CHAPTER 6

PRELIMINARY ENGINEERING DESIGN

6. PRELIMINARY ENGINEERING DESIGN

6.1 Road Design

6.1.1 Road Design Conditions

(1) Applicable Design Standard

1) Road Design Manual in Uganda

Uganda has a road design manual, which is composed of four (4) sets of volume; Geometric design manual, Hydrology and hydraulics design manual, Pavement design manual and Bridge design manual. The Geometric design manual is a part of the revised and developed version of the road design manual, which was published in 1994. The manual has been used by all road planners and engineers in designing rural roads in Uganda. For urban roads, only very limited description is available. In addition, it is noted that the manuals are only general rules so that some modifications may be required in cartain special cases.

2) Other Manuals

Southern Africa Transport and Communications Commission (SATCC) prepared documents of codes of practice consisting of Geometric, Pavement, Bridge and Rehabilitation Design for the road design work in SADC countries (Southern Africa Development Community; Tanzania, Zambia, Botswana, Mozambique, Angola, Zimbabwe, Lesotho, Swaziland, Malawi , Namibia, Malicious, Congo and Madagascar and S.A.) .. The SATCC standard for road design is commonly adopted even for countries which do not belong to SADC. It is also recognized that the above-mentioned Ugandan road design manuals were prepared based on SATCC standards so that it is applicable for the road design of the study road if necessary. In addition AASHTO which has been recognized as the most popular and authorized design standard for road design can also be used in some cases.

(2) Road Classification

1) Functional Class

Since there is no urban road standard in Uganda, road classifications were prepared for rural roads only. The rural roads in Uganda are divided into the following 5 classes according to their major function for the road networks.

Table 6.1.1 Functional Class

Class	Roads	Function
A	International Trunk Road	Roads that link International Important Centers. Connection between the national road system and those of neighboring countries. Major function is to provide mobility
B	National Trunk Road	Roads linking provincial capitals, main population centers and nationally important centers. Major function is to provide mobility
C	Primary Roads	Roads linking essential provincial centers to a higher class road (urban/rural centers). Linkage between districts local population centers and development areas with higher class road. Major function is to provide both mobility and access
D	Secondary Road	Roads linking locally important centers, to more important centers, or to higher class roads (rural/market centers) and linkage between locally important traffic generators and associated rural hinterland. Major function is to provide both mobility and access.
E	Minor Road	Any road link to minor center (market/local center) and all other motorable roads. Major function is to provide access to land adjacent to the secondary road system.

Source: Geometric Design Manual in Uganda

2) Design Class

In addition to the Functional Class there is a Design Class which is divided into 7 classes, as follows,

Table 6.1.2 Design Class

Design Class	Capacity [pcu x 1,000/day]	Road-way width[m]	Maximum Design speed Km/h			Functional Classification				
			Level	Rolling	Mountain ous	A	B	C	D	E
Ia Paved	12 – 20	20.80-24.60 (4-lane)	120	100	80	✓				
Ib Paved	6 – 10	11.0	110	100	80	✓	✓			
II Paved	4 – 8	10.0	90	70	60	✓	✓	✓		
III Paved	2 – 6	8.6	80	70	50	✓	✓	✓		
A Gravel	4 – 8	10.0	90	80	70		✓	✓	✓	
B Gravel	2 – 6	8.6	80	60	50				✓	✓
C Gravel		6.4	60	50	40					✓

Source: Table 4-2a: Road Design Classes, Geometric Design Manual in Uganda

(3) Design Criteria for the Study Road

1) Basic Policy

The study road is part of the Northern Corridor connecting Mombasa, Kenya and the land locked countries in EAC including DR. Congo, Burundi, Rwanda and Uganda. In this view the study road clearly falls into Class A (International Trunk Road) for the functional class which can be classified into either Ia, Ib II or III paved road as the design class.

Seemingly the study road area is identified as urban or semi- urban area with industrial centers as well as residential areas.

Given these conditions, the following considerations were made:

- The existing bridge is one of the bottle necks along the Northern Corridor and causing some obstacles for regional development, therefore it is predicted that development along the corridor will be stimulated after the new bridge opens.
- The design of the study road needs to satisfy international road standards, though its adjacent sections are recognized to lie in semi-urbanized area
- There is a project called the Rehabilitation of the Bugiri-Jinja Road, of which the design class is Ib Paved Road. The project commenced implementation as in November 2008 and that project should also be considered because it is a continuation of the study road.

The study road lies in the international trunk route. However, it is not expected to realize a high mobility in this road section, since three roundabouts and abridge are planned for short distance of 2.3 km only. Thus, it is not suitable to apply a high design standard of Ia Paved Road to the study road.

Consequently it was decided that the study road falls into Ib Paved Road which satisfies international road design standard and is in accordance with the design class of the Bugiri-Jinja Road, by the application of the design parameters shown in Table 6.1.3.

Table 6.1.3 Design Parameters

Parameters	Unit	Applied	AASHTO	SATCC		Uganda			Remarks
Design Speed	km/h	**80**	100-120	80	100	80	100	120	-
Design Vehicle	m	**Semi trailer combination large* W=2.6, L=16.7, H=4.1**	-	WB-15 (Semi-Trailer) W=2.6, L=17, H=4.1		Semi trailer combination large* W=2.6, L=16.7, H=4.1			*Second largest vehicle (DV-4) Maximum legal freight is 4.0m (refer to 5.1 in the design manual
Lane Width	m	**3.5**	3.6	-	-	3.5*	3.5*	3.65*	Less relationship to the design speed
Shoulder	m	**2.0**	Min 1.25	-	-	2.0	2.0	2.5	-
Min .R. of Horizontal Curve	m	**240**	280*	250	400	240	415	710	*In case of 4% of crossfall
Min. Curve Length	m	-	Not specified	300 (absolute 150)		Not specified			-
Min. R. of Curve for omitting Transition	m	**1200**	-	-		1200*	2300*	4000*	*R < (Design Speed)^3 / 432 : Rounded
Stopping Sight Distance	m	**115**	110	115	155	115	160	205	-
Max. Grade	%	**6.0**	4.0	5.0% (Flat)	4.0% (Flat)	6.0-8.0	4.5-6.5	3.0-4.0	-
Critical Length of Steep Section	m	-	-	240	300	Not specified			5% for 450m
Min. R. of Sag Curve	m	**2500**	2600	2500	3600	2500*	3700*	5000*	*Assumed from stopping sight distance
Min. R. of Crest Curve	m	**2100**	2600	3300	6000	3200*	8000*	10300*	*Assumed from stopping sight distance
Max. Superelevation	%	**4.0**	-	-	-	7.0 (4.0)*	7.0	7.0	*4.0% is applied for urban area.
Pavement Crossfall	%	**2.5**	-	-	-	2.5	2.5	2.5	-
Height Clearance	m	**5.0**	-	-	-	5.0			
Right of Way	m	-	-	-	-	50-60	60	60	-

Source: JICA Study Team

2) Design Speed

The design speed of 80 km/h will be adopted, which is the most important factor to be considered for the road design. This selection is in accordance with the design requirement of Ib Paved Road. The selection is considered as relatively lower in service level. However paying attention to safety is important because of the predicted urbanization/development of the hinterland as discussed above.

6.1.2 Preliminary Road Design

(1) Geometrical Design

1) Horizontal Alignment Design

The following design principles were considered for the horizontal alignment setting.

- Compliance with the required parameters of the adopted design speed,
- Integration with land-use plans,
- Balancing the size and length of curves,
- Balancing the horizontal , vertical and cross-sectional parameters,
- Visibility,
- Method of construction,
- Constraints from natural condition such as terrain, geological features and existing property,
- Minimization of social impact,
- Cost effectiveness and benefit,

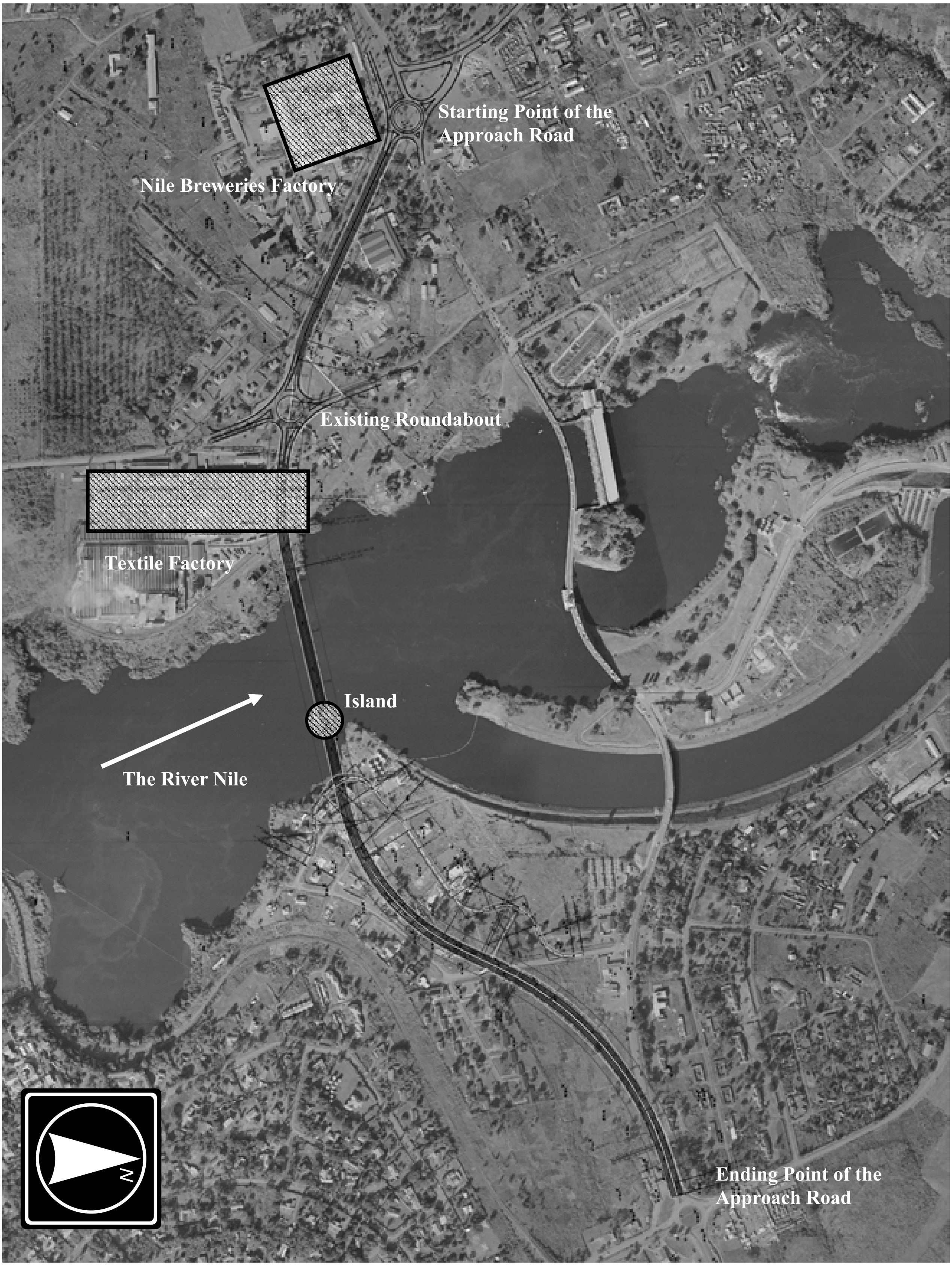
The horizontal alignment setting shall take into account mobility, safety and comfort derive from balanced design parameters.

