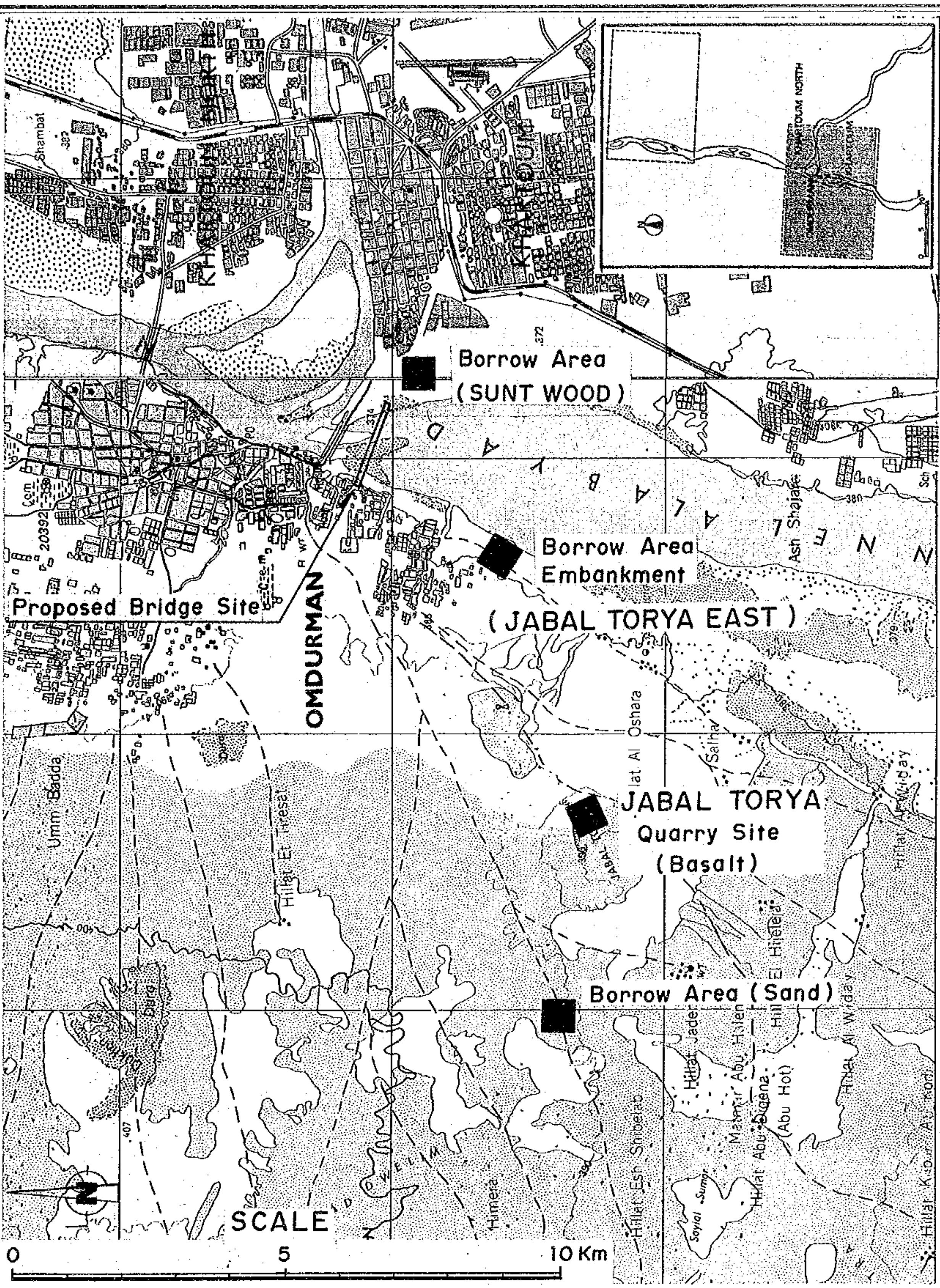
c) Promising sand material is not to be found.

d) Regarding embankment materials, comparatively good ones can be found near the proposed bridge site, 2 km upstream on the Omdurman side.

e) Clay of river deposits of the Nile will be limited to low embankment use in view of its soft condition.

TABLE 6.13 CHARACTERISTICS OF EACH MATERIAL

MATERIAL	GEOLOGY		LOCATION	DISTANCE	TECHNICAL CONDITIONS
	ORIGIN	TYPE			
1. CONCRETE AGGREGATE	a) ROCK HILL	BASALT	JABAL TORYA (MAIN QUARRY)	SOUTH-EAST 5km	GOOD ROCK. USED BY CRUSHING.
	b) ROCK HILL	GRANITE	JABAL SELETATE	NORTH 40km	FAIRLY GOOD ROCK. PROBABLY BRIDGE BROKEN IN FLAT SHAPE.
	c) DESERT SOIL	GRAVEL	JABAL SELETATE	NORTH 60km	FAIRLY GOOD PROPERTY. SIEVING AND WASHING ARE REQUIRED.
2. SAND FOR CONCRETE	a) ROCK HILL	BASALT	JABAL TORYA	SOUTH-EAST 5km	CRUSHING SAND
	b) ROCK HILL	GRANITE	JABAL SELETATE	NORTH 40km	CRUSHING SAND
	c)DESERT SOIL	WEATHERED SANDSTONE		SOUTH-EAST 15km	POOR GRAIN SIZE DISTRIBUTION SIEVIING IN-SITE AND WASHING ARE REQUIRED
3. EMBANKMENT	a)DESERT SOIL	CLAY WITH GRAVEL, WEATHERED SAND-STONE, MUDSTONE	UPSTREAM OF SITE (MAIN BORROW)	SOUTH 4km	FAIRLY GOOD. CAN BE USED FOR SUBGRADE MATERIAL.
	b) RIVER DEPOSIT	CLAY	UPSTREAM OF SITE	SOUTH 2km	SOFT AND FRAGILE. PROBLEM IN COMPACTION. USED FOR A SUPPLEMENTAL MATERIAL
4. ASPHALT AGGREGATE	ROCK HILL	BASALT	JABAL TORYA	SOUTH-EAST 5km	GOOD ROCK. PRODUCED BY CRUSHING PLANT
5. BASE COARSE FOR ROAD	ROCK HILL	BASALT	JABAL TORYA	SOUTH-EAST 5km	GOOD ROCK. PRODUCED BY CRUSHING PLANT
6. MASONRY AND RIPRAP	ROCK HILL	BASALT	LABAL TORYA	SOUTH-EAST 5km	GOOD ROCK



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 6.7 (A)	LOCATION OF QUARRY SITES AND BORROW AREAS(1)
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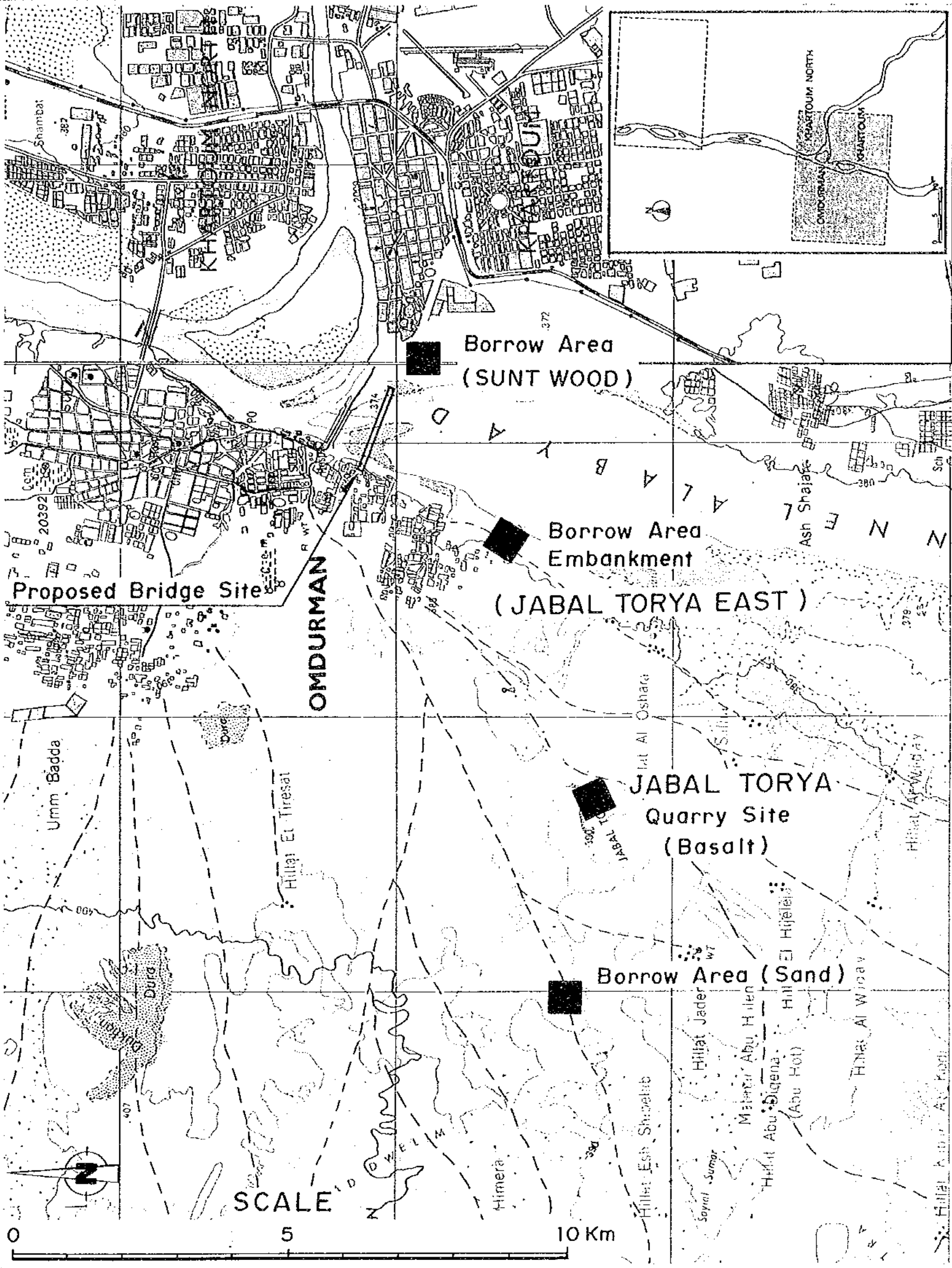
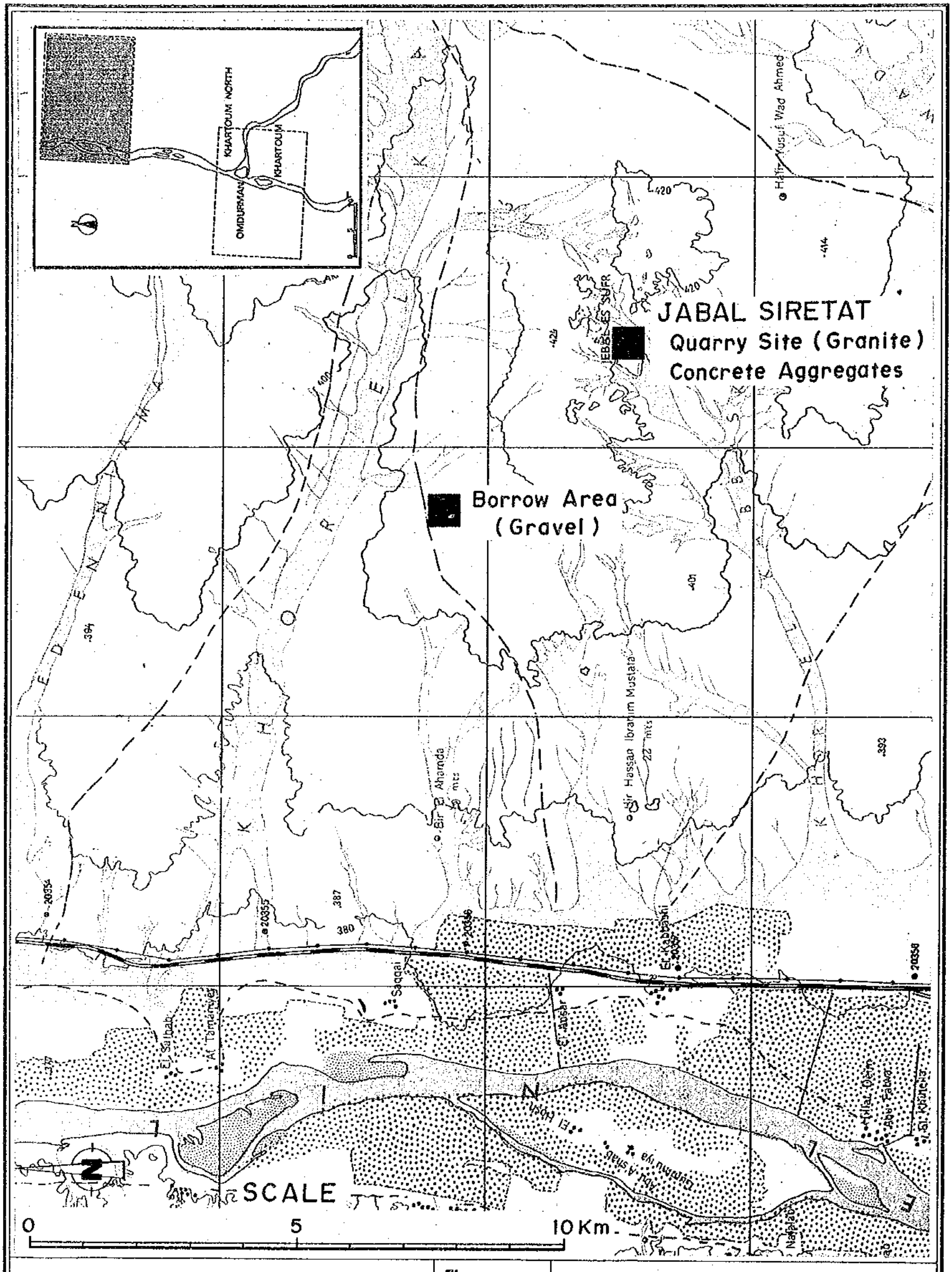


THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 6.7 (A)	LOCATION OF QUARRY SITES AND BORROW AREAS(1)
		JAPAN INTERNATIONAL COOPERATION AGENCY



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 6.7 (B)	LOCATION OF QUARRY SITES AND BORROW AREAS(2)
	JAPAN INTERNATIONAL COOPERATION AGENCY	

CHAPTER - VII

PRELIMINARY BRIDGE ENGINEERING

CHAPTER-VII PRELIMINARY BRIDGE ENGINEERING

7.1 GENERAL

In the preceding discussions in Chapter-V, it was determined that Route-B1 is the most favorable route from viewpoints of economy and engineering.

This chapter describes how a more detailed engineering study was carried out to determine the length, width, type and span arrangement of a proposed bridge.