The horizontal alignment starts from the existing junction in front of Nile Brewery Factory in Njeru Town. The alignment follows the exiting road up to the roundabout in front of the Textile Factory and then continues toward River Nile. Some property of the Textile Factory will be affected by the alignment. However the properties to be affected are the administrative office and canteen which are not in used for actual production so that the impacts on social aspects and regional industrial development are considered to be minor. The affected properties can be relocated by compensation in accordance with Ugandan regulations.

Across the River Nile, the alignment is set in such a way to pass over the existing small island in the River to achieve a favorable bridge span configuration considering cost effectiveness.

The alignment of the study road runs through free land at the right bank of the River Nile with the exception of small houses and terminates at the existing Nalufenya roundabout. Upon the approach to the terminal point, approximately 200m of the stretch will overlap with the existing road.

The horizontal alignment design is outlined in Figure 6.1.1



Source: JICA Study Team

Figure 6.1.1 Outline of Horizontal Alignment Design

2) Vertical Alignment Design

The following design principles were considered for the vertical alignment setting.

- Compliance with the required parameters of the Ugandan Standard
- Harmonization with horizontal alignment
- Minimizing of vertical change

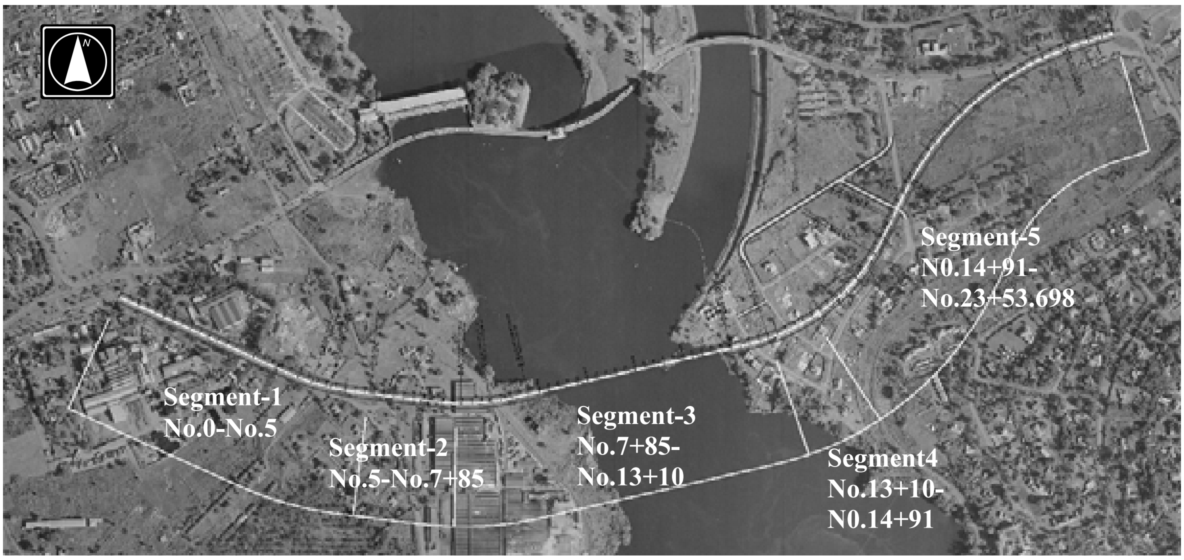
In addition to the above, the following conditions were taken into account for the vertical alignment design.

- Since there are some existing facilities at the Njeru Town side the existing vertical alignment is maintained in order not to affect the existing accessibilities.
- The aviation limit (the level of 1,216.0 m)should not be violated by the road facilities including the tower of the cable-stayed bridge.
- The navigation clearance (the level of 1,140.0 m) should not be violated by the road facilities in the bridge section.
- Since there is black cotton soil at the right bank of the River Nile minimum earth replacement of 1.0 m is considered for the vertical alignment design.

3) Typical Cross Section Design

The cross sectional designs are detailed in this Preliminary Engineering Design in consideration of the terrain, land use features and constraints to social environmental impacts (i.e. resettlement) bringing about various typical cross sectional designs

This section describes the explanation for the design of the cross sections by segments commencing from No.0 (starting point)-No.5, No.5-No.7+85, No.7+85-No.13+10 and No.13+10-No.14+91 and No.14+91-No.23+53.698 (terminal point)



Source: JICA Study Team

Figure 6.1.2 Segments for Cross Section

<u>No.0- No.5 (Segment-1):Njeru Town</u>

In this segment there are some industrial facilities contributing to some economical impacts on Njeru Town so that the cross section have been designed to minimize affected areas and compensation costs for the ROW. The cross section have been designed to the extent possible within the available existing space.

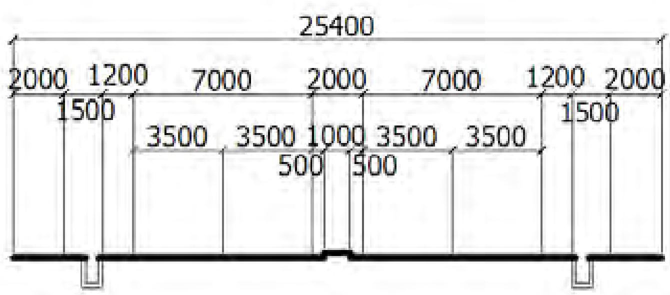
The lane was designed to be 3.5 m wide in accordance with the standard width of Ib Paved Road and the number of lane is two (2) which is the result of future traffic volume forecast. The median width is designed at 2.0 m inclusive of the inner shoulder of 0.5 m at both sides based on the minimum requirement of the Ugandan Standard and the shoulder width is designed to be 1.2 m which is also in accordance with the minimum requirement of the said Standard.

Consequently the total width per direction composed of the carriageway and the shoulders for a total of 8.7 m allowing three (3) vehicles to pass in parallel. In other words, the sectional width allows the use of at least 2-lane space, so as not to hinder traffic if there is a broken-down vehicle on the road.

A drainage space 1.5m wide is provided on both sides taking into account the existing drainage system.

The footways will be provided on both sides at 2 m wide in accordance with the requirement of Ugandan Standard.

As the result, the total width of the road for this segment is 25.4 m as shown in Figure 6.1.3.

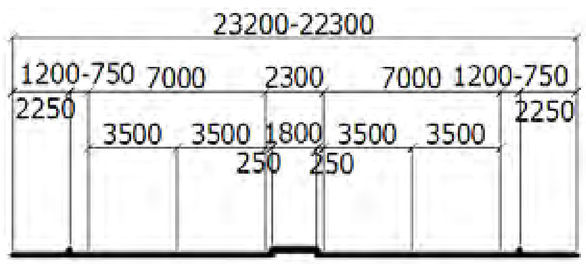


Source: JICA Study Team

Figure 6.1.3 Cross Section for Segment-1

<u>No.5-No.7+85 (Segment -2): Bridge Approach-1</u>

This segment is the transition section from Segment 1 (Njeru Town) to Segment 3 (Bridge) section. The outer shoulder width varies from 1.2 m to 0.75 m within this segment. Median and Footway width differ from those of Segment-1 at 2.3 m and 2.25 m respectively. Hence the total width for this segment varies from 23.2 m to 22.3 m as shown in Figure 6.1.4.



Source: JICA Study Team

Figure 6.1.4 Cross Section for Segment-2

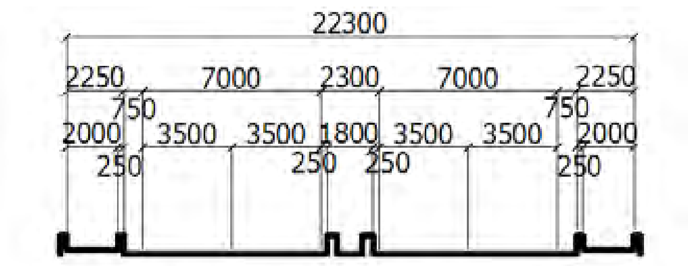
No.7+85-No.13+10 (Segment -3): Bridge

This segment is the section for the bridge. Since the bridge construction cost is huge and the determining factor on the project feasibility, the design for each element of the cross section should be economical and effective by observing minimum design requirements.

The lane width is maintained at 3.5 m. However, the outer shoulder width was reduced to 0.75 m complying with above-mentioned design policy. Median width is set at 2.3 m to provide the necessary space for the cable of the bridge.

Initially, the footway on the down-stream side was considered unnecessary, because the existing bridge is proposed to exclusively allow bicycle and pedestrian passage after the New Nile Bridge opens. However, it is recognized that there still exists a potential of generating pedestrian and bicycle traffic from the down-stream side to use the New Nile Bridge based on future land use development. In consideration of that the above,, the footway was thus provided with design width of 2.25 m including space for safety facility on both sides, hence maintaining 2.0 m of effective space for pedestrians and bicycles.

Consequently, the cross section width of this segment is designed at 22.3 m as shown in Figure 6.1.5.

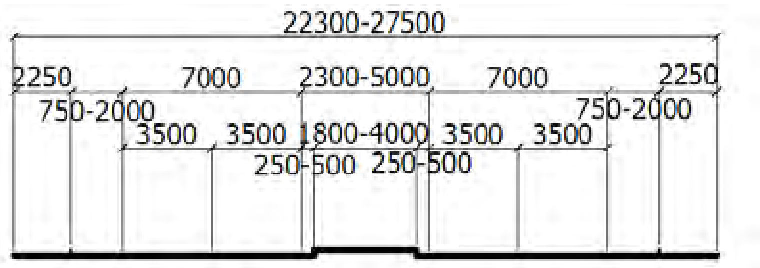


Source: JICA Study Team

Figure 6.1.5 Cross Section for Segment-3

No.13+10-No14+91 (Segment -4): Bridge Approach -2

This segment is the transition section from Segment 3 (Bridge) to Segment 5 (Jinja City). The outer shoulder width varies from 0.75 m to 2.0 m within this segment and the median width also varies from 2.3 m to 5.0 m, depending on land uses along the segment. Hence the total width for this segment varies from 22.3 m to 27.5 m as shown in Figure 6.1.6.



Source: JICA Study Team

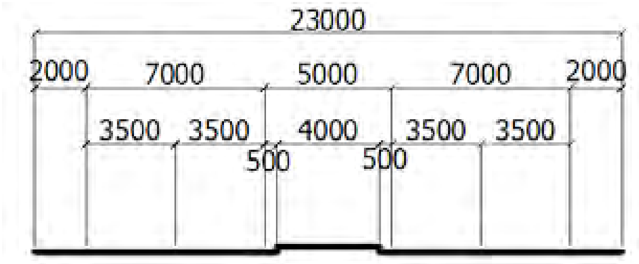
Figure 6.1.6 Cross Section for Segment-4

No14+91-No.23+ 53.698 (Segment -5): Jinja City

The cross section for this segment complies with the standard requirement of Paved Ib Road and it also complies with that of Jinja - Bugiri road as the continuation of the project road.

Footway was not provided but shoulders are provided for use of pedestrians and bicycles as stated in the Ugandan design standard.

The total width of the section is designed at 27.5 m as shown in Figure 6.1.7.



Source: JICA Study Team

Figure 6.1.7 Cross Section for Segment-5

(2) Pavement Design

1) Design Traffic

Applicable Design Standard

MOWT has prepared the design standard documents related to pavement design, published in July 2005 comprising as follows:

- Road Design Manual Vo.3: Pavement Design,

  Part I: Flexible Pavement;

  Part II: Rigid Pavement;

  Part III: Gravel Roads;

  Part IV: Pavement Rehabilitation Guide

The study road was designed to adopt flexible pavement (i.e. Asphalt Pavement) in consideration of durability, availability of material, ease of maintenance, cost performance and current trend of pavement method in Uganda, so that the pavement design shall mainly follow Part I, Flexible Pavement (hereinafter referred to as "Pavement Standards"); and other standards shall be referred to if necessary.

Design Procedure

Part I, Flexible Pavement is based on the methods provided by "SATCC Practice for the Design of Road Pavements"

SATCC provided the design procedure as shown in Figure 6.1.8:

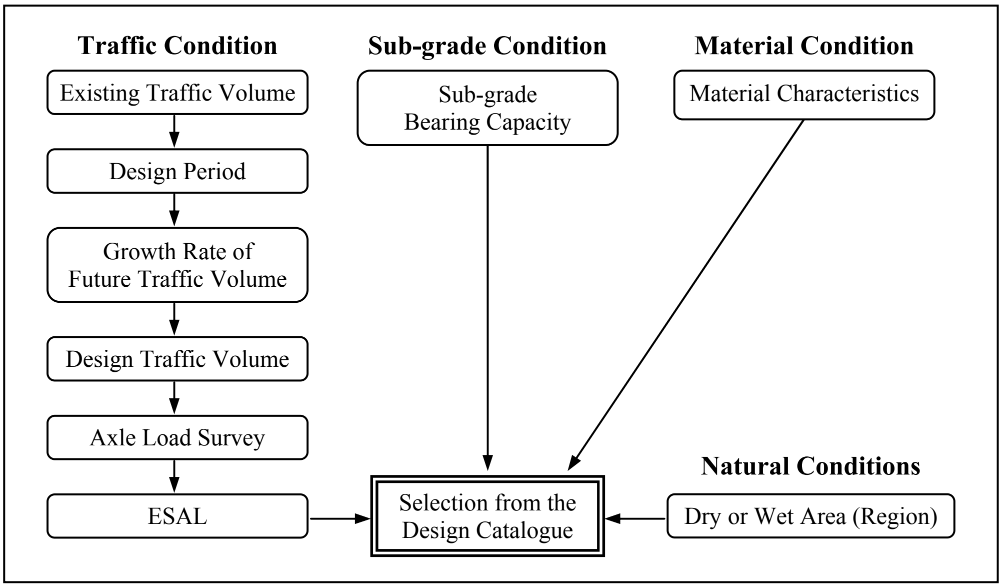


Figure 6.1.8 Flowchart for the Pavement Design based on SATCC Standard

<u>**Pavement Design Life**</u>

The design life of a road will have a large impact on the design specifications of the pavement structure and is therefore important to decide on an appropriate service life period. Usually, 10-, 15-, or 20-year period is adopted for the selection of an appropriate design life depending on the unique circumstances of the individual project. Table 6.1.4 provides some guidance on the selection based on the SATCC standard.

Table 6.1.4 Pavement Design Life Selection Guidance

		Importance/Level of Service	
		Low	High
Design Data Reliability	Low	10 – 15 Years	15 years
	High	10 – 20 Years	15 – 20 Years

Source: SATCC

For "long" design life, the initial capital cost may be huge for the road to be durable against forecasted cumulative traffic loadings over a lengthy period of service time. On the other hand, maintenance and rehabilitation costs would be lower in the long-run. Usually, a balance between initial and future investment costs is considered important. Another consideration is that there are uncertain factors with the use of a long design period, such as the difficulty in forecasting traffic over extended periods of time, especially in the case of unclear socio-economic trends. Such a situation can lead to over-design and misallocation of resources.

The study road is located in an area with no periodic traffic data being available and runs parallel to an existing railway line. In such cases, potential for making significant errors in long-term traffic forecasting is significant. Given this condition, a 15-year period is recommended for the design life, and which requires necessary maintenance and/or rehabilitation to be carried out based on the monitoring of traffic conditions to minimize the risk of over investment.

Traffic Class

The Pavement Standards provided the traffic class based on expected cumulative ESA's as shown in Table 6.1.5.

Table 6.1.5 Pavement Design Life Selection Guidance

Traffic Class Designation								
Traffic Ranges (million ESAs)	T1	T2	T3	T4	T5	T6	T7	T8
	<0.3	0.3-0.7	0.7-1.5	1.5-3	3-6	6-10	10-17	17-30

Source: Road Design Manual Vo.3: Pavement Design,

2) Equivalent Factors

Since there is no axle load survey data nearby the study road, equivalent factors were applied as those of Bugiri-Jinja Road design as follows:

- Bus: 0.56
- 2 axle truck: 4.2
- 3 axle truck: 11.3
- Semi trailer: 15.8
- Truck trailers: 22.4

3) Cumulative Equivalent Standard Axles

The traffic forecast for 2015, which is the expected year for the opening of operations, was carried out for this study. On this basis, the Cumulative Equivalent Standard Axles was estimated by the following formula:

$$DT = T \times 365 \times \frac{(1 + r/100)^p - 1}{r/100}$$ ......Equation 1

where:

DT is the cumulative design traffic in a vehicle category, for one direction, and

T = average daily traffic in a vehicle category for the first year (one direction)

r = average assumed growth rate, per cent per annum

p = design period in years (15 years)

Table 6.1.6 Factors for ESA's

	Mini Bus	Large Bus	Light Truck	Medium Truck	Heavy Truck	Semi Trailer	Truck Trailer	Total
T (2015)	3,827	231	485	1167	941	317	1,436	-
r (%)	2.6	4.4	6.5	6.5	6.5	5.0	5.0	-
EF	0.56	4.20	0.56	4.20	11.30	15.80	22.40	-
ESA's	1,063,665	671,298	254,663	4,598,925	9,967,851	3,777,895	24,284,147	18.6E+06

Source: JICA Study Team

Table 6.1.6 shows the estimation of ESA's for 15 years to be 18E+06, which is classified as T8 and eventually, it is compatible with that of Bugiri-Jinja Road.

4) Subgrade Design

In the study, CBR tests were carried out and the results are tabulated in Table 6.1.7.

Table 6.1.7 CBR Test Results

Name of Sample	Approx. Chainage	Specific Gravity (g/cu.cm)	MDD (g/cu.cm)	OMC (%)	PI (%)	CBR Value 95% of MDD (%)
A-1-A	No.8 LHS	2.52	1.48	23.0	42.0	5
A-1-B	No.8 LHS	2.51	1.43	24.0	53.0	9
A-2-A	No.13 LHS	2.50	1.52	25.0	45.0	8
A-2-B	No.13 LHS	2.57	1.56	22.0	41.0	4
A1-A	No.8 LHS	2.58	1.46	26.0	47.0	4
A2-A	No.13 RHS	2.50	1.62	20.0	32.0	6

Source: JICA Study Team

As shown above, the CBR values are relatively low and the PI values are extremely high, which suggest that the soils are problematic. In fact, the existence of the so called "Black Cotton Soil" was observed during the field survey.

Black Cotton Soil is well-known as expansive soil in Eastern Africa. Expansive soil exhibits particularly a large volumetric change (swell and shrinkage) following variations in their moisture contents. Expansive soil will be assessed also when it occurs below design depth. The chosen measures to minimize or eliminate the effect of expansive soil will be economically realistic and proportionate to the risk of potential pavement damage and increased maintenance cost.

There are many desk studies for measurement against problems caused by Black Cotton Soil but the replacement of the soil is considered most effective and applicable. The depth and extent vary depending on expansiveness of the soil. As for the determination of the expansiveness, many technical references are available. As a consequence, the Study Team recommended reference to the Pavement Materials Design 1999 MOW, Tanzania that gives clear indications for survey, design and construction method for the soil. Thus, the Tanzanian Standard is preferred for the subgrade design for the detailed design stage.

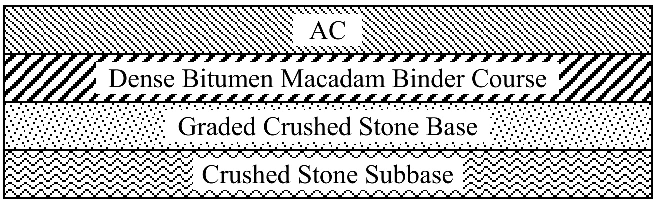
In this Study, 1 m of replacement with selected borrowed soil is designed as the measure against the Black Cotton Soil: The subgrade value depends on the quality of borrow material which requires more than CBR 15% according to the Ugandan Standard. Hence the subgrade design value should fall into CBR 15%.