7.2 DETERMINATION OF PLANNING CONDITIONS

7.2.1 Navigational Requirements

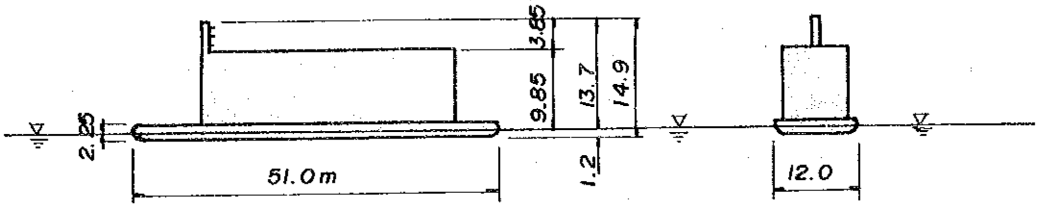
Inland water transportation from Khartoum to Juba over an approximate distance of 1,745km on the White Nile is the most prosperous line of communication in the country, especially from Kosti to Juba. Others, Karima to Dongola, Khartoum to Atabara (seasonal) on the Main Nile and Suki to Damazin (seasonal) on the Blue Nile are also utilized.

The River Transport Corporation (RTC), which is a governmental organization to operate inland water transportation, has 3 dockyards, at Khartoum, Karima and Kosti. Of these dockyards, Khartoum Dockyard is the largest and equipped with better facilities than the others. However, Khartoum Dockyard has not been fully utilized for repairing ships which are in service between Khartoum and Juba since the existing White Nile Bridge lost its mechanical function of a movable bridge (swing bridge).

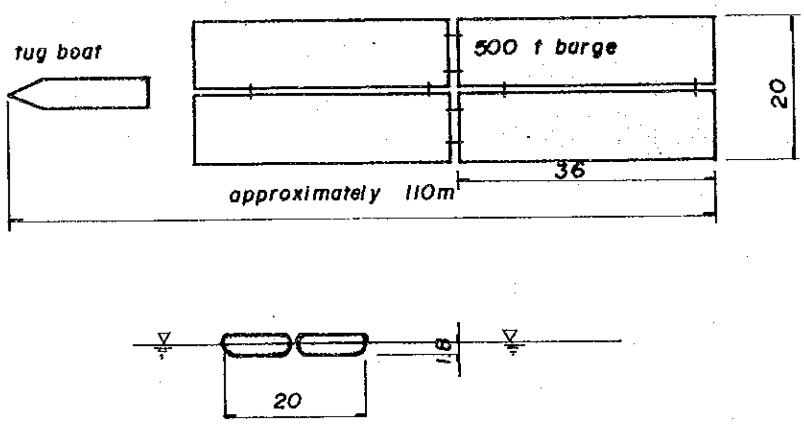
RTC owns 92 ships including steamers and tug boats and 248 barges of 78 to 500 tons displacement. The size of the biggest one is shown in Figure 7.1. The 3.85-meter high mast of this steamer can be tilted down. Therefore, it can be said that the free-board height of 10m governs the vertical requirement for navigation.

As for the horizontal requirement, the size of combined barges (maximum 4 barges) might be a dominant factor as shown in Figure 7.1. Sizes of the combined barges are 20 m in width and 1.8 m in draft, which are the maximum figures in the country. For discussing the navigational clearance to be adopted for the Study, the existence of private ships, of which sizes are very small, are not significant and can be ignored.

After many discussions between NCK, RTC and the Study Team during the months of January to February, 1989, the following were concluded:



SIZES OF MAXIMUM SHIP



CONFIGURATION OF COMBINED BARGES

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.1	MAXIMUM SHIP AND COMBINED BARGES
	JAPAN INTERNATIONAL COOPERATION AGENCY	

a) Vertical Clearance: 12m consisting of 10m free-board and 2m allowance

In this case, the elevation of water level which dominates the navigation will be at RL+379.96m recorded in the highest flood in 1946.

b) Horizontal Clearance: 45m for the following reasons:

- Maximum width of combined barges = 20m

- Clear width of swing span of the existing White Nile Bridge = 40m

7.2.2 Bridge Length

The determination of the bridge length is a major factor in the planning of a bridge since it may dominate the construction cost. The conditions to determine the bridge length, therefore, were carefully discussed with the NCK and other governmental staff concerned. After exchanging opinions with them, the following were set up as the requirements for the bridge length.

(1) River Width

It is required to keep the river width of the present low water channel (560.7 m, surveyed in June 1989) or more based on the hydrological and river engineering study discussed in Chapter-VI.

(2) Setback Distance

The location of abutments should be set back from the edge of the low water channel for the following purposes:

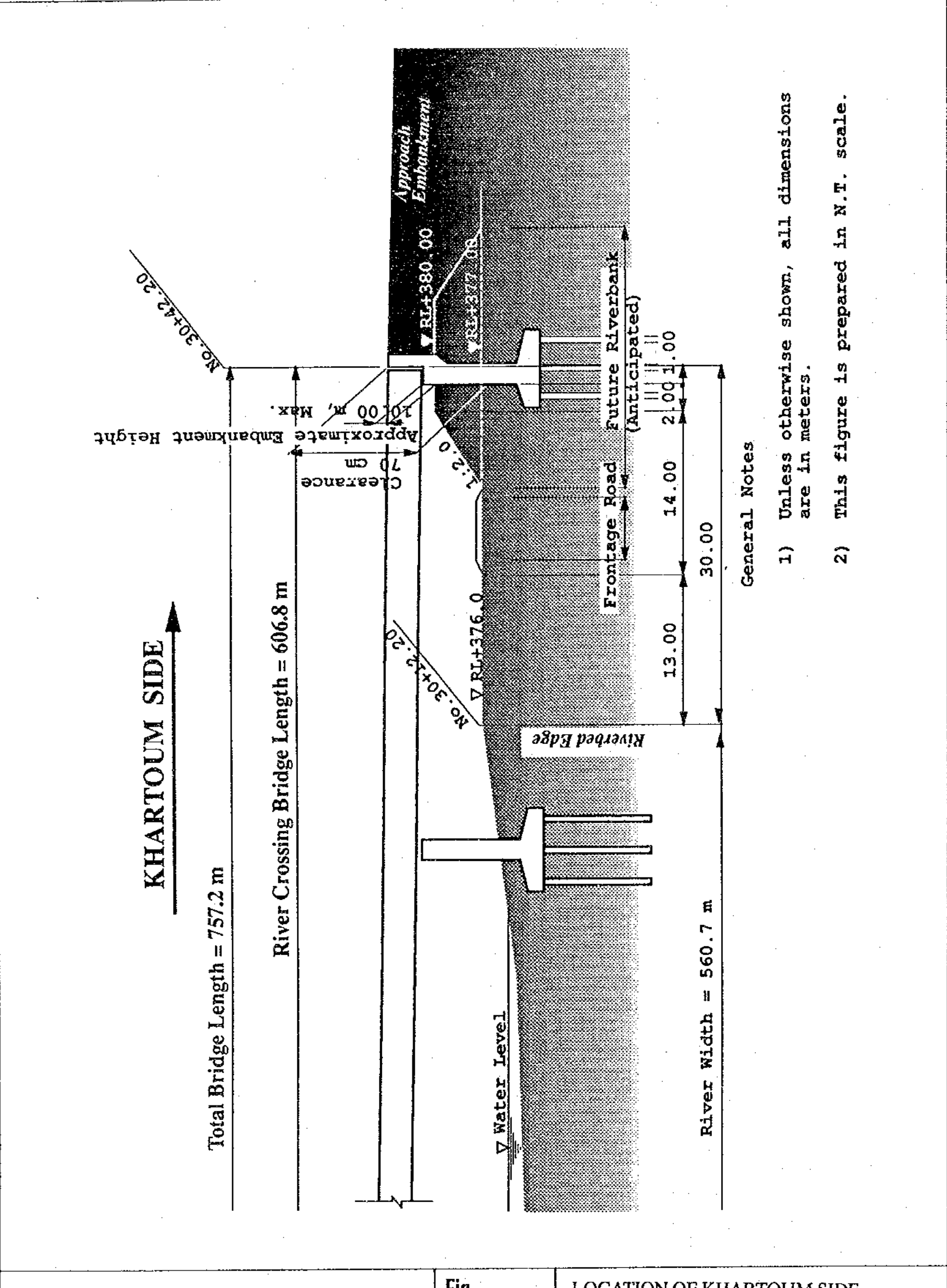
a) To leave a space which will be required for constructing the riverbank in the future

b) To provide space for a frontage road between the riverbank and the abutment, connecting up and down stream sides

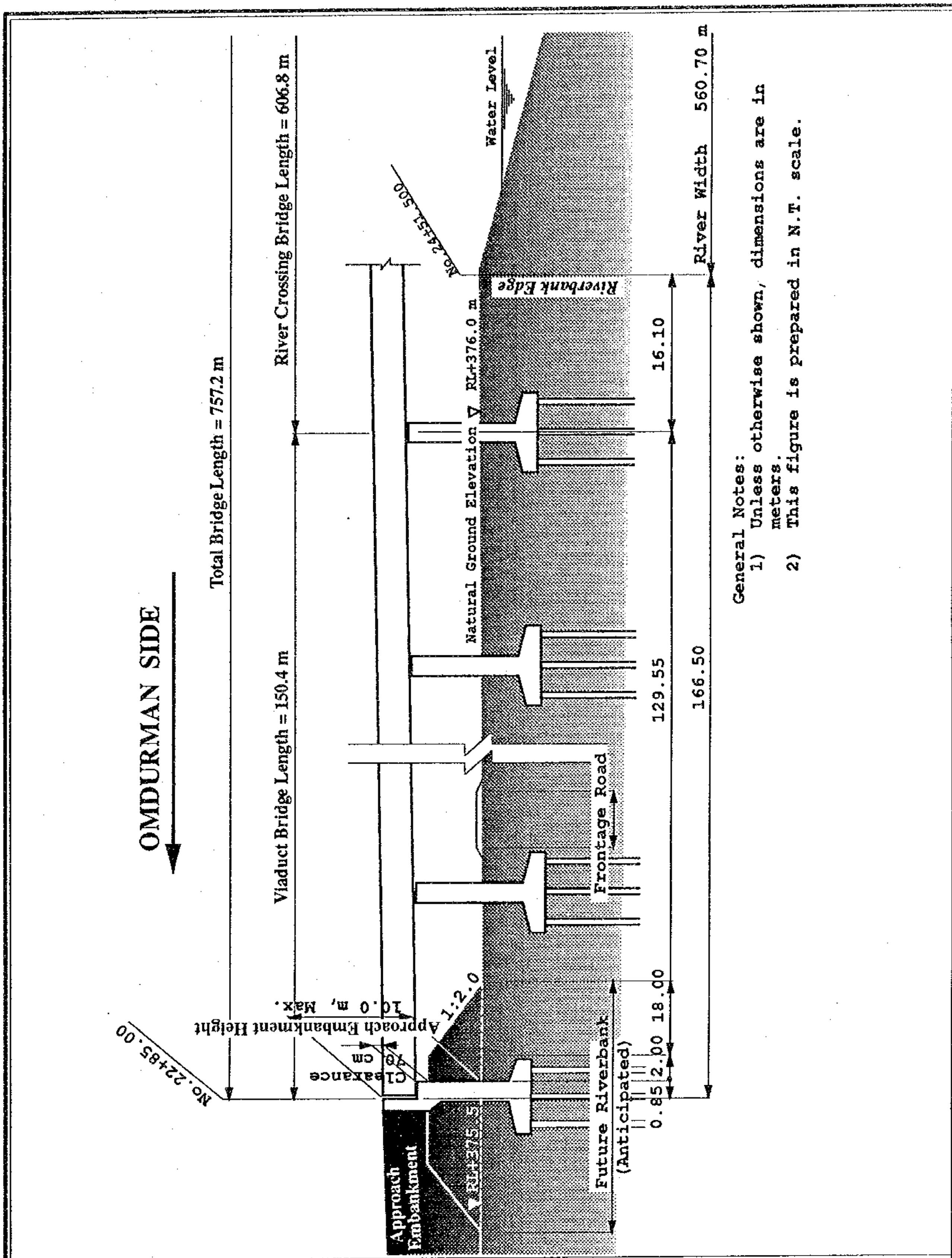
c) To maintain stability of the approach road embankment (refer to subsection 6.3.4)

d) To provide a vertical clearance above the future river bank to the girder bottom (Minimum 70 cm)

As a result of the above a) to c), the setback distance from the present riverbank was determined at 30.0 m on the Khartoum side and 166.5 m on the Omdurman side as shown in Figures 7.2 and 7.3. On the Omdurman side, viaduct bridges will be planned to maintain stability of the approach road embankment. Consequently, the bridge length was determined as:



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.2	LOCATION OF KHARTOUM SIDE ABUTMENT
	JAPAN INTERNATIONAL COOPERATION AGENCY	



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.3	LOCATION OF OMDURMAN SIDE ABUTMENT
	JAPAN INTERNATIONAL COOPERATION AGENCY	

Bridge Length crossing river = 16.1+560.7+30.0 = 606.8 m
Viaduct Bridge Length = 150.4 m
Total Bridge Length = 757.2 m

7.2.3 Location of Center Span

Location of the center span of a bridge crossing a major navigational artery is a fundamental item in bridge planning. After a series of discussions with the NCK staff and Steering Committee Members of Sudan, the Study Team came to the conclusion that the location of the central span should be:

a) in the middle of the river at average water level, of which the elevation will be maintained for more than seven months in the year

b) over the navigational route which was directed by RTC in July 1989

c) at a place which will avoid shallow water depth in the river such as the back of the sand-bar

d) able to maintain the balanced embankment height of the two approach roads in the planning of the proposed elevation

As a result the central span crossing the navigational route was fixed at 178.5 m from the riverbank on the Omdurman side and 382.2 m from that of the Khartoum side.

7.2.4 Minimum Design Span Length

The minimum design span length should be in conformity with the following river hydrological or navigational requirements:

a) Any span in the river course should be more than the length of drift wood. Such a length is thought to be at most 30 feet taking into consideration the present status of standing trees along the White Nile. According to the past experience in Japan the minimum span length required is 36 meters about four times of the drift wood length, except for navigational purposes.

b) The percentage of disturbed area by all the bridge piers should be less than 5 % against the total flow area formed by the design high water level and embankment.

c) At the main span position, the required length in longitudinal direction should be more than the navigational clearance (minimum 45 m) as discussed in 7.2.1. Further, it is recommendable that this span length should clear the length of combined barges (2 x 36.0 = 72.0 m) if the barges drifted away from a tugboat by accident. In this regard, the navigational span length should be maintained at 72 meters or more.

On the basis of the requirements above, minimum design span lengths were determined as follows:

- For Main Span (Navigational Span) : 72.0 m
- For Side Span : 36.0 m

7.2.5 Transverse Cross Section

As discussed in planning the approach road in Chapter-VIII, the bridge width was determined taking into consideration the future traffic volumes in the vicinity of the planned bridge, the previous transportation study (Khartoum Traffic Management Study by BCEOM) and the Japanese Geometric Standards. The typical transverse cross-section is depicted in Figure 8.8 in Chapter-VIII. This section has a clear width of 22.75 meters consisting of:

a)	Roadway width (carriageway and shoulder)	:	8.75 m in each direction
b)	Median Strip	:	1.25 m
c)	Sidewalk	:	2.00 m on both sides

Cross fall of roadway is 2.0 % as a normal practice.

7.2.6 Bearing Stratum

The foundations of the bridge can be borne on the Nubian formation which is the basal rock layer around the proposed project area, as discussed in Chapter-VI.

According to subsurface exploration by mechanical boring, the Nubian formation lies approximately 10 meters or more below the ground surface. The Nubian formation consists of sandstone alternating with mudstone which is moderately consolidated. Considering the engineering properties in detail, the upper part of this layer for several meters is decomposed and loosened for a thickness of several meters, therefore, it is necessary to fix the foundations onto the stiff rock zone which is estimated to be approximately 15.0 m below the riverbed taking into consideration the requirements for foundation design.

The outline of soil profile along the proposed New White Nile Bridge is shown in Figure 6.5 and the proposed design values of the soil mechanics constants are given in Table 6.2 in Chapter-VI.

7.2.7 Design Criteria and Standards

(1) Design Loads

A) Dead Load

The following weights are to be used in computing the dead load based on the Standard Specifications for Highway Bridges adopted by the Japan Road Association (JRA).

		tf/cu.m
a)	Structural steel or cast steel	7.85
b)	Cast iron	7.25
c)	Plain concrete	2.35
d)	Reinforced concrete	2.50
e)	Prestressed concrete	2.50
f)	Asphaltic concrete pavement	2.30

In addition to the above, the following public utilities will probably be installed on the bridge in the future by the government's own work force. Anticipated weights are as follows:

a) Water main: 100 kgf/m (estimated by National Urban Water Corporation)

b) Petrol pipe line (4 inch pipe): 172 kgf/m(estimated by Public Corporation of Petroleum)

c) Telephone line: 70 kgf/m (estimated by the Study Team)

d) Electric line (5 No. 300 sq.mm cables): 200 kgf/m (tentatively estimated by the Study Team)

B) Live Load

Ministry of Finance and Economic planning (MOFEP) and the Roads and Bridges Public Corporation (RBPC)conducted similar surveys on the axle load and total weight of vehicles at major arterial roads between Port Sudan and Khartoum,in April 1986 and October 1988 respectively. The duration of each survey was about 1 week. These surveys have revealed the following:

a) About 99% of vehicles was below 20 tons

b) Maximum vehicle weight recorded was a 114.4t trailer of which maximum axle load showed the maximum 26 tons.

Recently, two new bridges were built over the White Nile at Kosti (Kosti Bridge) and over the Blue Nile at Wad Medani (Wad Medani Bridge). Both bridges were so designed as to carry a combination load of HA and a 115 ton vehicle having a maximum axle load of 30 tons.

From the above, the following live load will be adopted for the New White Nile Bridge.

Live Load: HA and 30 units of HB
stipulated in BS 5400: part 2-1978 adopted
by British Standards Institution (BSI)

In case of 30 units of HB, the maximum axle load is 30.6 tons which can carry more than the maximum axle loads of 26 tons recorded by MOFEP survey as well as the design live load of 30 tons adopted for Kosti Bridge and Wad Medani Bridge.

C) Impact

As discussed in the preceding paragraph, HA and HB30 loads adopted by BSI are to be used for the New White Nile Bridge. Both HA and HB30 include dynamic and impact effects. However, impact effect along with a 115t load was considered in the past design on Kosti Bridge and Wad Medani Bridge. In this case, the combination of live load and impact becomes heavier than HA and HB30 loads.

In order to maintain uniformity in bridge design in Sudan, the Study Team recommends addition of the impact effect to the live load. The following formula by JRA is to be adopted.

a) For steel bridge : $i = 20/(50+L)$

b) For reinforced concrete : $i = 7/(20+L)$

c) For prestressed concrete : $i = 10/(25+L)$

Where, i : impact fraction
L : span length in meter

D) Sidewalk Load

Sidewalk floors, stringers and their immediate supports will be designed for a live load of 500 kilogram per square meter of sidewalk area. Girders and other members will be designed for the following sidewalk live loads as specified in JRA.

Span 0 to 80 meters in length ---	350	kgf/sq.m
Span 80 to 130 meters in length ---	(430-L)	kgf/sq.m
Span over 130 meters in length ---	300	kgf/sq.m

E) Live Load Surcharge

As live loads will be calculated based on HA and HB30 unit loads of BSI, live load surcharges are to be calculated by the following:

for HA loading : 1.0 tf/sq.m
for HB30 unit loading : 1.25 tf/sq.m

F) Earth Pressure

There are various formulae to estimate earth pressure,i.e. Rankin's, Coulomb's, and Terzaghi's. In this study, earth pressure will be calculated by Coulomb's formula as adopted in JRA and other international standards.
Soil texture values of back-filling are as follows:

Unit weight	: 1.8 tf/cu.m
Angle of shear resistance	: 35 °
Cohesion	: 0 tf/sq.m

G) Stream Current Force

The effect of flowing water on piers is to be calculated by the following formula:

$$P=A*K*V^2$$

where,

P : Force from stream current (tf)
V : Velocity of current (m/sec)
A : Vertical exposed area (sq.m)
K : Constant, being 0.07 for square ends, 0.04 for circular or angle ends, and 0.02 for streamlined ends

The acting position of P is to be at 0.5*H from the riverbed, where H is the water depth in m. Velocity of current and water depth are as discussed in Chapter-VI and a summary is given in Table 7.1.

Table 7.1 Velocity and Water Level

	Velocity V(m/sec)	Water Depth H(m)
High Water Level (H.W.L) (R.L + 379.96 m)	0.35	7.86
Low Water Level (L.W.L) (R.L + 373.54 m)	1.32	1.44

H) Wind Load

Wind load produced by a uniform wind blowing horizontally will generally be derived from the following formula:

$$P=1/2*D*V^2*A*CD$$

where,

P : Wind pressure in kgf/sq.m
D : Air density in kgf.sec2/m**4
V : Design base wind velocity in m/sec
V = 31.5 m/sec
(Refer to the preceding Chapter VI)
A : Effective vertical exposed area in sq.m
CD: Drag coefficient

I) Thermal Force

According to the data of atmospheric temperature at the bridge site in Khartoum City for the years 1902 to 1988, the highest and lowest atmospheric temperatures have been 46.8° C and 6° C, respectively, and the mean monthly atmospheric temperature varies from 23.2° C to 33.7° C. Meanwhile, according to the actually observed atmospheric temperature in Japan, the highest and lowest ones are 38.0° C and -6.9° C, and the mean monthly atmospheric temperature varies from 5.0° C to 27.4° C. The fluctuation of atmospheric temperature at the bridge site can be deemed to be of a similar range to Japan. In addition, the temperature variation of the bridge girder is observed to be half of the atmospheric temperature variation in general. The design fluctuation range for the structural analysis of the New White Nile Bridge is based on the following Table 7.2:

Table 7.2 Fluctuation Range of Temperature

Bridge Type	Design fluctuation Range
Steel Bridge	+/- 30 deg.C
Concrete Bridge	+/- 15 deg.C

J) Wave Pressure

Wave pressure is estimated by different formulae according to the relationship between wave height and water depth. When the water depth is greater than twice the wave height, it is estimated for clapotis. Otherwise, it is estimated for a breaking wave.

According to the results obtained in Chapter VI, the wave height and the wave length are as follows:

Wave Height = 2.1 m,   Wave Length = 29.1 m

Such wave pressure is considered in combination with other loads in flood time only.

K) Ship Collision Force

A static design force from ship collision is taken as the equivalent impact force that would be consequential from the collision of a 2000 ton ship combined with 4 barges each of 500 tons. Ship collision force is calculated by the following formula:

$$F = W*V^2/(4*g*D)$$

where,

F : Ship collision force in metric ton
W : Weight of ship   W = 2,000 tf
V : Velocity of ship at collision time
  in flood time   V = 0.4 m/sec, and
  in the dry season   V = 1.6 m/sec
  These velocities are increased by 20 % of the stream current velocity.
D : Stopping-distance of ship   D = 1.0 m
g : Acceleration due to gravity   g = 9.8 m/sec2

The acting position of the collision force F is 1.0 meter above the water level.

L) Earthquakes

Earthquakes have occurred around the Red Sea, but no earthquake has been recorded along the Nile river. From the geological point of view, the Red Sea is situated on the tectonic line of the African rift. But Khartoum is situated about 700 kilometers inland from the Red Sea, and on part of the Continental crust. Therefore it is expected that earthquakes will not occur in Khartoum.

Even if a strong earthquake of a magnitude 7 on the Richter scale occurs beneath the Red Sea, the expected ground acceleration in Khartoum (about 700 km away from the epicenter) will be 14.5 gal only which is equivalent to a static seismic coefficient of 0.01. Such acceleration is not significant for bridge design.

As for existing structures in Sudan, earthquake effects were neglected in their design; i.e. Kosti Bridge, Gebel Aulia Dam, Sennar Dam and so on. In this regard, the earthquake effect will be neglected in the design of the Project.

(2) Design Method and Combinations of Loads

The designs of structures are to be made by means of the Allowable Stress Design Method adopted in JRA.

When the bridges are designed, loads to be considered are as follows:

Principal Load ---------------------------------- [P]

- Dead Load ---------------------------------- [D]
- Live Load including Sidewalk Load ------- [L]
- Impact ------------------------------------- [I]
- Earth Pressure -------------------------- [E]
- Stream Current Force ------------------- [SC]
- Buoyancy ----------------------------------- [B]

Subsidiary Load ------------------------------ [S]

- Wind Force --------------------------------- [W]
- Thermal Force ---------------------------- [T]

Particular Load ------------------------------ [PA]

- Wave Pressure ---------------------------- [WP]
- Ship Collision Force ------------------- [CO]
- Erection and Executive Load ------------ [ER]

The combinations of loads and their increase factors to allowable stresses are as follows:

Table 7.3 Combinations of Loads for Superstructure

Combination of Loads	K
[P]	1.00
[P] + [T]	1.15
[P] + [W]	1.25
[P] + [T] + [W]	1.35
[P] + [CO]*	1.50
[P] + [ER]	1.25

Notes: 1) K is the increase factor to allowable stress.

2) [CO]* indicates the Vehicle Collision Force.

Table 7.4 Combinations of Loads for Substructure

Combination of Loads	K
[P] + [WP]	1.00
[P] + [T] + [WP]	1.15
[P] + [W] + [WP]	1.25
[P] + [T] + [W] + [WP]	1.35
[P] + [CO] + [WP]	1.50
[P] + [ER] + [WP]	1.50

Notes, 1) K is the increase factor to allowable stress.

2) [WP] is considered in combination with other loads in flood time only.

(3) Materials

A) Concrete

The specified cylinder compressive strength of concrete at 28 days (fc) is shown in Table 7.5.

Table 7.5 Compressive Strength of Concrete

Kind of Concrete	fc (kgf/sq.cm)
Prestressed Conc., Precast	400
Prestressed Conc.,Cast-in-place	350
Cast-in-place Concrete Pile	300
Reinforced Conc., Superstructure	240
Reinforced Conc., Substructure	240
Non Reinforced Conc.	210

B) Structural Steel

Structural Steels are rolled materials for structures conforming to or equivalent to SS41, SM50 and SM53 as specified in Japan Industrial Standard (JIS).

C) Prestressing Steel

Steel wire, strand or bar for prestressing are materials conforming to or equivalent to SWPR, SWPD, or SBPR as specified in JIS.

D) Reinforcing Bars

Ordinary reinforcement is hot rolled deformed bars conforming to or equivalent to SD30 or SD35 as specified in JIS.

E) Others

Other structural materials are in accordance with the articles specified in JIS or other internationally authorized standards.

(4) Design Method

The designs of structures are to be made by means of Allowable Stress Design Method in principle. Limit-State Design Method (Load Factor Method), however, is to be employed to check the ultimate resistance of structures as required.

7.3 COMPARATIVE STUDY OF ALTERNATIVE BRIDGE TYPES

7.3.1 Alternative Bridge Types

Establishing the most promising bridge types is the initial and fundamental process in bridge planning and very important in achieving the most favorable bridge type which will normally dominate the construction cost.

Requirements for alternative bridge types are to be :

a) an applicable span length against the required navigational clearance

b) advantageous type for the transportation of structural members or materials

c) easy procurement of construction materials

d) ease of maintenance as much as possible

e) minimized construction cost

Consequently after examining conceivable bridge types, the following six types were selected for subsequent comparative study.

Type-a: Steel Lohse and Steel Plate Girder
Type-b: Steel Truss and Steel Plate Girder
Type-c: Steel Box Girder and Steel Plate Girder
Type-d: PC Box Girder with T-Type Pier and PC T (or I)-Girder
Type-e: PC Box Girder with V-Type Pier and PC T (or I)-Girder
Type-f: PC Cable-stayed Girder and PC T (or I)-Girder

7.3.2 Optimum Span Length by Bridge Types

Cost-span relation was examined for the respective bridge types, and results are summarized as shown in Fig 7.4. From the economic point of view, the following approximate span lengths are recommended for the side span portion:

a) Steel Plate Girder : 40 m

b) PC T(or I)-Girder : 36 m

As for the main (navigational) span portion, a horizontal clearance of 72 m, as discussed in the preceding section 7.2, governs the span length rather than the optimum one.