5) Design Pavement Layer Thickness

According to the field survey pavement material such as borrow (subgrade), subbase, base and aggregate for AC are available near the New Nile Bridge. Bituminous material is normally imported from outside Uganda and its discharging port is Mombasa, Kenya.

As described previously, the Bugiri-Jinja Road is the continuation of the study road. In consideration for ease of maintenance, it is preferable that the pavement material (types) for the study road and Bugiri-Jinja Road will be the same.

The Bugiri-Jinja Road applied the pavement composition as shown in Figure 6.1.9.



Source: JICA Study Team,

Figure 6.1.9 Pavement Composition for Bugiri-Jinja Road

The following equation was employed for the design of pavement layer thickness as specified in the Ugandan Standard.

$$DSN = (a_1h_1+a_2h_2+a_3h_3+a_4h_4) / 25.4$$

where:

DSN = Weighted Design Structural Number for entire Pavement Structure:3.8 (T8, CBR15)

a_1, a_2, a_3, a_4 = Structural Number Coefficient for the Wearing Course, Binder Course, Base Course and Subbase Course

h_1, h_2, h_3, h_4 = Layer Thickness for the Wearing Course, Binder Course, Base Course and Subbase Course in mm

Given this condition, the pavement design thickness for the study road is obtained as shown in Table 6.1.8.

Table 6.1.8 Pavement Design Thickness

Material	Layer Coefficient (L)	Thickness (T) mm	L x T
AC	0.35	60	21.0
Dense Bitumen Macadam Binder Course	0.20	150	30.0
Graded Crushed Stone Base	0.14	175	24.5
Crushed Stone Subbase	0.11	200	22.0
		Σ (L x T) / 25.4 =	3.8

Source: JICA Study Team

(3) Drainage Design

1) Design Principle

Proper drainage system is considered as a key factor to attain sustainable and stable traffic flow. The arrignment shall be made by considerations of not only satisfaction of run-off estimation results but also maintenance workability. In addition, information and opinions which are available locally will be considered for the design. As previously described, the construction of Bugiri-Jinja Road Project is still underway and its design is not outdated, so that its design result will be used as reference to the maximum extent in designing the study road.

In preparing the optimum design, the cost effectiveness is also considered in addition to the above design principles.

2) Applicable Design Standard

For the drainage design, "Road Design Manual Vol.2: Drainage Design" is generally applied to national roads in Uganda. The design standard consists of 10 sections which cover all necessary drainage design exercise.

3) Flood Flow Hydrology

In the Bugiri-Jinja Road two (2) methods were studied and compared, which are the TRRL and the Richards (Rational) method

TRRL method is specifically developed for small to medium catchments (approx. 200 sq. km) in East Africa by Transport and Road Research Laboratory (TRRL) UK and the various East African Governments. This method has been used extensively in East Africa for various river/stream catchments that are within the size limitation of the original model. The model considers two phases of storm run-off; namely the period between the rains hitting the ground surface and flowing into a water course and passage of the flood down the water course to catchments outfall. That run-off does not occur uniformly over a catchment and that some parts of a catchment area are less permeable than others due to variation in soil type. This fact is also taken into consideration for comparison purposes.

Prior to the development of the TRRL method, the Rational Method was widely used in East Africa for a wide range of catchments and is still in use on catchments larger than those covered by the TRRL method. The method is reported to give consistently good results for rivers in East Africa and hence is commonly used in flood studies. The method takes account of the rainfall pattern and intensity, the size, shape of slope of the catchment and run-off characteristics in the form of a run-off-coefficient K.

As the result of comparative analysis, the Rational Method was employed in this study.

Additionally, the Rational Method is considered appropriate, based on the analysis result of the Bugiri-Jinja Project.

4) Drainage Facility Design Requirement

In the Drainage Design Standard, Design Average Recurrence Interval for Flood/Storm (Yrs) by Geometric Design Standard is regulated as shown in Table 6.1.9.

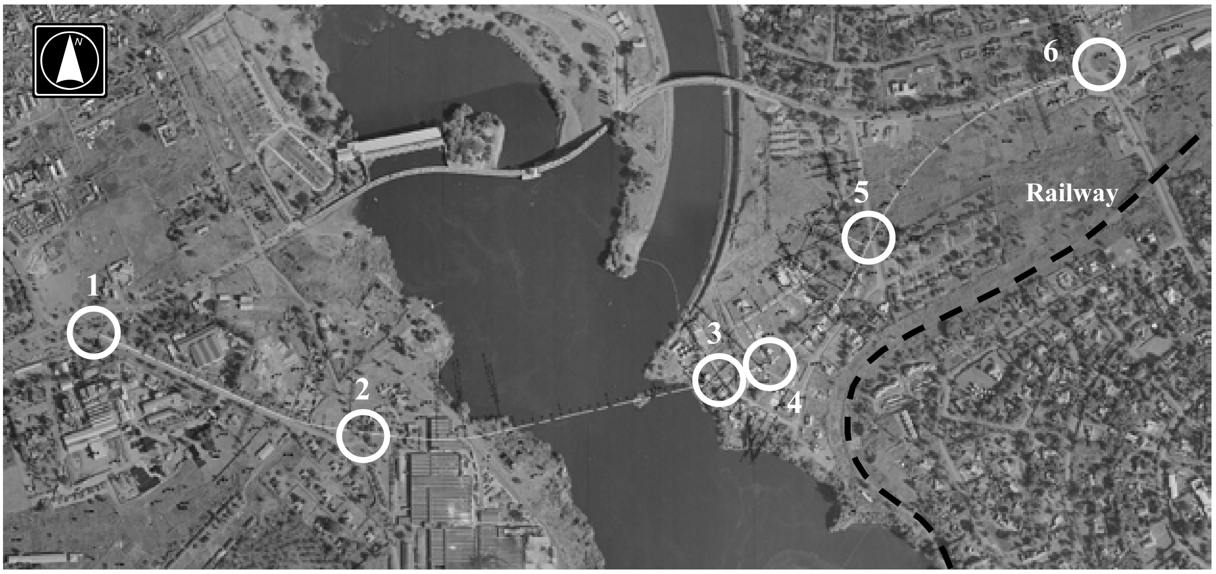
Table 6.1.9 Design Year for Drainage Facilities

Structure Type	Geometric Design Standard			
	PIa, PIb	PIII Gravel A	PIII, Gravel B	Gravel C
Gutters and Inlets	10/5	2	2	-
Side Ditches	10	10	5	5
Ford/Low-Water Bridge	-	-	-	5
Culvert Pipe (see Note) Span < 2m	25	10	5	5
Culvert 2 m < Span < 6 m	50	25	10	10
Short Span Bridges 6 m < Span < 15 m	50	50	25	25
Medium Span Bridge 15 m < Span < 50 m	100	50	50	50
Long Span Bridge Span > 50 m	100	100	100	100
Check/Review Flood	200	200	100	100
PIa=Paved Ia, PIb=Paved Ib, PII=Paved II, PIII=Paved III				

Note 1: Span in the above table is the total clear-opening length of a structure. For example, the span for a double 1.2-meter diameter pipe is 2.4 meters, and the design storm frequency is therefore "culvert, 2m < span < 6m." Similarly a double box culvert having two 4.5-meter barrels should use the applicable design storm frequency for a short span bridge and a bridge having two 10-meter spans is a medium span bridge.
Source: Road Design Manual Vol.2: Drainage Design

(4) Intersection Design and Feeder Road

Based on the selected project alignment, six (6) intersections with the existing road network are identified as shown in Figure 6.1.10.



Source: JICA Study Team

Figure 6.1.10 Intersections with Existing Road Network

Njeru Town

On the Njeru side, there are two (2) points and they are intersection points of the roundabout type under the existing conditions. There are two (2) options for the intersection type which are the junction and the roundabout. Comparisons were made so as to determine the suitable type of intersection for each point as shown in Figure 6.1.11, 6.1.12 and Table 6.1.10, 6.1.11.

Option-1:Junction

Option-2:Roudabout

Source: JICA Study Team

Figure 6.1.11 Comparison of Intersection Types at No.1

Table 6.1.10 Comparison of Intersection Types at No.1

Item	Junction	Roundabout	Remarks
High Mobility	Cursing speed of 80 km/hr is possible	Cursing speed is forced to reduce to be less than 30km/hr	Design Speed is 80km/hr
	++	+	
Safety	Signalization is required. The cursing speed is not reduced when the signal is blue.	No special treatment is required. The traffic is forced to reduce speed that makes increase safety.	
	+	+++	
Local Access	Service road shall be considered.	Existing condition remains unchanged	
	+	++	
Resettlement	3 nos. of EL pole shall be resettled	2 nos. of EL pole shall be resettled	
	+	++	
Maintenance	In addition to the routine maintenance signal maintenance is required and it is considered to be expensive..	Routine maintenance is required.	
	+	++	
Construction Cost	Pavement area is wider than roundabout and other facilities cost is needed.	Lower than Jct.	Incl . resettlement cost
	+	++	
Total (Score of +)	7	12	

Source: JICA Study Team

Option-1:Junction

Option-2:Roudabout

Source: JICA Study Team

Figure 6.1.12 Comparison of Intersection Types at No.2

Table 6.1.11 Comparison of Intersection Types at No.2

Item	Junction	Roundabout	Remarks
High Mobility	Cruising speed of 80 km/hr is possible	Cursing speed is forced to reduce to be less than 30km/hr	Design Speed is 80km/hr
	++	+	
Safety	Right turn traffic creates danger	No special treatment is required. The traffic is forced to reduce speed that makes increase safety.	
	+	+++	
Local Access	Traffic access to the Nile Brewery Plant from I.C.1will be forced to make u turn at junction that is inconvenient and dangerous.	Traffic accessing to the Nile Brewers factory from I.C.1 run along the roundabout that does not disturbs opposite traffic.	
	+	++	
Resettlement	There is no significant resettlement.	There is no significant resettlement.	
	++	++	
Maintenance	No specific maintenance work is required.	No specific maintenance work is required.	
	++	++	
Construction Cost	Construction cost is lower than that of RA.	Construction cost is higher than that of junction and however the difference is small.	Incl . resettlement cost
	+++	++	
Total (Score of +)	11	12	

Source: JICA Study Team

Based on the results of the comparison, roundabouts are selected as the intersection type for both No.1 and 2.

<u>Jinja City</u>

There are four (4) crossing points to be made for the study road but the crossing point No.6 is the existing intersection which should remain as it is. As discussed above, the study road is classified as an International Trunk Road so that effort to unite and minimize crossing point was made in the design.