7.3.3 Span Arrangement by Bridge Types

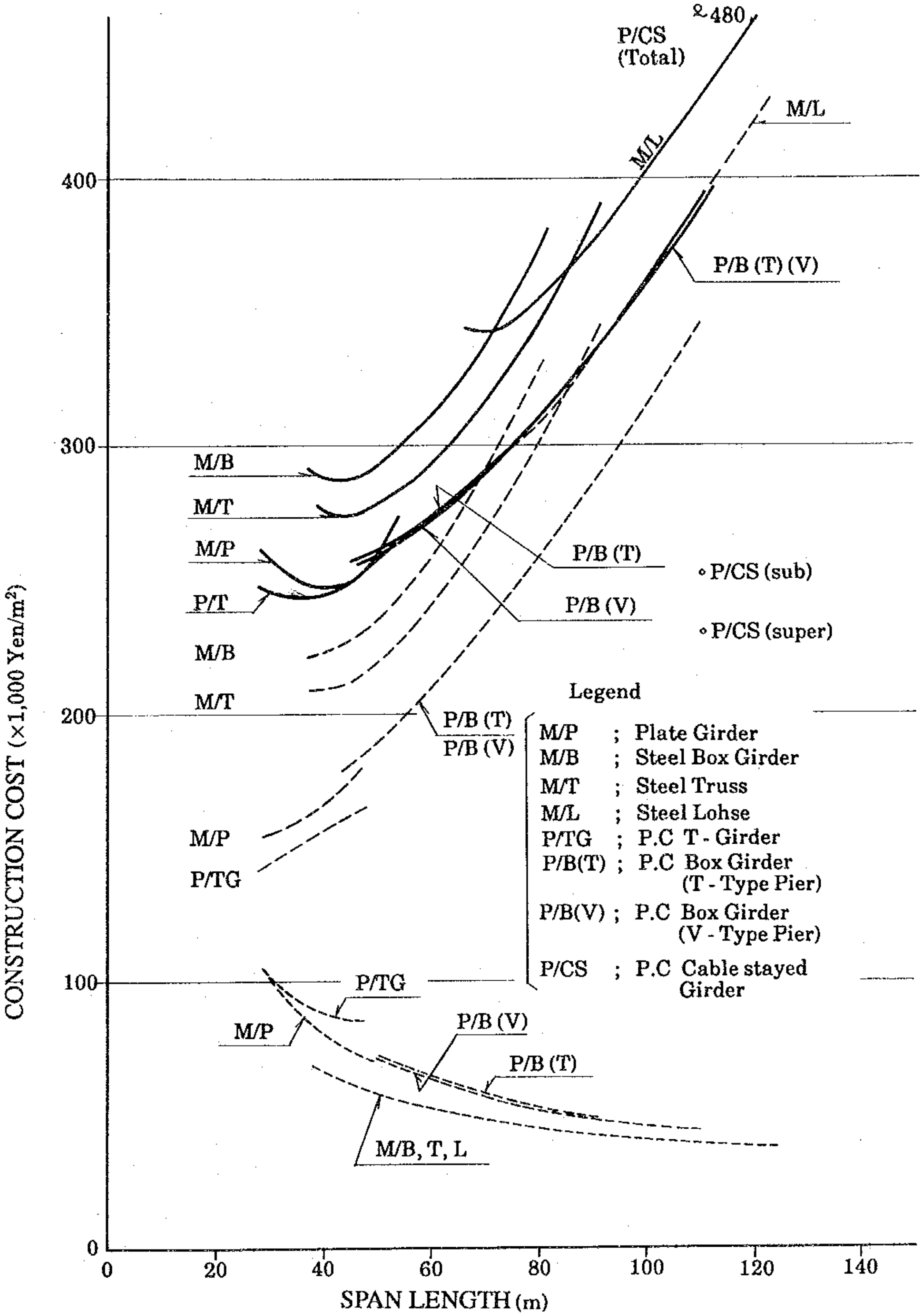
Six bridge types were selected as alternative ones in the preceding subsection 7.3.1. The initial step was to apprehend the possibility and advantages of the following two cases which differ in the types of side span:

Case-1 : Side spans are the same type as the main span (repetition type) or similar type

Case-2 : Side spans are the least cost type; plate girder for steel type bridges and PC T or I-girder for concrete type bridge

Figure 7.5 shows the results of initial examination of the above two cases. From this , it is clear that adoption of Case-2 will be advantageous with more than 10 % cost saving expected over that of Case-1.

In this regard, it is concluded that side spans for the various alternatives should be either steel plate girder or PC I-beam for the subsequent study.



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.4	COST-SPAN RELATION BY BRIDGE TYPE
	JAPAN INTERNATIONAL COOPERATION AGENCY	

7.3.4 Evaluation of Alternative Bridge Types

Evaluation was made based on the effects as tabulated in Figure 7.4 and 7.5. Briefly:

(1) Type-a: Steel Lohse and Steel Plate Girder

Material procurement and transportation is comparatively disadvantageous. Periodic maintenance like anti-corrosive painting will be required for major structural members. Such a steel bridge was not considered appropriate in the Sudanese environment due to its onerous maintenance commitment. From the viewpoint of aesthetics, this bridge type was thought the best. The rest of the evaluation items were ranked fair.

(2) Type-b: Steel Truss and Steel Plate Girder

The smaller structural members are advantageous for easier transportation and assembly. However, this bridge type was considered inferior from the viewpoint of aesthetics and periodic maintenance for corrosion.

(3) Type-c: Steel Box Girder and Steel Plate Girder

Transportation of large size structural members were considered inferior. Moreover, construction cost and maintenance cost were judged higher.

(4) Type-d: PC Box Girder with T-type Pier and PC T (or I)-Girder

The large girder depth at the pier section makes it necessary to raise the deck elevation more than the other alternatives, and such a high deck elevation is disadvantageous to the riding of vehicles as well as from an aesthetic viewpoint. The construction cost of this alternative is low.

(5) Type-e: PC Box Girder with V-type Pier and PC T (or I)-Girder

The girder depth at the pier section becomes less than that of the Type-d, which can reduce construction cost to the lowest among the alternatives. In addition, it is aesthetically more pleasing. Therefore, this alternative is superior in every evaluation item.

(6) Type-f: PC Cable-stayed Girder and PC T-Girder

Aesthetic viewpoint is superior. The construction cost, however, is the highest among the alternatives.

BRIDGE TYPE MAIN SPAN	 SIDE SPAN	SIDE VIEW AND SPAN ARRANGEMENT (UNIT; m)	CONSTRUCTION COST RATIO
TYPE-a STEEL LOHSE	STEEL LOHSE	VIADUCT / LOHSE / LOHSE / LOHSE 2@77.0 =154.0 / 80.0 / 5@74.6=373.0	1.270
	PLATE GIRDER	PLATE GIRDER / LOHSE / PLATE GIRDER 3@38.0 =114.0 / 40+80+40 =160.0 / 8@41.625=333.0	1.000
TYPE-b STEEL TRUSS	STEEL TRUSS & PLATE GIRDER	PLATE GIRDER / TRUSS / TRUSS 2@45.0 =90.0 / 64+80+64 =208.0 / 5@61.8=309.0	1.092
	PLATE GIRDER	PLATE GIRDER / TRUSS / PLATE GIRDER 2@45.0 =90.0 / 64+80+64 =208.0 / 8@38.625=309.0	1.000
TYPE-c STEEL BOX	STEEL BOX & PLATE GIRDER	PLATE GIRDER / BOX / BOX 2@45.0 =90.0 / 64+80+64 =208.0 / 6@51.5=309.0	1.086
	PLATE GIRDER	PLATE GIRDER / BOX / PLATE GIRDER 2@45.0 =90.0 / 64+80+64 =208.0 / 8@38.625=309.0	1.000
TYPE-d P.C BOX (T-TYPE PIER)	BOX	BOX / BOX / BOX 31+62 =93.0 / 62+80+62 =204.0 / 5@62.0=310.0	1.100
	T-GIRDER	T-GIRDER / BOX / T-GIRDER 3@35.0 =105.0 / 49+80+49 =178.0 / 9@36.0=324.0	1.000
TYPE-e P.C BOX (V-TYPE PIER)	BOX	BOX / BOX / BOX 31+62 =93 / 62+80+62 =204.0 / 5@62=310.0	1.096
	T-GIRDER	T-GIRDER / BOX / T-GIRDER 3@35 =105.0 / 49+80+49 =178.0 / 9@36.0=324.0	1.000
TYPE-f P.C CABLE-STAYED	BOX	BOX / CABLE STAYED / BOX 2@69.5 =139.0 / 110+70 =180.0 / 5@57.6=288.0	1.078
	T-GIRDER	T-GIRDER / CABLE STAYED / T-GIRDER 4@34.75 =139.0 / 110+70 =180.0 / 8@36.0=288.0	1.000

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE

Fig. 7.5 SPAN ARRANGEMENT BY BRIDGE TYPES

JAPAN INTERNATIONAL COOPERATION AGENCY

7.3.5 Selected Bridge Type

As a result of evaluation of the alternative bridge types, a PC Box Girder with V-type pier and PC T(or I)-Girder is recommended as the New White Nile Bridge. The reasons are summarized as follows:

a) the space below the bridge girder is wider

b) the girder depth at the piers of the main span is smaller than other cantilever concrete bridge types

c) driving is more comfortable because of continuous girder type

d) the construction materials for the structure are available in Khartoum except for cement, prestressing tendons and rebars

e) a saving in maintenance costs is possible

f) agreeable aesthetics are expected

g) the construction period is shorter

h) the construction cost is as low as for PC Box Girder with T-type pier

Further reasons to select the V-type pier rather than T-type pier are as follows:

The clearance requires an area of 12 m x 45 m above high water level in a normal situation. The width of 72 m in the horizontal direction is requested only for drifting barges in an emergency as mentioned in the planning conditions. Therefore, a shorter span length in midspan is better from the economic viewpoint. The V-type pier can ensure a shorter span length than the T-type pier.

As the bridge width is relatively wide at 23.55 m , the transverse cross-section will be designed as a multi-cell type to maintain structural stiffness and minimize girder depth. Moreover, the construction method of a cantilever has an advantage that the erection of the box girder is above water. The cantilever construction employs two sets of traveling wagons which determine the minimum length of in-situ girder segment on the pier (so called hammer head). The required length for setting the traveling wagon on the hammer head is more than 13 m in case of a 350 ton-type of carriage, which will be selected by requirements of the preparatory works and construction period. In such a case, it makes little difference between the scaffolding costs of V-type and T-type pier, however, the hammer head of the adjacent pier rather guarantees

BRIDGE TYPE		SIDE VIEW AND SPAN ARRANGEMENT (UNIT; M)	STRUCTURAL FEATURES	MATERIAL PROCUREMENT & TRANSPORT	MAINTENANCE	AESTHETICS	CONSTRUCTION PERIOD	CONSTRUCTION COST (X 1,000 Ls)	OVERALL EVALUATION
MAIN SPAN	SIDE SPAN								
TYPE-a STEEL LOHSE	PLATE GIRDER	PLATE GIRDER 3@38.0 =114.0 / LOHSE 40+80+40 =160.0 / PLATE GIRDER 8@41.625=333.0	△	×	×	◎	40 MONTHS △	SUPER : 336,060 SUB : 115,430 TOTAL : 451,490 △	×
TYPE-b STEEL TRUSS	PLATE GIRDER	PLATE GIRDER 2@45.0 =90.0 / TRUSS 64+80+64 =208.0 / PLATE GIRDER 8@38.625=309.0	△	△	×	×	37 MONTHS ○	SUPER : 350,460 SUB : 111,980 TOTAL : 462,440 ○	×
TYPE-c STEEL BOX	PLATE GIRDER	PLATE GIRDER 2@45.0 =90.0 / BOX 64+80+64 =208.0 / PLATE GIRDER 8@38.625=309.0	×	×	×	△	34 MONTHS ○	SUPER : 365,690 SUB : 111,980 TOTAL : 477,670 ×	×
TYPE-d P.C BOX (T-TYPE PIER)	T-GIRDER	T-GIRDER 3@35.0 =105.0 / BOX 49+80+49 =178.0 / T-GIRDER 9@36.0=324.0	△	○	◎	△	34 MONTHS ○	SUPER : 299,880 SUB : 131,020 TOTAL : 430,900 ○	○
TYPE-e P.C BOX (V-TYPE PIER)	T-GIRDER	T-GIRDER 3@35 =105.0 / BOX 49+80+49 =178.0 / T-GIRDER 9@36.0=324.0	○	○	◎	◎	34 MONTHS ○	SUPER : 299,880 SUB : 130,780 TOTAL : 430,660 ○	◎
TYPE-f P.C CABLE-STAYED	T-GIRDER	T-GIRDER 4@34.75 =139.0 / CABLE STAYED 110+70 =180.0 / T-GIRDER 8@36.0=288.0	△	△	△	◎	40 MONTHS △	SUPER : 286,310 SUB : 227,410 TOTAL : 513,720 ×	△

LEGEND : ◎ VERY GOOD ○ GOOD △ FAIR × BAD

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE

Fig. 7.6 EVALUATION OF ALTERNATIVE BRIDGE TYPE

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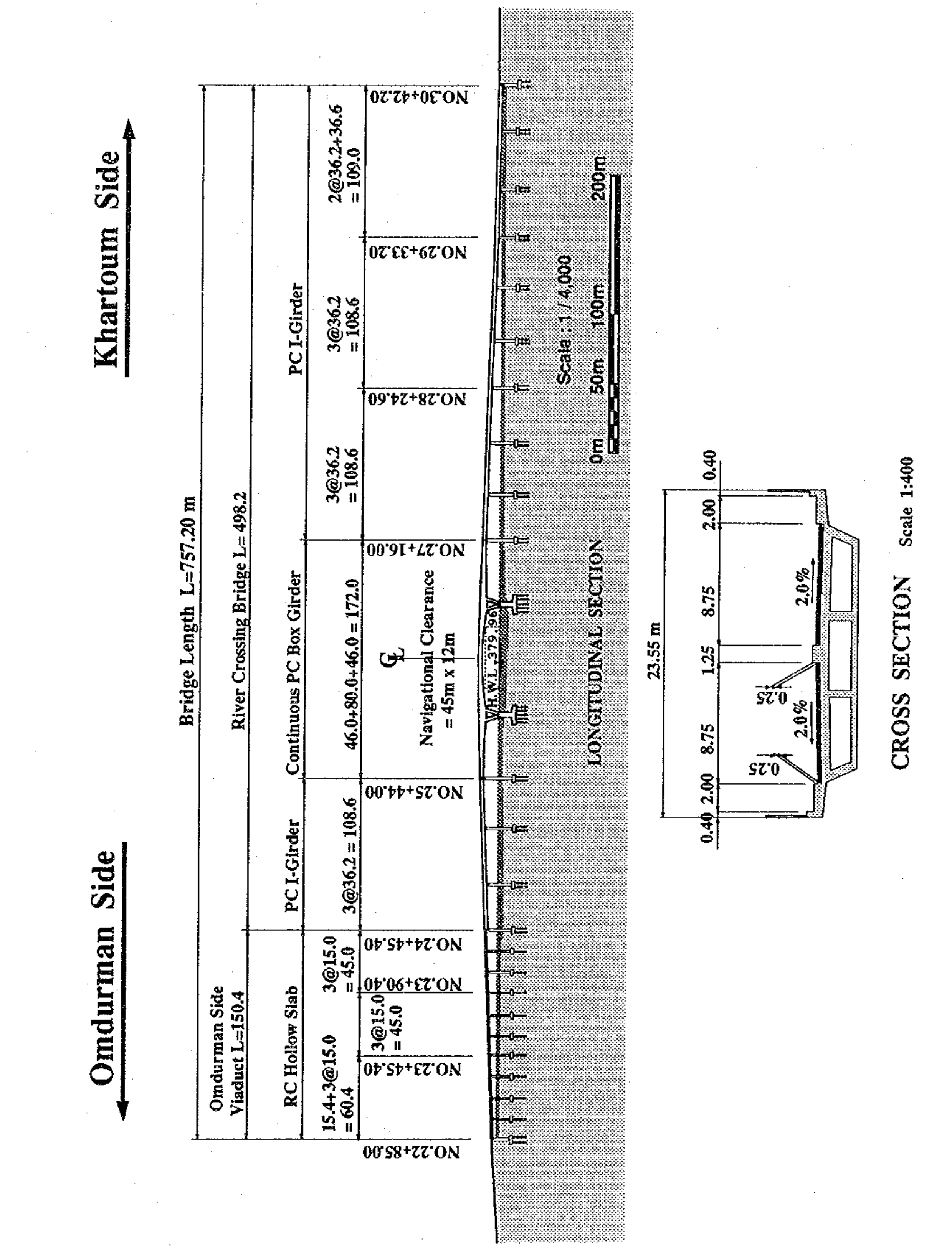
structural stability by the framed V- leg triangular form during cantilever construction.