Crossing points No.3 and 4 are so close; approximately 90 m apart. They are identified as less important intersections, although few houses exist nearby. About 300 m apart from No.4, crossing point No.5 exist which is close to the army's dormitory and other private properties, and the crossing road at point No.5 is wider than that of No.3 and 4.

Given these conditions, the intersections at point No.3 and 4 should be consolidated with point No.5 for the planning the crossing facility as feeder road under the new Nile Bridge as shown in Figure 6.1.13.

Source: JICA Study Team

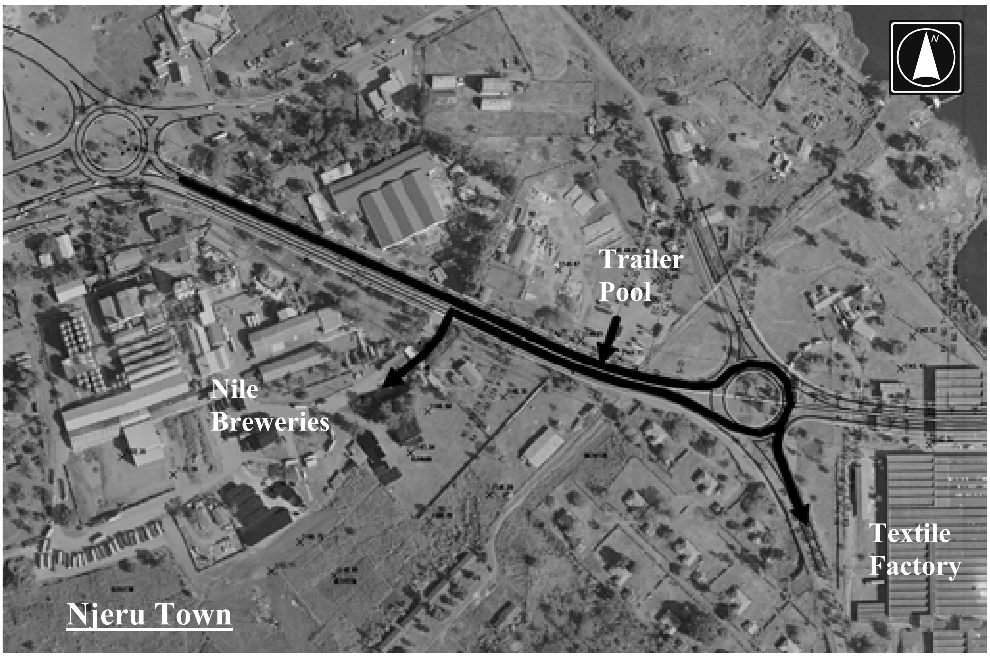
Figure 6.1.13 Feeder Road Plan at Jinja (Consolidated Plan for No.3, No.4 and No.5)

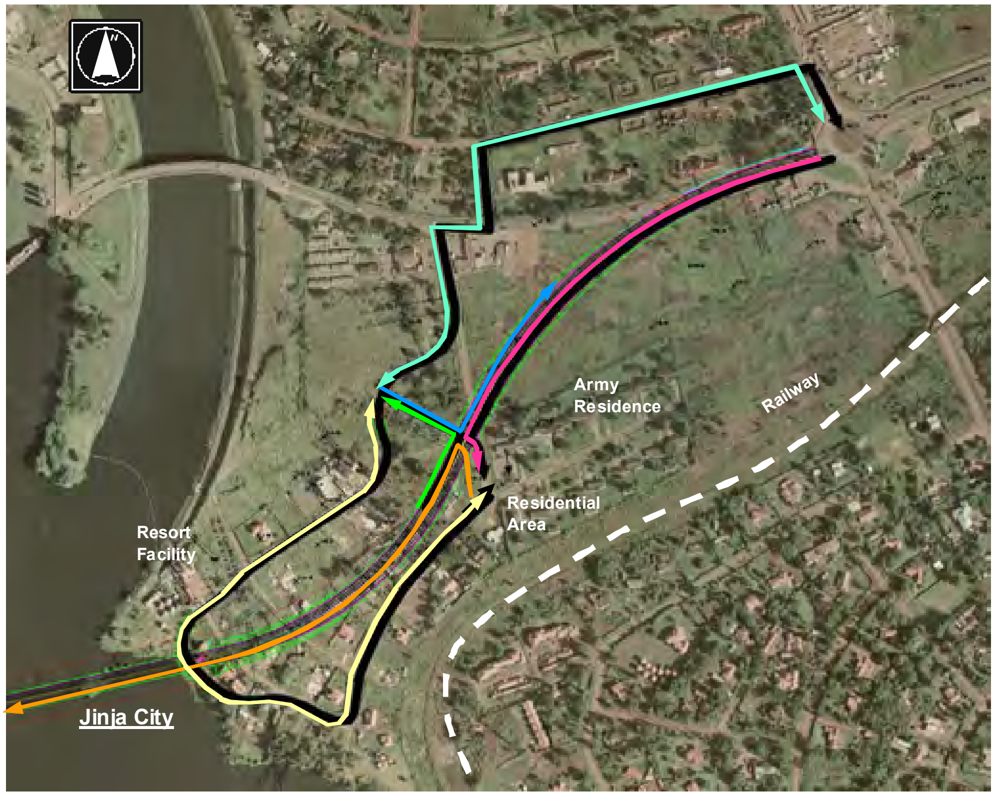
At No.5, the junction was designed not to allow traffic to cross the study road, so that the median of the study road will remain un-opened at the junction. This plan was made because of the following considerations:

- The study road is an International Trunk Road with high design speed so that traffic crossing of the road is not allowed, even when traffic volume is small, for safety concerns.
- Majority of traffic generated are from the residential areas at the southern side of the study road is considered to be motorized in the future, so that traffic using/diverting on the planned feeder road when traffic is towards the northern side of the study road, so as not to suffer a large loss of time and cost.
- The traffic from Njeru side to the above-mentioned residential areas is also using the feeder road inevitably but the time and cost loss is less for the same reason as stated above.

(5) Local Traffic Management

Given the above intersection and feeder road arrangements, local traffic will be managed as shown in Figure 6.1.14.





Source: JICA Study Team

Figure 6.1.14 Local Traffic Management

6.2 Bridge Design

6.2.1 Bridge Design Conditions

(1) Design Standards

Based on the discussion with UNRA and MOWT, the design standards to be used for the project bridge were determined as follows:

Main Standard

- Road Design Manual Volume 4 : Bridge Design, Ministry of Works, Housing and Communications, 2005

Sub-Standard

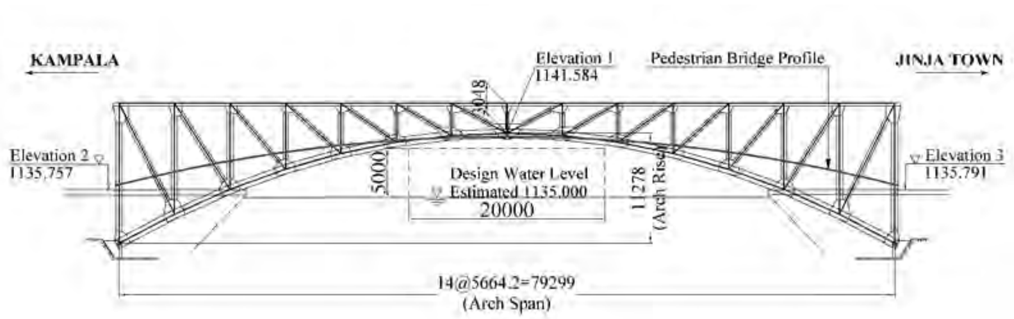
- Latest British Standard BS 5400
- BS 8002:1994, Earth retaining structures
- BS8004: 1986, Foundations
- AASHTO LRFD 2007, Section 3

(2) Restriction on the Bridge

1) Navigation Clearance

Navigation clearance similar to that for the existing Railway Bridge (Nile Bridge) will be provided for the New Nile Bridge. The navigation clearance is 20 m horizontal and 5 m vertical.

The arch rib member of Nile Bridge will be limited to a vertical clearance of 5 m, the design flood level of 1,135.0 m and 20 m horizontal clearance for navigation, as shown in Figure 6.2.1. Horizontal clearance of 20 m allows two ways passing of boats safely.



Source: As-built drawings of Arch Bridge over Nile at Jinja (1930) and site survey results

Figure 6.2.1 Existing Navigation Clearance for the Railway Bridge

Table 6.2.1 shows list of boats/vessels navigating Lake Victoria as provided by the Transport Licensing Board, Ministry of Works and Transport. However, it is confirmed that these boats/ vessels, with the exception of small size boats for the local fishery and/or leisure, are no longer operating in the River Nile at present. In addition, there is no proposal for their operation along the River Nile in future.

Table 6.2.1 Existing Vessels/Boats under Operation in Lake Victoria

Name	Dimensions	Estimated DWT	Vessel Speed (knot)
Tugboat	-	200 ton	6
Weed harvester	-	100 ton	6
Passenger boats	Height – 2m; Width – 3m; Length – 20m	5 ton	12
Cargo boats/ barges	Height – 2m; Width – 3m; Length – 20m	300 ton	8
Patrol boats for military, police & fisheries	Height – 3m; Width – 5m; Length – 8m	3 ton	25
Fishery boats (Fisheries Research Vessels)	Height – 10m; Width – 5m; Length – 20m	10 ton	25
Leisure boats	Height – 7m; Width – 5m; Length – 30m	2 ton	20

Note: Height is from river water surface to the highest portion of boat/vessel.
Source: JICA Study Team

Based on interviews conducted by the JICA Study Team with officials concerned with Nalubaale Dam Complex and major factories within the river reach, it was confirmed that they are utilizing the road and/or railway transport at present, for shipment of their equipment and commodities. Vessel transportation does not exist at present and neither is there a proposal for its use in the future.

In the light of the foregoing, by applying similar navigation clearance as that of the Nile Bridge, the proposed bridge will allow the passage of vessels not restricted by the Nile Bridge clearance.