The bending moments at the pier during cantilever construction are dominated by its cantilevered length. The bending moment of the V-type is about 76 % of that of the T-type when the cantilevered girders will encounter each other at the middle of the center span.

Furthermore, the box girder depth of the V-type pier can be reduced more than that of the T-type pier since the former pier is supported by the framed V-legs of the pier. Therefore, the construction cost of the V-type pier can be lowered.

7.3.6 Configuration of the Proposed Bridge

As a result of comparative study of alternative bridge types, the proposed bridge consists of 3-span continuous PC box girder with V-type pier, composite PC I-girder and continuous RC hollow slab. A configuration of the proposed bridge is shown in Figure 7.7.



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.7	THE PROPOSED BRIDGE
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7.4 STUDY ON THE PROPOSED SUPERSTRUCTURE

7.4.1 PC Box Girder with V-Type Pier for Main Span

(1) Structural Layout

The structural layouts for the main span were examined based on the following conditions:

a) The bending moment due to variation of structural layouts is to be minimized
b) The displacement or restriction of the girder due to temperature fluctuation, drying shrinkage and creep is to be minimized
c) The bridge surface is to be continuous in the direction of the bridge axis
d) The forces transmitted to the foundations are to be minimized
e) The structural layout is to be superior from the aesthetic viewpoint

Variation of the structural layouts are discussed in Appendix-7.5. Consequently, a rigid frame type without hinge is recommended because of less displacement, comfortable driving and less maintenance cost.

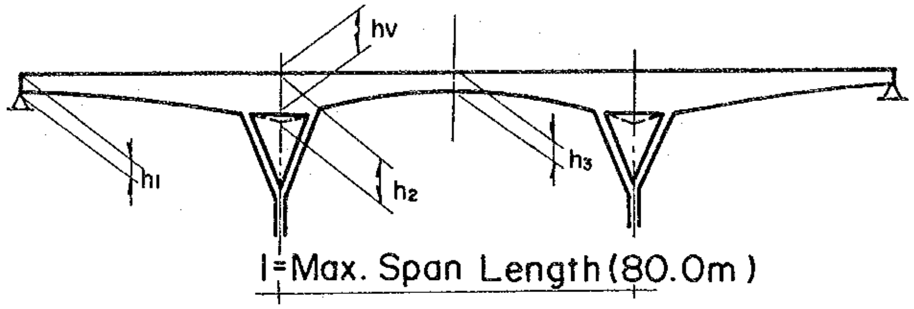
Structural analysis was conducted and results are summarized in Appendix-7.6.

(2) Girder Depth

Girder depth and its variation are illustrated in Figure 7.8.

As the maximum span length is 80.0 m, the girder depth at both ends of the box girder, h1, is computed to be 2.0 m to 2.7 m. Considering HA and HB loadings and the balanced depth with the girder of the side span, h1 = 2.1 m is adopted. On the other hand, the girder depth at the pier, h2, is computed to be 4.0 to 5.0 m, considering the V-type pier with rigid frame effects. Lastly, the girder depth at the center of midspan is computed to be 1.3 to 1.8 m. h3 = 2.1 m is adopted based on the sectional forces transmitted by HA and HB loadings.

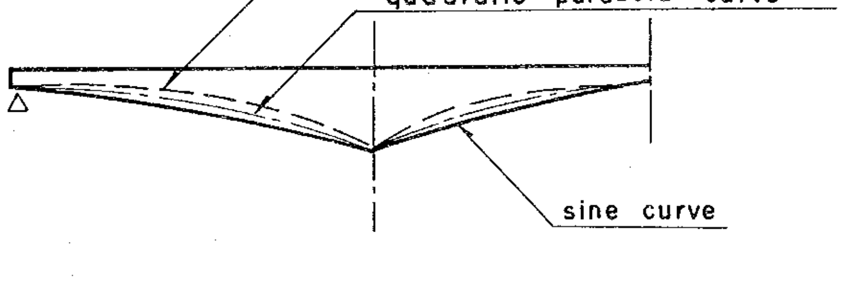
Generally, three different kinds of transition curve are used for the girder depth change in the intermediate parts between span end and span center; sine curves, quadratic parabola curves and cubic parabola curves. Where a continuous girder is supported by bearings not as in the rigid frame type, a quadratic parabola is adopted as a usual practice. A Cubic parabola is often
.pa



h1 = 1/30 ~ 1/40 (2.0 ~ 2.7m)
h2 = 1/16 ~ 1/20 (4.0 ~ 5.0m)
h3 = 1/45 ~ 1/60 (1.3 ~ 1.8m)

Note : Values in the brackets show the girder depth for a maximum span length of 80.0m.

Girder Depth of Box Girder



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.8	GIRDER DEPTH AND ITS VARIATION
	JAPAN INTERNATIONAL COOPERATION AGENCY	

used when less girder depth is required by the limitation of vertical clearance below the girder bottom. A Sine curve is used where hinges are installed at the span center.

As the live load to be considered in designing the New White Nile Bridge is very heavy and shear force is quite big, a sine curve is more advantageous than others from the viewpoint of shear resistance. Therefore, a sine curve was chosen as the transition curve for the girder depth change.

(3) Sectional Structure

The sectional structure type for the box girder bridge was determined after consideration of:

a) required bridge width
b) reduction of self weight
c) prestressing steel arrangement
d) sufficient resistance to torsion

The width of the bridge leads to different transverse cross-sections as shown in Figure 7.9.

Judging from the required bridge width of 22.75 m, Type-3 is recommended. Two varieties are considered for Type-3 as shown in Figure 7.9; a single three-cell box girder and a twin single box girder. The former is expected to have sufficient rigidity for the girder in the transverse direction due to its closed section. On the other hand, the rigidity for the latter is smaller. Regarding the bending moment in the floor slab, this may be increased by different settlements between the two boxes.

For the above-mentioned reasons, a single three-cell box girder was determined to be adopted in the subsequent design.

(4) Proposed Elevation of Bridge

The proposed elevation is discussed in detail in Chapter-VIII. As regards the proposed elevation of the PC Box Girder portion, the following two points are considered:

a) Clearance above the HWL to the bottom slab of girders: being more than 12.0 meters at the navigational course.

b) Girder depth: refer to paragraph (2) above.

Consequently, deck elevation was decided as shown in Figure 7.10.

TYPE - 1 Single Box Girder

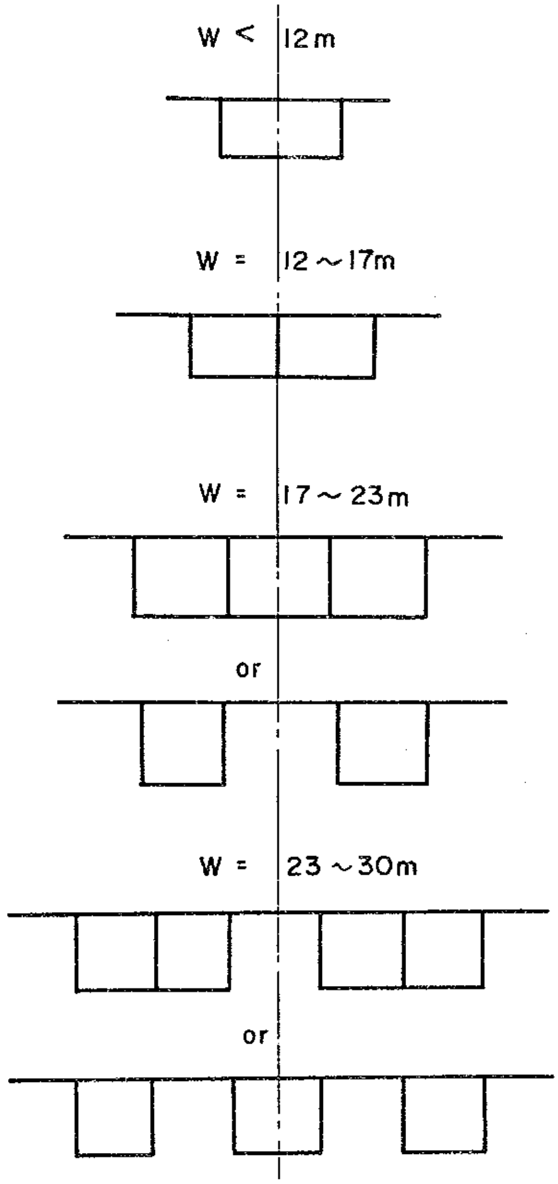
TYPE - 2 Single Two - cell Box Girder

TYPE - 3 A : Single Three - cell Box Girder

B : Twin Single Box Girders

TYPE - 4 A : Twin Two - cell Box Girders

B : Triplet Single Box Girders



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.9	ALTER NATIVES OF BOX ARRANGMENT
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(5) Segmentation of Box Girder

The supporting capacity of travelling wagons will determine the segment lengths of balanced cantilever progressing method, since it must withstand the bending moment induced by the segment weight. As the girder depths in the vicinity of the pier are bigger and their weights per length are heavier than those at span end or midspan, the segment lengths will be shorter in the vicinity of the pier and longer in the span end or midspan. According to past experiences of this construction method, a 350-ton travelling wagon is judged appropriate and segment lengths of 3.0 to 3.5 meters will be employed.

On the other hand the zero(0) segment on the pier, so-called hammer head portion, will be determined by the size of the travelling wagon and will not be determined by its supporting capacity. For reference purposes, standard segment lengths are shown in Table 7.6.

Table 7.6 Standard Segment Length on Piers

Construction Method	200-ton Wagon	350-ton Wagon
Symmetric cantilevering	12 m	15 m
Asymmetric cantilevering	10 m	13 m

(6) Preliminary Study on Structural Elements

A) Thickness of Structural Components of Box Girder

Detailed discussions are made in Appendix-7.7 and a summary is given below:

The thickness of top slab is determined at 30 centimeters by the minimum requirement for the prestressing arrangement such as steel tendons, sheaths and anchors.

The Box girder section consists of four webs, each of which has the same thickness. The thickness is determined by the required area to withstand bending moments and shearing forces as well as the minimum requirement for the prestressing arrangement.

The thickness of the bottom slab is governed by such factors as the compressive and tensile sections of the positive and negative bending moments in the box girder. These thicknesses vary from 25 to 65 centimeters.
The configuration of these thicknesses is shown in Figure 7.10.

B) Diaphragms

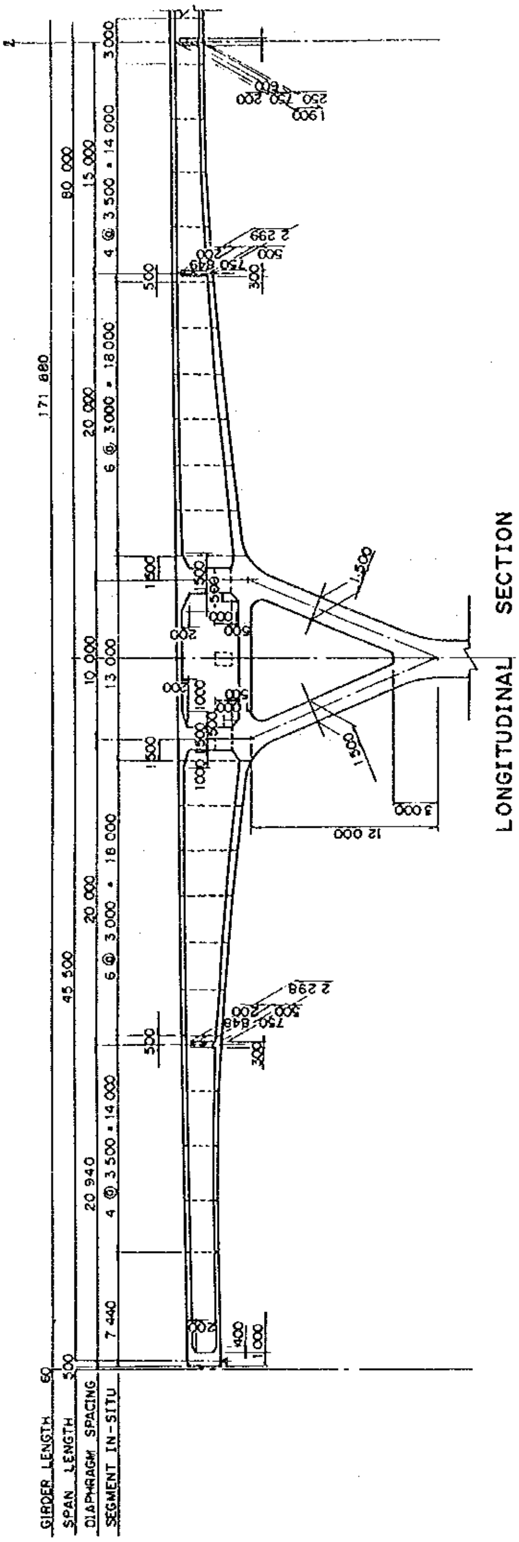
Detailed discussion is made in Appendix 7.7. The proposed new bridge has three different types of diaphragms: end diaphragm at span end; intermediate diaphragm at midspan; and intermediate support diaphragm on the points of conjunction between girder and V pier.

The configuration of these diaphragms is as shown in Figure 7.10.

C) Others

Bearings at span ends were determined as roller type ones mainly consisting of forged or cast iron because of the relatively big reactions.

Expansion joints between span ends of this box girder and PC I-girder were determined as rubber type ones mainly for the comfort of driving. This rubber type joint should be stiffened by steel members taking into consideration the Sudanese environment due to its onerous maintenance commitment.



DECK LEVEL (RL+m)	GIRDER DEPTH (m)	THICKNESS (m) LOWER SLAB	WEB	UPPER SLAB
394.623	2.100	0.250	0.600	0.300
394.623	2.100	0.250	0.600	0.300
394.624	2.100	0.250	0.600	0.300
394.760	2.100	0.250	0.350	0.300
394.820	2.132	0.290	0.350	0.300
394.877	2.228	0.340	0.350	0.300
394.931	2.385	0.380	0.350	0.300
394.982				
394.983	2.598	0.430	0.400	0.300
395.025	2.822	0.460	0.450	0.300
395.065	3.079	0.500	0.500	0.300
395.103	3.364	0.540	0.550	0.300
395.139	3.671	0.580	0.600	0.300
395.173	3.995	0.610	0.650	0.300
395.205	4.328	0.650	0.650	0.300
395.220	4.497			
395.267	4.500	0.650	0.650	0.300
395.309	4.503			
395.320	4.334	0.650	0.65	0.300
395.342	4.000	0.610	0.650	0.300
395.361	3.675	0.580	0.600	0.300
395.378	3.367	0.540	0.550	0.300
395.394	3.081	0.500	0.500	0.300
395.407	2.824	0.460	0.450	0.300
395.418	2.599	0.430	0.400	0.300
395.419				
395.429	2.385	0.380	0.350	0.300
395.437	2.228	0.340	0.350	0.300
395.442	2.132	0.290	0.350	0.300
395.445	2.100	0.250	0.350	0.300
395.445	2.100	0.250	0.350	0.300

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE

Fig. 7.10 CONFIGURATION OF STRUCTURAL ELEMENTS

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7.4.2 PC Composite I-girder for Side Span

(1) Structural Layout

The structural layouts for the side span were examined based on the following conditions:

a) reduction of number of expansion joints due to a large number of queued piers with short span length
b) minimum maintenance cost
c) easier construction, especially considering the site conditions and required capacity of cranes

Appendix 7.8 deals with detailed discussion and, accordingly, connecting girder type, in which the cast-in-situ RC deck is constructed as a continuous structure, is determined for side spans.

(2) Girder Depth

The girder depth of PC I-girder is usually computed from a depth to span ratio of from 1/18 to 1/20 for 20m to 40m. In this study, the designed side span length is to be 36m and the girder depth is to be 1.8m to 2.0m. However, 2.1m is adopted for the girder depth from the viewpoint of HA and HB loadings and concrete strength, considering the availability at the bridge site. Figure 7.11 shows the girder depth of I-girder.

(3) Preliminary Study on Structural Elements

A) Main Girder

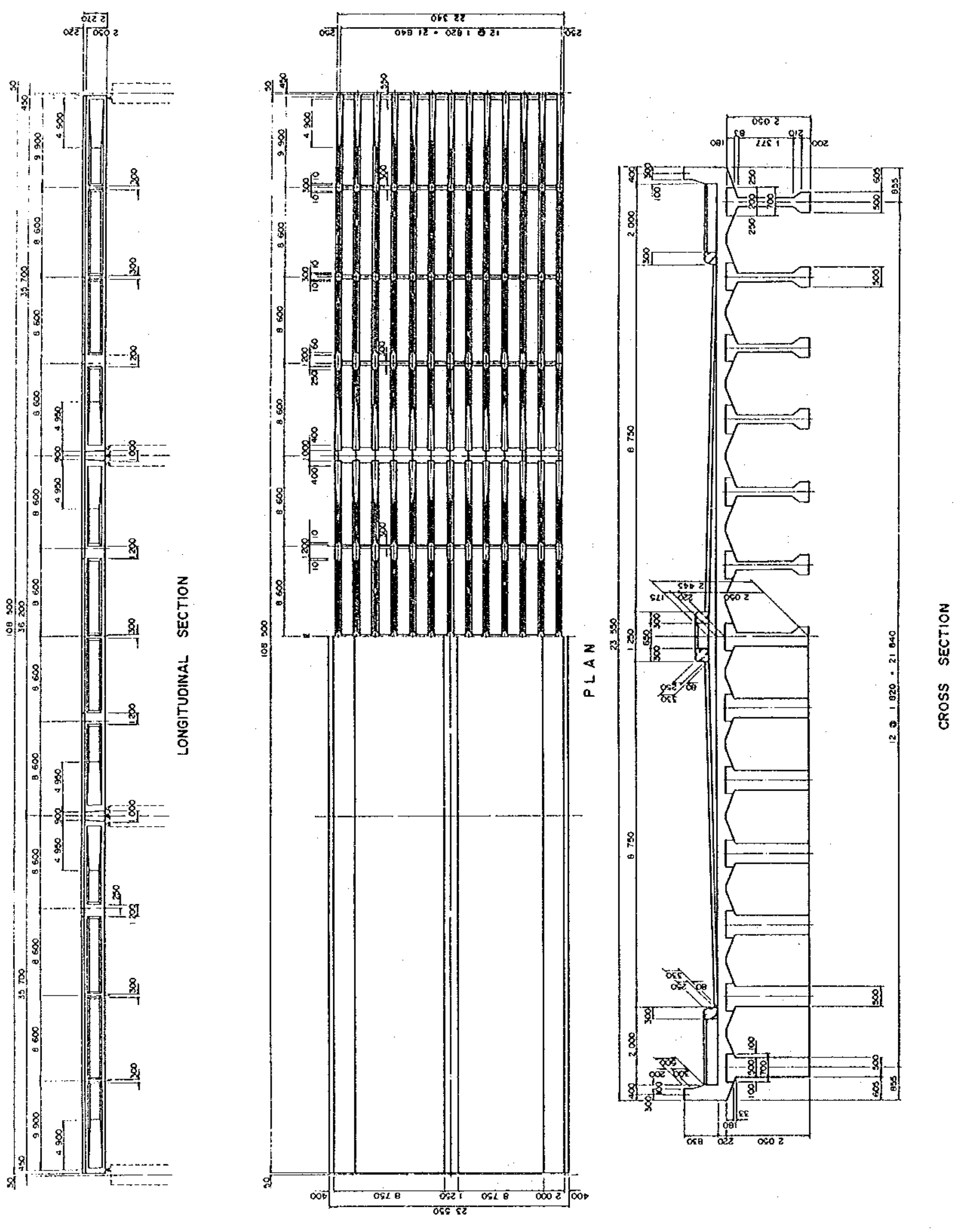
Anticipated section of PC I-girder was determined based on past practice experienced on similar projects in Japan, however, in which some modification was made due to the difference of live loads. Configuration of girders is shown in Figure 7.11.

B) Diaphragms

Four different types of diaphragm were considered: end diaphragm on end bearing; intermediate normal diaphragm at midspan; intermediate diaphragm adjacent to pier; and connection diaphragm at connecting portion between precast girders just on pier.

Thicknesses of these were tentatively determined as:

a) End diaphragm: 45 cm
b) Intermediate normal diaphragm: 30 cm
c) Intermediate diaphragm adjacent to pier: 120 cm
d) Connection diaphragm: 100 cm



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.11	CONFIGURATION OF PC I GIRDER
	JAPAN INTERNATIONAL COOPERATION AGENCY	

C) Others

Bearings were determined as elastomeric rubber type bearings in view of cost.

Similar to PC box girder, expansion joints between span ends were determined as rubber type ones.

7.4.3 RC Hollow Slab for Omdurman Side Viaduct

(1) Structural Layout

Three span continuous RC Hollow Slab was determined as the structural type of the viaduct. Total length is 45 meters having a span length of 14.76 meters for each span.

(2) Girder Depth

From past experience in Japan and on due consideration of design live loads, girder depth was determined at 90 centimeters having a bottom slab width of 18.95 meters as shown in Figure 7.12.

(3) Preliminary Study on Structural Elements

A) Void

A 65 centimeter void which is one of the standard types in Japan will be used in the slab.

B) Diaphragms

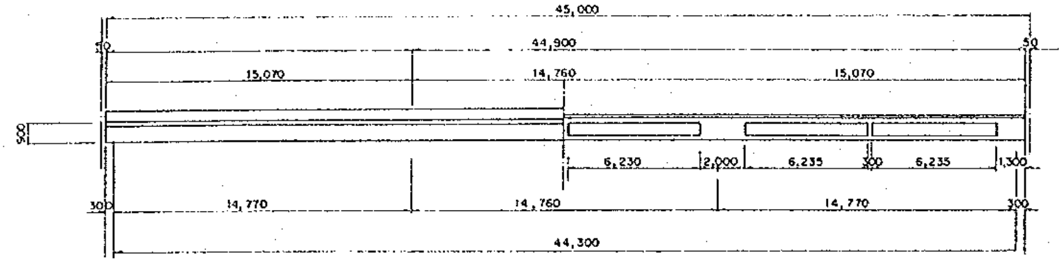
Three different types of diaphragm were considered: end diaphragm at span end; intermediate diaphragm at midspan; and intermediate bearing diaphragm on pier.

The thicknesses tentatively proposed in this preliminary study are as follows:

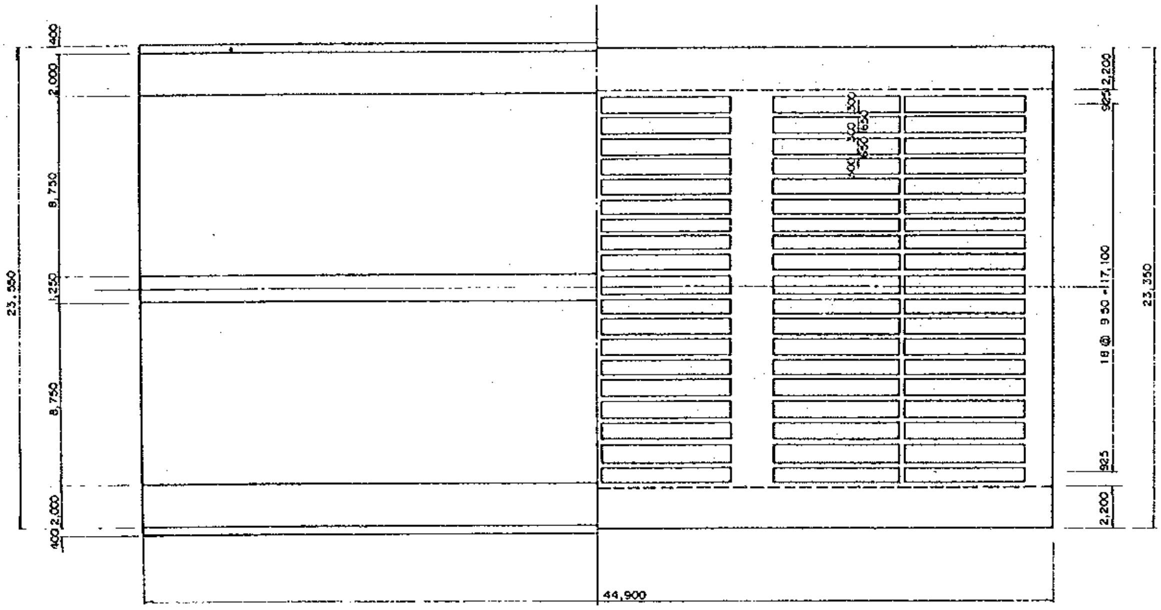
a) End diaphragm: 130 cm
b) Intermediate diaphragm: 30 cm
c) Intermediate bearing diaphragm: 200 cm

C) Others

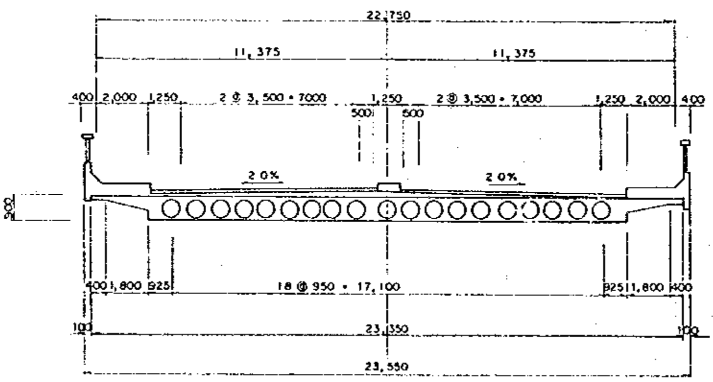
Similar to PC Composite I-girder, elastomeric rubber bearings and rubber type expansion joints were selected.



LONGITUDINAL SECTION



PLAN



CROSS SECTION

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	**Fig.** 7.12	CONFIGURATION OF RC HOLLOW SLAB
	JAPAN INTERNATIONAL COOPERATION AGENCY	

7.4.4 Other Miscellaneous Facilities

(1) Lighting

Lighting poles equipped with 300 watt bulbs will be installed on the outside of both sidewalks at an interval of 30 to 40 meters. By them, an average illuminance of 7 lux will be obtained.

(2) Drain Pipes

Drain pipes having a diameter of 12.5 centimeters will be provided on the outside edges of both roadways at 20 meter intervals.

7.4.5 Tentative Provision of Utility Appurtenances

The following public utilities will probably be installed on the new bridge in the future by the government's own work force. As discussed in Subsection 7.2.7, the dead weight of these appurtenances were considered in structural calculations.

a) Water main by National Urban Water Corporation

b) Petrol pipe line by Public Corporation of Petroleum

c) Telephone line by Sudan Telecommunications Corporation

d) Electric cable by The National Electricity Corporation

It seems clear that space for installing the above utility appurtenances can be kept between webs of the girder or on the outside faces of the webs. Therefore, determination of specific locations of such appurtenances was not conducted in the Study.

7.5 STUDY ON THE PROPOSED SUBSTRUCTURE

7.5.1 Configuration of Substructure

(1) Abutments

Two abutments are to be scheduled in this study.

Abutments are always provided at the junction of a bridge structure and an approach road, with a twofold structural function; they support the superstructure of the bridge and they withstand the embankment structure of the approach road. Therefore, some parts of the abutments relate to elements of the approach road embankment. The relationship between abutments and embankments was checked by taking into consideration the alignment determined in Chapter VIII and its result is represented in Figure 7.13.

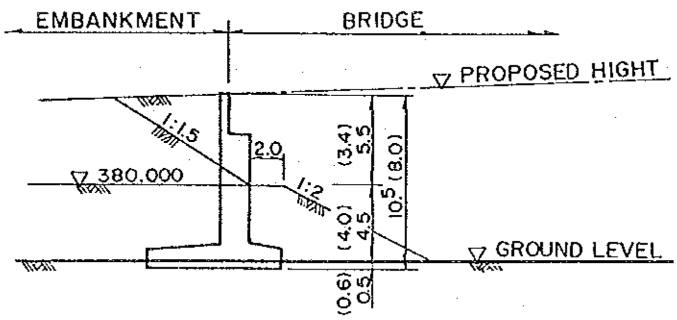


Figure 7.13 Longitudinal Section at Abutment Site

The pilecaps shown in this figure are embedded in the ground about 0.5 meters in depth for the following reasons:

a) Earth cover, which is laterally measured from the front edge of the embankment to the front face of the stem wall of the abutment, is only 2.0 m. Resistance resulting from passive earth pressure cannot be expected to be sufficient. Therefore, a stub abutment type is not recommended and a full height abutment with the pilecaps embedded in the existing ground is the recommended type for the proposed bridge.

b) Since no construction yard is scheduled on the Khartoum side, the embankment will be started after the abutment work.