2) Obstacle Restriction for Jinja Airfield

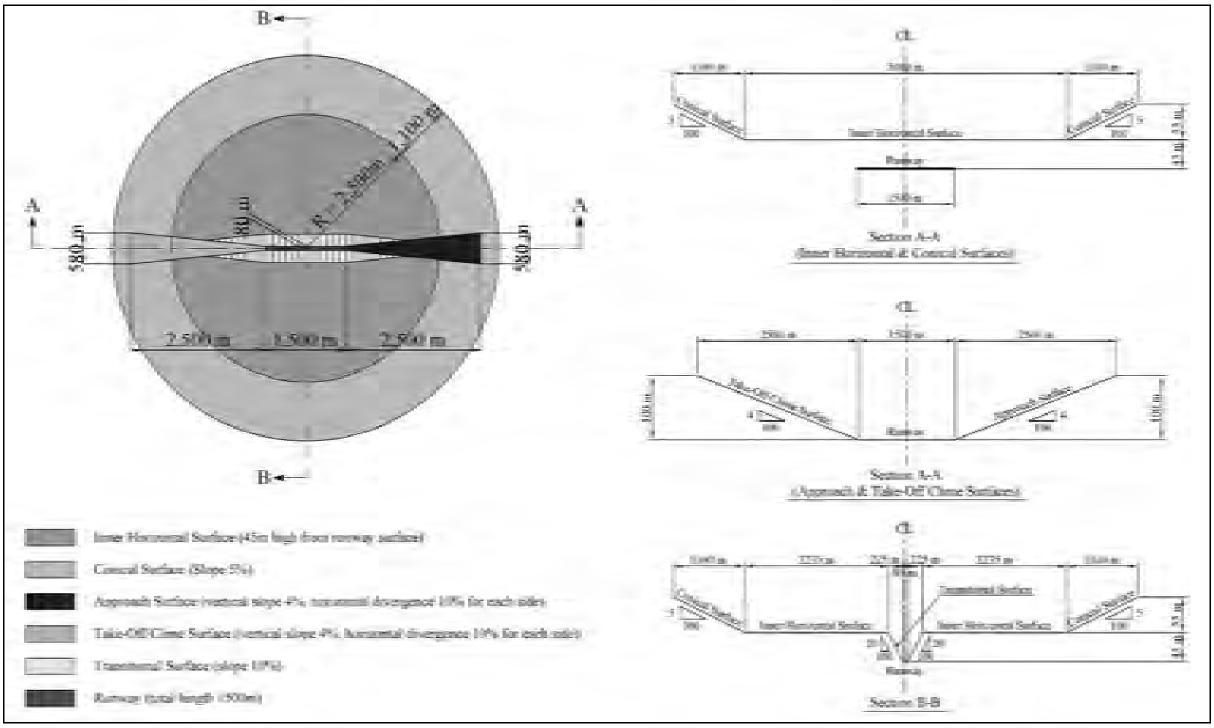
Jinja Airfield is located 4 km from Jinja City along Jinja-Budondo road, and is close to the Bridge Location. The bridge should be designed to ensure it will not infringe the obstacle limitation requirements for the airfield.

Obstacle Limitation Surface, for Jinja Airfield is illustrated in Figure 6.2.2.

According to the Civil Aviation Authority (hereinafter referred to as CAA), Jinja Airfield is classified as Runway Code 2 for Non-instrument Runway and has the following aerodrome features in accordance with Aerodrome Register for Jinja Airfield:

- Geographical Position : 00o 27’ N, 033o 12’E
- Altitude above Mean Sea Level : 3,840 ft (1170 m) from Aerodrome Register, 1,168m based on Uganda Standard Datum (as the survey result)
- Runway Length : 1,500 m
- Runway Width : 30 m
- Runway Orientation : 13/31
- Surface : Murrum

Details above were confirmed by CAA based on the result of the review and checked of bridge alternatives, for which CAA accepted bridge alternative AA4 (3-span cable stayed bridge) so as not to infringe the safety requirements for surface Aerodrome, based on their letter dated March 26, 2009.



Source: JICA Study Team

Figure 6.2.2 Obstacle Limitation Surface for Jinja Airfield

(3) General Statements

1) Definitions

The following definitions and explanations of terms applies to the study.

- Loads : The loads to be considered in determining the load effects, S, on the structure. For certain loads, statistical distributions are available and for these a return period of 120 years has been adopted.
- Strength of Material : Where statistical data is available on the strength of material, characteristic values are given in applicable sections of BS5400.

2) Design Life

A design life of 120 years has been assumed based on BS5400.

3) Verification of Structural Adequacy

For satisfactory design, the following relation will be followed:

$$R^* \geq S^*$$

where:

R^* : design resistance, $R^* = \text{function}(f_k / \gamma_m)$

S^* : design load effects, $S^* = \gamma_{f3} * \gamma_{fL} * Q_k$

fk : characteristic (or nominal) strength of material

γm : reduction factor specified in relevant sections of BS5400

γf3 : partial inaccuracy factor as specified below

γfL : partial load factor based on Table 6.2.2

Qk : nominal loads

		ULS	SLS
Steel bridge		1.10	1.00
Concrete bridge	Elastic analysis	1.10	1.00
	Plastic analysis	1.15	1.00
Composite bridge	Structural steel	1.10	1.00
	Reinfrorcement	1.15	1.00
	Concrete	1.50	1.00 or 1.30
	Shear connector	1.10	1.00
Bridge bearing		1.10	1.00

Source: BS5400

(4) Load Combination

Load combination for the design shall basically be divided into 5 groups and the partial load factors γfL are summarized in Table 6.2.2.

Table 6.2.2 Loads to be taken for Each Combination with Appropriate γ_{fL}

Load		Limit States	γ_{fL} to be considered in combination				
			1	2	3	4	5
Dead:	Steel	ULS[1]	1.05	1.05	1.05	1.05	1.05
		SLS	1.00	1.00	1.00	1.00	1.00
	Concrete	ULS[1]	1.15	1.15	1.15	1.15	1.15
		SLS	1.00	1.00	1.00	1.00	1.00
Superimposed dead	Deck surfacing	ULS[2]	1.75	1.75	1.75	1.75	1.75
		SLS[2]	1.20	1.20	1.20	1.20	1.20
	Other loads	ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
Reduced load facor for dead and superimposed dead load where this has a more severe toal effect		ULS	1.00	1.00	1.00	1.00	1.00
Wind:	During erection	ULS		1.10			
		SLS		1.00			
	With dead & superimposed dead load only, and for members primarily resisting wind loads	ULS		1.40			
		SLS		1.00			
	With dead & superimposed dead & other appropriate combination-2 loads	ULS		1.10			
		SLS		1.00			
	Relieving effect of wind	ULS		1.00			
		SLS		1.00			
Temperature:	Restraint to movement, except frictional	ULS			1.30		
		SLS			1.00		
	Frictional restraint	ULS					1.30
		SLS					1.00
	Effect of temperature difference	ULS			1.00		
		SLS			0.80		
Differential settlement		ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
Exceptional loads			To be assessed and agreed between the engineer and the appropriate authority				
Earth pressure: retained fill And/or live load	Vertical loads	ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
	Non-vertical loads	ULS	1.50	1.50	1.50	1.50	1.50
		SLS	1.00	1.00	1.00	1.00	1.00
	Relieving effect	ULS	1.00	1.00	1.00	1.00	1.00
		SLS	1.00	1.00	1.00	1.00	1.00
Erection: temporary loads		ULS		1.15	1.15		
		SLS		1.00	1.00		
Highway bridges live loading:	HA alone	ULS	1.50	1.25	1.25		
		SLS	1.20	1.00	1.00		
	HA with HB or HB alone	ULS	1.30	1.10	1.10		
		SLS	1.10	1.00	1.00		
	Footway and cycle track loading	ULS	1.50	1.25	1.25		
		SLS	1.00	1.00	1.00		
	Accidental loading[3]	ULS	1.50				
		SLS	1.20				

Note: ULS : ultimate limit state, SLS : serviceability limit state.
Source: BS5400, Part 2

1 γfL shall be increased to at least 1.10 and 1.20 for steel and concrete respectively to compensate for inaccuracies when dead loads are not accurately assessed.
2 γfL may be reduced to 1.2 and 1.0 for ULS and SLS respectively, subject to approval of the relevant authority.
3 Accidental wheel loading shall not be considered as acting with any other primary live loads.

Table 6.2.2 Loads to be taken in Each Combination with Appropriate γ_{fL} (cont.)

Load		Limit States		γ_{fL} to be considered in combination 1	2	3	4	5
Loads due to vehicle collision with parapets and associated primary live load	Local effects: parapet load		Each secondary live load shall be considered separately together with the other combination-4 loads as appropriately					
	Low and normal containment	ULS SLS					1.50 1.20	
	High containment	ULS SLS					1.40 1.15	
	Associated primary live load: low, normal & high containment	ULS SLS					1.30 1.10	
	Global effects; parapet load 1. Massive structures 1) bridge superstructures and non-elastomeric bearings 2) bridge superstructures and non-elastomeric bearings 3) elastomeric bearings	 ULS ULS SLS					 1.25 1.00 1.00	
	2. Light structures a) bridge superstructures and non-elastomeric bearings b) bridge superstructures and non-elastomeric bearings c) elastomeric bearings	 ULS ULS SLS					 1.40 1.40 1.00	
	Associated primary live load: massive and light structures a) bridge superstructures and non-elastomeric bearings, bridge superstructures and non-elastomeric bearings b) elastomeric bearings	 ULS SLS					 1.25 1.00	
Vehicle collision loads on bridge supports and superstructure	Effects on all elements excepting elastomeric bearings	ULS					1.50	
	Effects on elastomeric bearings	SLS					1.00	
Centrifugal load and associated primary live load		ULS					1.50	
		SLS					1.00	
Longitudinal load:	HA and associated primary live load	ULS					1.25	
		SLS					1.00	
	HB and associated primary live load	ULS					1.10	
		SLS					1.00	
Accidental skidding load and associated primary live load		ULS					1.25	
		SLS					1.00	
Foot/cycle track bridges	Live load and effects due to parapet load	ULS SLS		1.50 1.00	1.25 1.00	1.25 1.00		
	Vehiclecollision loads on supports and superstructures[4]	ULS					1.50	
Railway bridges: Type RU and RL primary and secondary live loading		ULS SLS		1.40 1.40	1.20 1.00	1.20 1.00		

Note: For loads arising from creep and shrinkage, or from welding and lack of fit, see Parts 3, 4 and 5 of BS5400, as appropriate.
Source: BS5400, Part 2

[4] This is the only secondary live load to be considered for foot/cycle track bridges.

Earthquake loads

Based on AASHTO LRFD, earthquake load will cater for Extreme Event I limit state. Since it is highly unlikely that an earthquake will occur simultaneously with a major flood, considering that that the mean discharges are used to calculate the water current load that acts simultaneously with an earthquake load.

In combining live and earthquake loads, Turkstra's rule (Section 3.4.2 of AASHTO LRFD) for combining uncorrelated loads may be applied. The rule specified that 0.50 of the load factor for live load to be applied simultaneously with seismic loads is reasonable for a wide range of values of Average Daily Truck Traffic (ADTT).

Water Current Load

The bridge was designed to withstand water current loads during major hydraulic occurrences. Based on AASHTO LRFD, water current load during major hydraulic events is to be covered under Extreme Event II limit state. The recurrence interval of extreme events exceeded the design life. Furthermore, the extreme events are even much less likely to occur simultaneously. As such, extreme events such as major floods, vessel collision and vehicle collision were applied separately. A 0.50 load factor for live load was applied along with extreme events indicating the low probability of maximum vehicular load occurring together with the extreme events.

In Table 3.4.1-1 of AASHTO LRFD, it was observed that a factor of 1.0 is applicable to water current load throughout all the limit states from Strength I to Service IV. It should be noted that the water load (and steam pressure if applicable) under mean discharges of the river shall be used. It was also observed from the table that all extreme events are not only applied separately under limit states of Extreme Event I and II with load factor of 1.0, but are also applied in conjunction with reduced live load factors.

Vessel Collision Load

The bridge is designed to withstand vessel collision loads. Based on AASHTO LRFD Section 3, a 0.50 load factor for live load is applicable along with vessel collision load.

In Table 3.4.1-1 of AASHTO LRFD, it was observed that a factor of 1.0 is applied to vessel collision load throughout all the limit states from Strength I to Service IV. It should be noted that vessel collision load under mean discharges of the river will be adopted.

Summary

- Exceptional loads will be considered only for Ultimate Limit State (ULS).
- A reduced live load factor will be applied to vehicle and pedestrian live loads when exceptional loads are considered.
- When designing for earthquake load and vessel collision load, mean discharges shall be used to calculate water current load. However, when the water current load is applied separately in the design, maximum flood discharges will be adopted.

Given the above mentioned observations, the following load combinations 6, 7 & 8 are recommended for exceptional loads under BS 5400:

Table 6.2.3 Proposed Load Combinations 6, 7 and 8 with Load Coefficients γ fL

Load		Limit States	γ_{fL} to be considered in combination		
			6	7	8
Dead	Steel	ULS	1.05	1.05	1.05
	Concrete	ULS	1.15	1.15	1.15
Superimposed dead	Deck surfacing	ULS	1.75	1.75	1.75
	Other loads	ULS	1.20	1.20	1.20
Earth pressure: Retained fill and/or live load	Vertical loads	ULS	1.20	1.20	1.20
	Non-vertical loads	ULS	1.50	1.50	1.50
	Relieving effect	ULS	1.00	1.00	1.00
Highway bridges live loading:	HA alone	ULS	0.50	0.50	0.50
	HA with HB or HB alone	ULS	0.50	0.50	0.50
	Footway and cycle track loading	ULS	0.50	0.50	0.50
Centrifugal load and associated primary live load		ULS	0.50	0.50	0.50
Earthquake load		ULS	1.00		
Water current at mean discharge		ULS	1.00	1.00	
Vessel collision load		ULS		1.00	
Water current at flood time		ULS			1.00

Source: JICA Study Team

(5) Design Load

Design loads shall be defined based on both “Road Design Manual, Volume 4: Bridge Design” and “BS 5400 Part-2 (1978)”.

Permanent Load

- Dead lolad
- Superimposed dead load
- Earth pressure
- Shrinkage and Creep
- Differential settlement
- Water current
- Water Pressure
- Buoyancy

Transient Load

- Primary live load
- Secondary live load
- Footway & cycle track live load
- Wind load
- Temperature
- Earthquake
- Erection load
- Vessel collision load

Nominal loads for the above are described below:

1) Dead Loads and Superimposed Dead Loads

Dead and superimposed dead loads are calculated using the load intensity in Table 6.2.4.

Table 6.2.4 Dead Load Intensity

Category	Item	Unit	Value	Remarks
Dead load	Reinforced Concrete	kN/m^3	25.00	
	Pre-stressed Concrete	kN/m^3	25.00	
	Plain Concrete	kN/m^3	23.50	
	Asphalt Concrete	kN/m^3	23.00	
	Steel	kN/m^3	78.50	
	Compact Sand	kN/m^3	19.00	
	Loose Sand	kN/m^3	16.00	
Superimposed	Wearing Surface	mm	50	
Dead Load	Bridge Parapet	kN/m	-	To be checked
	Handrail	kN/m	-	To be checked
	Water & Sewage	kN/m	-	4-ϕ500 for water, 4-ϕ500 for sewage
	Others	kN/m	-	To be checked

Source: JICA Study Team

2) Earth Pressure

Earth pressure will depend on the type of soil, water content and creep behaviour, degree of compaction, location of groundwater table, soil-structure interaction, surcharge loads and dynamic effects.

For earth pressure coefficient, either Coulomb or Rankine wedge theory, will be adopted for the bridge abutment and retaining wall.

Active Earth Pressure at Normal Time

$Pa = Ka * (\gamma * z + q)$ for cohesionless soils

where:

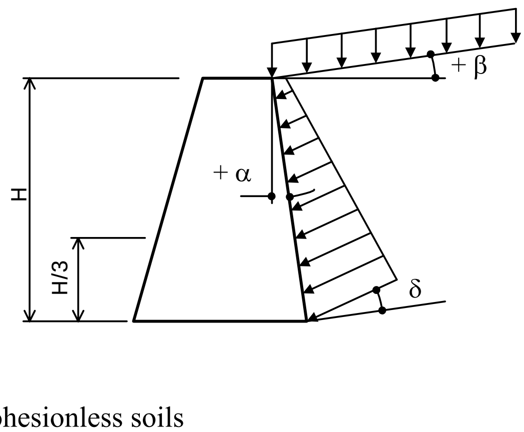
Pa = active lateral earth pressure (kN/m2)

Ka = coefficient of active earth pressure (reference to Annex A of BS 8002:1994)

γ = unit weight of soil above/under the water surface (kN/m3)

z = depth below the surface of earth (mm)

q = live load surcharge (kN/m2)



Passive Earth Pressure at Normal Time

$Pp = Kp * (\gamma * z + q)$ for cohesionless soils

where:

Pp = passive lateral earth pressure (kN/m2)

Kp = coefficient of passive earth pressure (reference to Annex A of BS 8002:1994)

Coefficients of Earth Pressure at Normal Condition

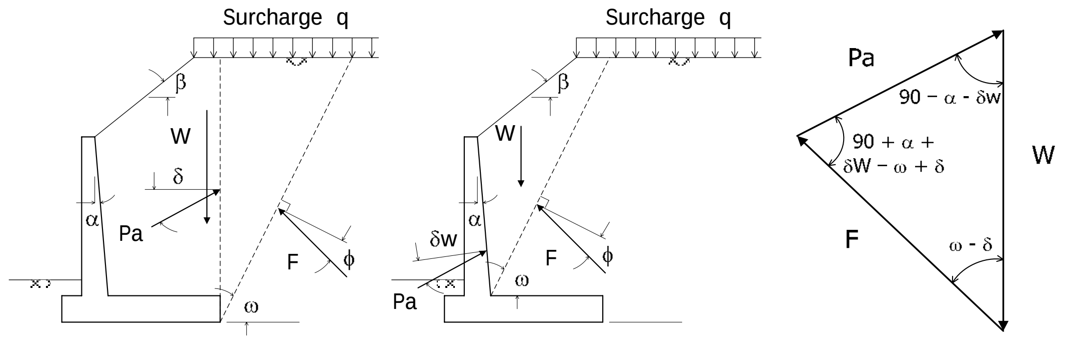
The coefficients for both active and passive earth pressures at normal time shall basically be obtained from Annex A of BS 8002:1994 considering wall friction angle δ, effective friction angle of soil ϕ', inclination of wall face α and inclination of surface of retained soil β.

In special case for smooth vertical wall (δ = 0, α = 0) and horizontal retained soil surface (β = 0), the following equations will be used, as applicable:

Ka = (1 – sin ϕ') / (1 + sin ϕ')

Kp = (1 + sin ϕ') / (1 - sin ϕ')

If the retained soil surface is irregular and the wall face is inclined, active earth pressure for cohesionless soils shall be determined by graphical procedure as follows:



where:

α = inclination of wall (degree)
β = inclination of surface of retained soil
δ = angle of wall friction for stability check (degree)
δw = angle of wall friction for structural design (degree)
ω = assumed sliding plate angle from the horizontal surface (deg)

Figure 6.2.3 Graphical Determination of Active Earth Pressure for Cohesionless Soils

Live Load Surcharge

Nominal load due to live load surcharge q shall be assumed as follows:

HA Loading	:	10.0 kN/m2	
HB Loading	:	20.0 kN/m2	(45 units)
		Interpolation	intermediate
		12.0 kN/m2	(30 units)

Active Earth Pressure at Seismic Time

Pae = Kae * γ * (1 - kv) * z for cohesionless soils

where:

Pae = seismic active lateral earth pressure (kN/m2)
Kae = coefficient of seismic active earth pressure
kv = vertical acceleration coefficient

Active Earth Pressure at Seismic Time

Ppe = Kpe * γ * (1 - kv) * z for cohesionless soils where:

Ppe = seismic passive lateral earth pressure (kN/m2)

Kpe = coefficient of seismic passive earth pressure

Coefficient of Earth Pressure at Seismic Time

The coefficients for both active and passive earth pressures at seismic time shall basically be obtained from the Mononobe-Okabe equations.

$$K_{ae} = \frac{\cos^2(\phi-\theta-\alpha)}{X_1 \cos\theta \cos^2\alpha \cos(\delta+\alpha+\theta)}$$

$$X_1 = \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-\beta)}{\cos(\delta+\alpha+\theta)\cos(\beta-\alpha)}}\right]^2.$$

where:

Kae = seismic active earth pressure coefficient

θ = arc tan (kh/(1-kv)) (deg)

kh = horizontal acceleration coefficient

kv = vertical acceleration coefficient

$$Kpe = \frac{\cos^2(\phi-\theta+\alpha)}{X_2 \cos\theta \cos^2\alpha \cos(\delta-\alpha+\theta)}$$

$$X\gamma = \left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta+\beta)}{\cos(\delta-\alpha+\theta)\cos(\beta-\alpha)}}\right]^2.$$

where:

Kpe = seismic passive earth pressure coefficient

3) Shrinkage and Creep

Where it is necessary to take into account the effects of shrinkage or creep in concrete, requirements specified in appropriate section of BS5400 shall be adopted.

4) Water Current

Since no specification was provided in the British Standard, loads due to water current were calculated in accordance with AASHTO LRFD

5) Water Pressure

Static pressure of water shall be assumed to act perpendicularly to the surface retaining the water. Pressure shall be calculated as the product of the height (Z) of water above the point of consideration and the density of water (γw).