The heights of the abutments become 10.5 and 8.0 meters on the Omdurman and Khartoum side respectively. Such heights are the most favorable sizes to employ inverted T-shaped abutments from the viewpoint of least construction cost. In this regard, inverted T-shaped full-height abutments are employed in the subsequent study and design.

(2) Piers

PC Box Girders, PC Composite I-girders and RC Hollow Slabs were proposed in the preceding Section 7.4 as the types of superstructure. A configuration study was made for substructures to support these different types of superstructure. Special emphasis was placed upon what types would harmonize with the proposed superstructure.

Types and configuration of the piers are determined based on the following:

a) to support tightly the superstructures

b) not only to harmonize with the respective superstructure types but also to achieve a good aesthetic appearance for the bridge as a whole

c) to be superior in construction aspects

d) to employ shapes of pier column or wall which can minimize the effect of adverse natural flow distribution; caution should be exercised to avoid undue turbulent flow

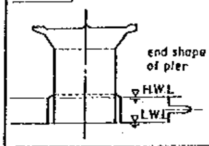
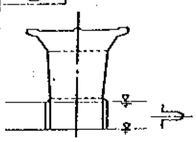
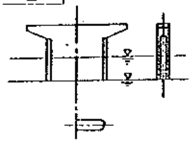
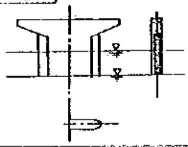
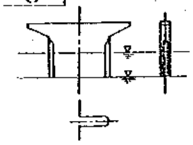
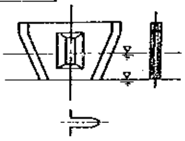
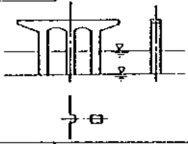
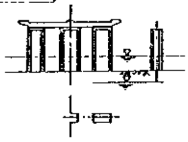
e) to have the flexibility in range to cope with the fluctuation of river water levels and changes in the deck elevation of the superstructure

f) to keep the area disturbed by the piers at less than 5 % of the total flow area in the dry season

For the purpose of determining the types and shapes of the piers, conceivable alternatives (cases 1 to 8) were selected for comparative study as shown in Figure 7.14. From this table, configurations for various piers is recommended as follows:

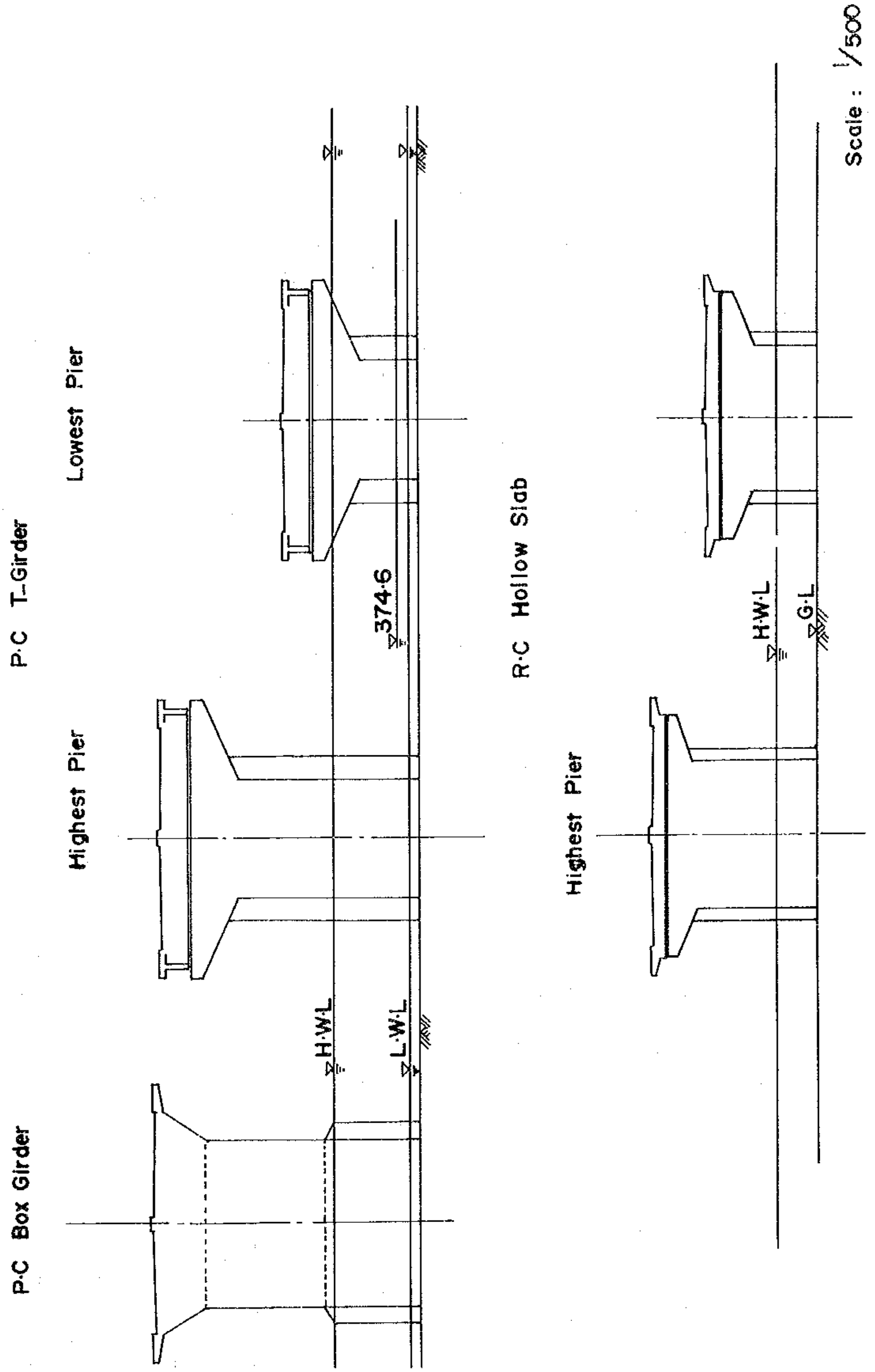
Piers for PC Box Girder	: Case 1
Piers for PC Composite Girder	: Case 4
Piers for RC Hollow Slab	: Case 4

The concept of the above is represented in Figure 7.15.

Case No. Configuration of Pier	Objective Superstructure	Evaluation Item				Overall Evaluation
		Aesthetics	Constructon Aspects	Response to Turbulent Flow	Response to Fluctuation Pier Height	
1 	P.C Box Girder	△	△	△	— Pier height is not varied	Overall evaluation is better than case 2. So, Recommended for P.C. Box Girder
2 		○	×	○	— Pier height is not varied	Aesthetics is better than case 1. But construction aspect is worse. So, Not Recommended
3 		○ (△)	○ (○)	○ (○)	○ (○)	Overall evaluation is best as well as case 3. But, this does not give us a new impression. So, Not Recommended
4 		○ (△)	○ (○)	○ (○)	○ (○)	Overall evaluations are the best. So, Recommended for P.C. Composite Girder R.C. Hollow Slab.
5 	P.C Composite Girder R.C Hollow Slab	○ (○)	○ (○)	○ (○)	○ (○)	Construction aspects are worse than case 3, 4. So, Not Recommended
6 		○ (○)	× (×)	○ (○)	× (×)	Construction aspects are bad. In case of the same gradient of wall being adopted, the width at the bottom of walls differs from others. Not Recommended.
7 		△ (△)	○ (○)	× (×)	○ (○)	This has a cross section which causes turbulent flow. So, Not Recommended.
8 	R.C Hollow Slab	○	○	×	○	This is in the best harmony with the superstructure. But this shape is likely to cause turbulence. So, Not Recommended.

Note: Judging marks: ○ : Good, △ : Fair, × : Bad.

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.14	EVALUATION OF PIER TYPE
	JAPAN INTERNATIONAL COOPERATION AGENCY	



THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.15	RELATION BETWEEN PIERS AND HWL & GROUND LEVEL
	JAPAN INTERNATIONAL COOPERATION AGENCY	

7.5.2 Structural Details

(1) Widths of Bridge Seat

In the direction of the bridge axis, the bridge seat guarantees sufficient space for the bearings to be installed with the gaps (S) between the girders or the ends of the girders and the parapet wall of the abutment. The bearing bed, which is transmitting the force from the superstructure to the substructure, should contribute to avoiding stress concentration. Fig. 7.16 and the formulae below give guidelines to the width (S) from the edge of the bearing to the front of the substructure.

$$S = 20 + 0.5 \times L$$

Where, L : Span length (m)

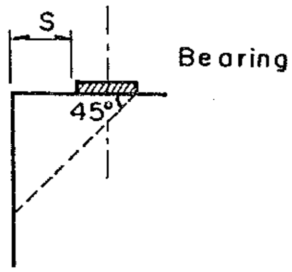


Figure 7.16 Bearing Seat

As a result, widths of the bridge seats are standardized as shown in Table 7.7.

Table 7.7 Widths of Bridge Seat By Superstructure Type

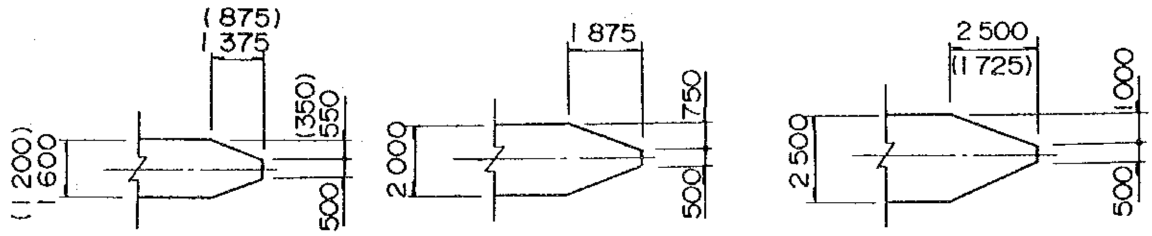
Superstructure	Substructure	Width of Bridge Seat Min. Req.	Adopted
RC Hollow Slab	Abutment	850	1,000
	Mid. Piers	1,600	1,600
--------------------	End Pier	1,770	2,000
PC Composite Girder	Mid. Piers	1,980	2,000
--------------------	End Pier	2,470	2,500
PC Box Girder	Mid. Piers	Rigid Frame	
--------------------	End Pier	2,470	2,500
PC Composite Girder	Mid. Piers	1,980	2,000
	Abutment	990	1,000

(2) End Shape of Wall

The end shape of the walls was determined taking account of the following:

A) to have a shape to minimize the adverse effect of turbulent flow around a pier

B) to maintain unified shapes in a whole



(a) Hollow Slab (b) P.C Composite Girder (c) P.C Box Girder

Figure 7.17 End Shape of Wall

(3) Ratio of Pier Width to Flow Width in Low Water Channel

Ratio of pier width to flow width in low water channel is estimated as follows;

total pier width (P11 to P23) in low water channel
L1 = 28.0 m
total flow width in low water channel
L2 = 560.7 m

Disturbed Ratio = L1/L2 = 4.99 % < 5.0 %

7.5.3 Summary of Structural Calculation

Sizes of the substructures are to be determined so as to guarantee sufficient strengths to resist the forces which are transmitted from the superstructure, and are directly acting on the substructure.

The sizes are determined after checking stresses for the configurations of the substructure in the preceding paragraphs.

Results of the stress calculations for the representative substructures are shown in Appendix-7.9(1) - (3).

7.6 STUDY ON THE PROPOSED FOUNDATIONS

7.6.1 Study Conditions of Foundations

(1) Constants of Soil Mechanics

The strata at the project site consist of Alluvial layers and the basal rock layer (Nubian formation). The alluvial clay layers lie at a depth of 5-9 m from the ground surface and the alluvial sand layer has thickness of approximately 3 m under the clay layers. The basal rock layer lies under them.

Their constants of soil mechanics are as described in the preceding section 6.3.

(2) Bearing Stratum

As discussed in Subsection 6.3.2, bridge foundations are to be born on the Nubian formation in Figure 6.5.

(3) Ultimate Bearing Capacity of Bearing Stratum

A) In Case of Cast-in-place Pile

As the thickness of clayey soil above the bearing stratum is relatively thin, embedment effect of foundation is thought negligibly small.

Therefore an ultimate bearing strength is estimated as point-bearing capacity given by the following Terzaghi's formula:

$$q_d = a \times C \times N_c = 2,925 \text{ tf/sq.m}$$

where,

q_d : Ultimate bearing strength (tf/sq.m)

a : Shape factor
(in case of circular, a = 1.3)

c : Cohesion of stratum = 50 (ft/sq.m)

N_c : Coefficient of bearing capacity = 45

B) In Case of Steel Pipe Pile

As steel pipe piles cannot be driven sufficiently not less than the pile diameter into the bearing stratum (hard-rock), a plugging effect at the pile tip should be considered for estimated bearing capacity. Subsequently, the ultimate bearing strength for the steel pipe pile is given by the following:

$$q_d' = q_d \times B = 1,250 \text{ tf/sq.m}$$

where,

q_d' : Ultimate bearing strength of steel pipe pile (tf/sq.m)

B : Coefficient for plugging effect at pile tip = 0.428

7.6.2 Foundation Type in The Low Water Channel

(1) Alternative Foundation Types

The classification of foundation types can be described along four different types, as in Figure 7.19. Selection of foundation type depends on many factors such as topography, geological conditions, const-ruction requirements, loading conditions and so on.

The following factors are considered as predominant for the selection of foundation type of the proposed bridge.

A) Topography and Geological Conditions

a) Alluvial layers are slightly soft clay and medium density sand, and contain no gravel.

b) Bearing layer is the sandstone layer with a very gentle slant, which exists about 9 meters below the riverbed.

B) Load Conditions

a) Very heavy loads are applied since the super-structure is PC girders.

b) Magnitude of horizontal loads is very small.

C) Construction Requirements

a) Foundation length becomes approximately 10 m.

b) Most of the foundations will be constructed in the river water.

c) Procurement of imported construction material will require very long transportation, about 1,200 km from Port Sudan to Khartoum.

In this regard, the following four different foundation types were selected for consideration:

a) Precast Steel Pipe Pile
b) Cast-in-place RC Pile
c) Open Caisson
d) Steel Pipe Sheet Pile Caisson

The reasons for discarding other foundation types are as follows:

a) Bearing layer is too deep to adopt spread footings.

b) As a pneumatic caisson requires special facilities and equipment, it is much more costly than an open caisson.

c) Driving of precast RC and PC piles into the sandstone layer would be very difficult.

d) Cast-in-place RC piles by hand excavation are very difficult to construct in the river, and moreover it is almost impossible to penetrate them into the sandstone layer.

(2) Selection of Optimum Foundation Type

Calculation of foundation stability was made for each alternative and followed by a construction cost estimate for the same. The results are shown in Figure 7.18, which shows the cost - size relations by alternative foundation types.

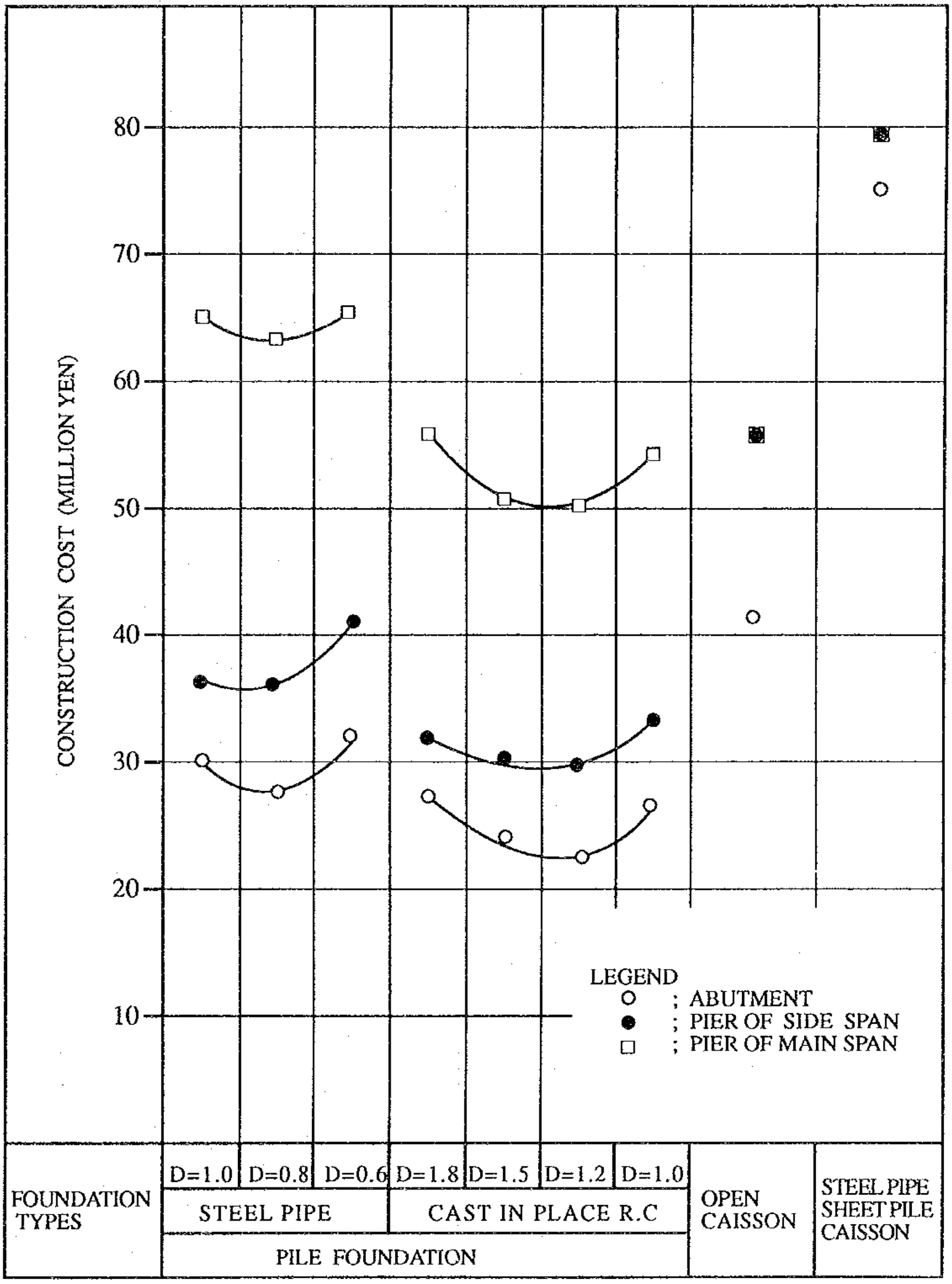
Evaluation was made to guide the selection of foundation type as shown in Figure 7.19. This evaluation is summarized below:

A) Steel Pipe Pile Foundation

Steel pipe pile is evaluated as good on construction aspects, but inferior to cast-in-place RC pile with respect to material procurement, transport and construction cost.

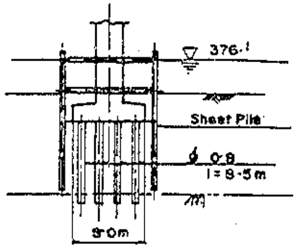
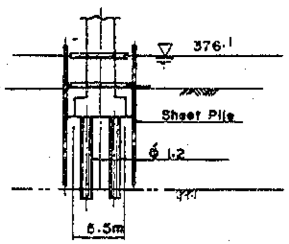
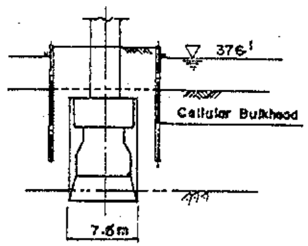
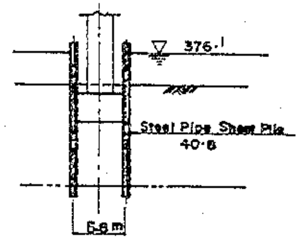
B) Cast-in-place RC Pile Foundation

Cast-in-place RC pile excels others in every aspect. In particular, it has the advantage that penetration of pile tip to the sandstone is very easy.



NOTE;
(1) TEMPORARY JETTY IS EXCLUDED.
(2) D REPRESENTS DIAMETER(m) OF PILES.
(3) YEN 1.0= Ls 0.087

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.18	COST-SIZE RELATION BY FOUNDATION TYPE
	JAPAN INTERNATIONAL COOPERATION AGENCY	

Foundation Type		Steel Pipe Pile	Cast-in-place R.C Pile	Open Caisson	Steel Pipe Sheet Pile Caisson
Configuration of Foundation					
Principal Materials of Foundation		Steel Pipe	Concrete in water Reinforcing bar	Concrete Reinforcing bar	Steel pipe sheet pile Concrete Reinforcing bar
Materials Procurement and Transport		× Many imported materials are required, and transport of steel pipe is not efficient	○ Many materials can be procured around Khartoum, and cement and re-bars are good for transport conditions.	○ Same as Cast-in-place R.C Pile	× Same as Steel Pipe Pile
Construction Aspect	Construction Methode	Pile driving on the jetty by pile driving machine. Dry construction by single sheet pile cofferdam	Same as Steel Pipe Pile	Construction of cellular bulkhead with sheet piles and sand. sinking and excavation of caisson	Driving steel pipe sheet pile with both functions of foundation and cofferdam.
	Construction Period	○ 19.5 months	○ 19.3 months	× 35.0 months	○ 19.8 months
Economic Aspect (Ratio)		△ 1.212	○ 1.000	× 1.742	× 2.522
Overall Evaluation		Not Recommended	Recommended	Not Recommended	Not Recommended

Note: Judging marks ○ ; Good △ ; Fair × ; Bad

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE

Fig. 7.19 EVALUATION OF ALTERNATIVE FOUNDATION TYPES

JAPAN INTERNATIONAL COOPERATION AGENCY

C) Open Caisson Foundation

Despite open caisson being one of the most stable types, it is inferior in construction aspects and from an economic viewpoint.

D) Steel Pipe Sheet Pile Caisson Foundation

Steel pipe sheet pile is the most advanced foundation type and much more advantageous where the foundation is constructed by an off-shore construction method. In the case of the White Nile, however, the water depth is very shallow, at most 2.5 meters in the dry season. Therefore, the merits of a steel pipe sheet pile caisson cannot be enjoyed in this case. Construction cost becomes higher than others.

As a result, cast-in-place RC pile foundation is recommended. A diameter of 1.2 meters will be adopted the most economical.

7.6.3 Foundation Type for the Omdurman Side Viaducts

According to the results of the preceding Paragraph 7.6.2, it is obvious that cast-in-place RC piles are the most suitable foundation type even where foundation work, which might be carried out in dry season, is not affected by the river water. In this regard, cast-in-place RC piles are recommended for the foundations of the viaduct. The reasons are as follows:

a) The proposed bridge will be located on a reliable bearing stratum and may not be affected by severely strong horizontal forces such as earthquake. In this case, the sizes of foundation will be dominated mainly by the vertically acting forces, and cast-in-place piles are a favorable foundation type to support the superstructure weight.

b) A caisson foundation requires an extremely large size section, which is required for workability of equipment or machinery, and becomes costly.

The diameter-cost relation diagram is shown in Figure 7.17, from which a diameter of 1.2 meters is recommended as the most economic one. Piles are to be arranged in one row as shown in Sheet No.17/32 and 23/32 of the Drawings. However, it should be noted that pile arrangement, whether it is in one row or two rows, be re-considered in the detailed design stage especially for the piers supporting the fixed bearings.

7.6.4 Connection between Foundation and Footing

Piles will be connected rigidly with the pile-cap to transmit the vertical and horizontal forces from the superstructure to the pile foundation as shown in Figure 7.20.

7.6.5 Summary of Stability Calculation

The numbers of piles are to be determined to satisfy the following conditions:

a) The bearing stratum can withstand the vertical forces from the superstructure and the substructure transmitting through the pile foundation.

b) The horizontal displacement at the top of the piles does not exceed the allowable value.

c) The pile foundations have sufficient strength for the forces acting.

The results of the stability calculations of the representative substructure of each superstructure are shown in Appendix (1) - (3).

In the preceding conditions, the allowable horizontal displacement and the allowable bearing capacity of pile are based on the following ideas.

A) Allowable horizontal displacement

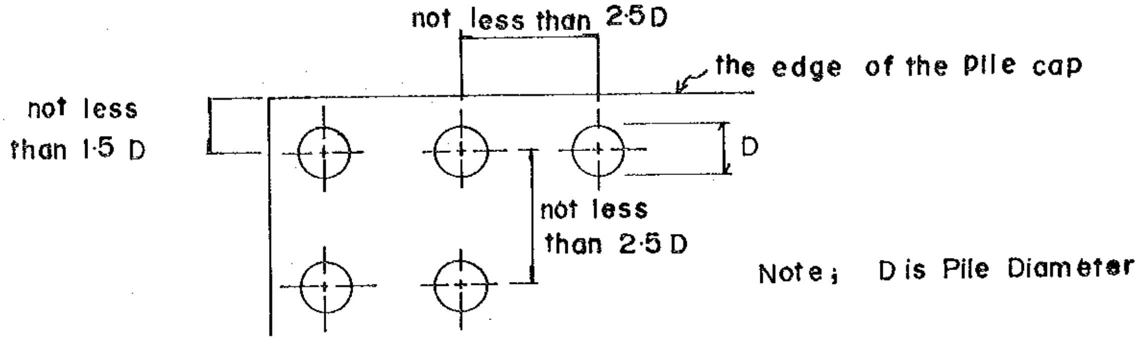
It is said that soils have both an elastic and a plastic property. In case of the very small displacements, the elastic property is superior to the other.

It is desirable that the displacement of piled foundations is small. Therefore, an allowable horizontal displacement of 1.0 centimeter is adopted.

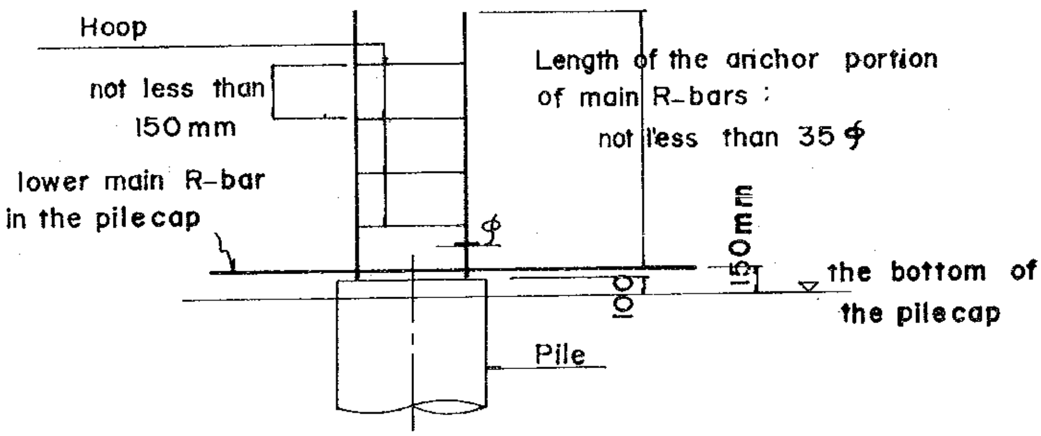
B) Allowable bearing capacity of pile

The allowable bearing capacity of a pile is to be determined by the geological conditions and the stresses of the pile itself.

An allowable bearing capacity of a pile, determined by the geological conditions, is calculated from the sum of the skin friction forces and the point-bearing strength. In the case of a very short pile length, the skin friction forces can be neglected. Therefore, an allowable bearing capacity of a pile is at 1,100 tf according to JRA standards.



Center-to-center Spacing of Piles



Details of Connection of Pile Head and Pilecap

THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE NEW WHITE NILE BRIDGE	Fig. 7.20	PILECAP ARRANGEMENT
	JAPAN INTERNATIONAL COOPERATION AGENCY	

On the other hand, the allowable bearing capacity governed by fiber stresses in pile sections is obtained at 414 tf also according to JRA standards.

Consequently, the allowable bearing capacity is determined at 414 tf per one pile.

7.7 SUMMARY OF QUANTITIES

The quantities of the substructure and superstructure are computed based on the preliminary design drawings and shown in Table 7.8.

Table 7.8 Work Quantities of Bridge Structure

Description	Quantities
1. Substructure & Foundation	
1.1 Cast-in-place concrete pile	2,930 m
1.2 Cofferdam	14 No.
1.3 Abutments	2 No.
1) Concrete Class A	1,030 cu.m
2) Reinforcing bar	50 tf
1.4 Piers	24 No.
1) Concrete Class A	12,910 cu.m
2) Concrete Class P	640 cu.m
3) Concrete Class C	270 cu.m
4) Reinforcing bar	690 tf
1.5 Foundation Stone Boulder	780 cu.m
2. Superstructure	
1.1 PC Box Girder Bridge	172 m
1) Concrete Class P	4,010 cu.m
2) Reinforcing bar	380 tf
3) Prestressing PC Cable	240 tf
1.2 PC I-Girder Bridge	434.4 m
1) Concrete Class P	7,430 cu.m
2) Reinforcing bar	600 tf
3) Prestressing PC Cable	440 tf
4) Erection of PC-I-Girder	156 No.
1.3 RC Follow Slab Bridge	6.04 m
1) Concrete Class A	2,210 cu.m
2) Reinforcing bar	430 tf
1.4 Expansion Joint	170 m
1.5 Rubber shoes	255 No.
1.6 Handrailing	1,500 m
1.7 Asphalt wearing surface	13,250 sq.m