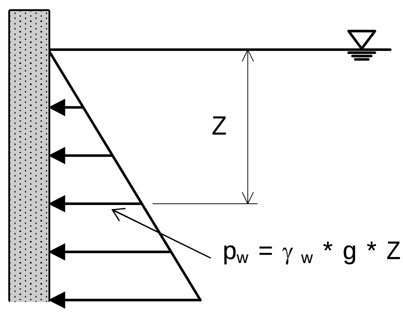


Figure 6.2.4 Static Water Pressure

6) Buoyancy

Buoyancy shall be considered as an uplift force, taken as the sum of the vertical components of static pressures acting on all components below the design water level.

7) Primary Live Load

Standard highway loading consists of HA and HB loadings. Both loadings include impact load.

HA Loading

HA Loading consists of a uniformly distributed load (UDL) and knife edge load (KEL) combined, or of a single wheel load.

- Uniformly distributed load : As shown in Figure 6.2.7, shows the uniformly distributed over the full width of a notional lane. Full UDL for two (2) notional lanes and one-third (1/3) UDL for all other lanes.
- Knife edge load : 120 kN per notional lane, describes the uniformly distributed over the full width of a notional lane. Full KEL for two (2) notional lanes and one-third (1/3) KEL for all other lanes.
- Single nominal wheel load : 100 kN, uniformly distributed over a square contact area (300mm side) assuming an effective pressure of 1.1 N/mm2

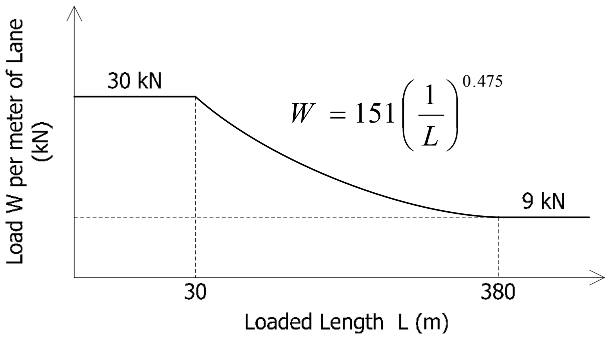
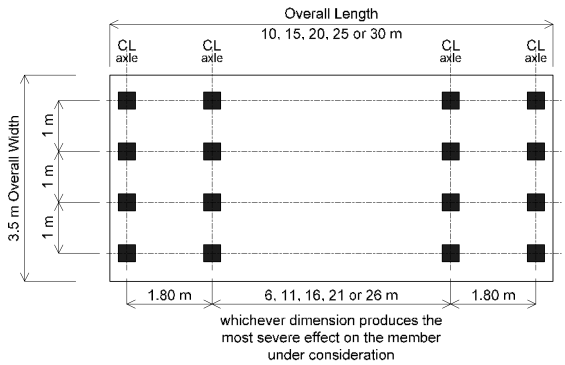


Figure 6.2.4 HA Loading

HB Loading

In the Republic of Uganda, 45 units of HB loading are being applied. Figure 6.2.6 shows the plan and axle arrangement for one unit of nominal HB loading. One unit shall be taken as equal to 10 kN per axle (2.5 kN wheel).

However, since the project bridge is of great importance in crossing the River Nile not only for Uganda but also for East Africa, adequacy for 45 units of HB loading as provided for shall be checked during the design.



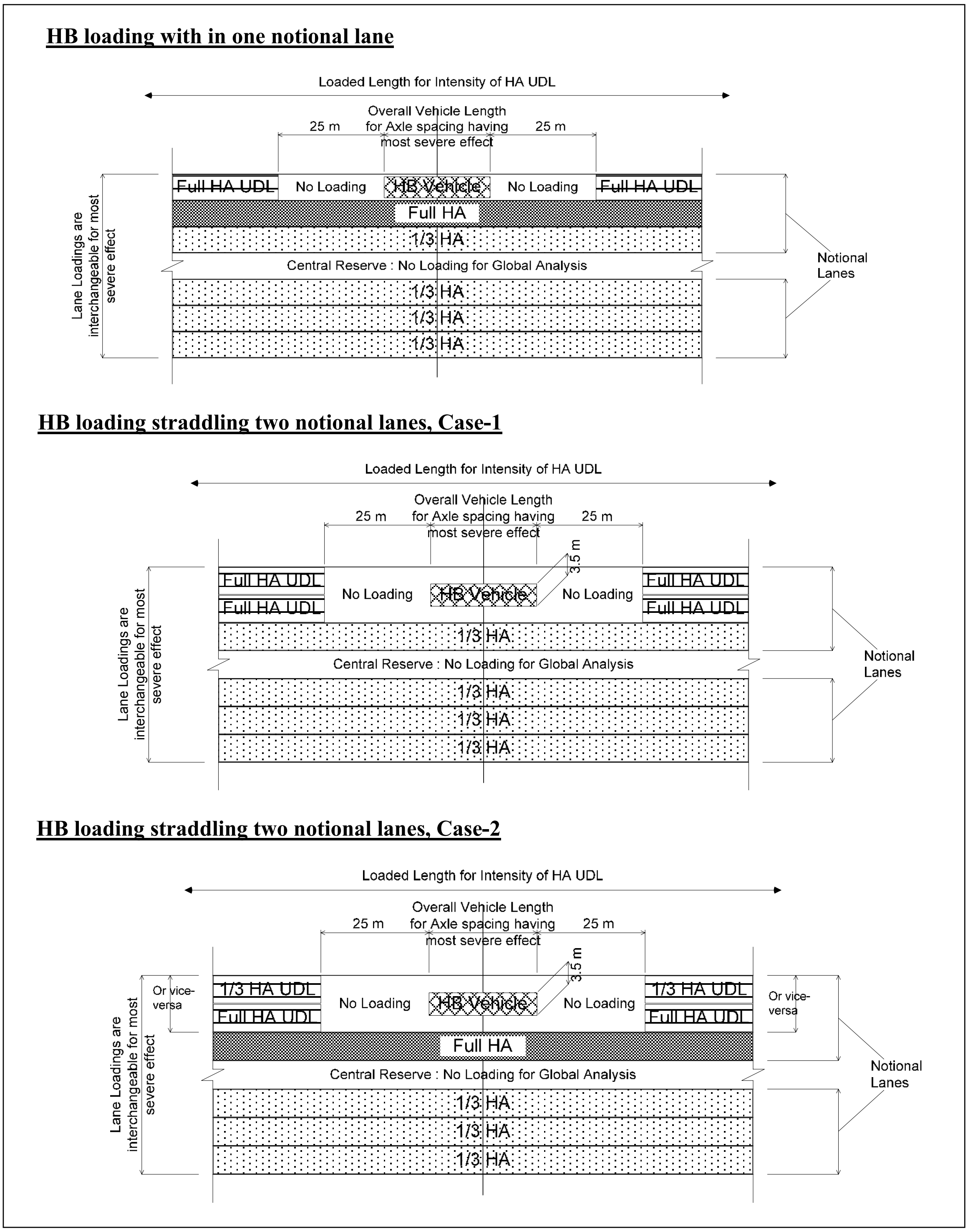
Source: Uganda Road Design Manual

Figure 6.2.5 HB Loading

Nominal HB wheel load was assumed as uniformly distributed over a square contact area (300 mm side) with an effective pressure of 1.1 N/mm2.

HA and HB Loading Combined

HA and HB loadings shall be applied in combination to for the following typical 3 cases as shown in Figure 6.2.7.



Source: Uganda Road Design Manual

Figure 6.2.6 HA and HB Loading in Combination

8) Secondary Live Load

Secondary live loads consist of centrifugal load, longitudinal load, accidental load due to skidding, load due to vehicle collision with parapets, collision load on bridge supports and loading for fatigue investigation.

a) Centrifugal Load

The nominal centrifugal load Fc and associated vertical load Vc were considered for curved bridges and structures.

Fc	= 30,000／(r + 150) kN	: point loads applied in any two (2) notional lanes at 50 m centres, acting in a radial direction at the surface of road and parallel to it.
Vc	= 300 kN	: distributed uniformly over the notional lane for a length of 5 m.

where, r = radius of curvature of the lane (m)

These loads may be subdivided into one-third (1/3) and two-thirds (2/3).

b) Longitudinal Load

Longitudinal load resulting from tracking or braking of vehicles was considered based on the severity between the following loadings:

HA Loading	: 200 + 8.0 L (<= 700) in kN	
HB Loading	: 25% of HB Loading	: equally distributed between eight (8) wheels of two (2) axles of vehicle, 1.8 m apart.

where, L = loaded length in meters

Both loads shall be applied at the road surface and parallel to it in one notional lane only.

Associated nominal live load shall be HA or HB load to act with the longitudinal load as appropriate.

c) Accidental Load due to Skidding

Nominal accidental load was taken as 300 kN as a single point load, considered in one notional lane only, acting in any direction parallel to the surface of the road. Associated nominal live load shall be HA load.

d) Load due to Vehicle Collision with the Parapet

The local effects of vehicle collision with parapets shall be considered in the design of elements of the structure supporting the parapets, and the global effects of vehicle collision with high level of containment parapets was also considered in the design of superstructures, bearings, substructures and retaining walls and wing walls. Nominal loads for each effect are described as follows:

Local Effects

For concrete parapet : Calculated ultimate design moment resistance and design shear resistance of a 4.5 m length of parapet, applied at the parapet base uniformly over any 4.5 m length of supporting element.

For metal parapet : Calculated ultimate design moment resistance of a parapet post at each base up to 3 adjacent posts, and the lesser of

i) calculated ultimate design moment resistance of a parapet post divided by the height of the centroid of the lowest effective longitudinal member above the base, applied at each base of up to any 3 adjacent parapet posts,

ii) calculated ultimate design shear resistance of a parapet post applied at each base of up to 3 adjacent parapet posts.

Associated nominal primary load having the plan, axle and wheel load arrangement shown in Figure 6.2.8, called accidental wheel loading, shall be applied in a position that produces the most adverse effect on the element.

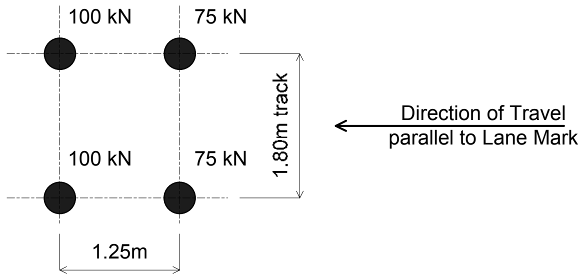


Figure 6.2.7 Associated Nominal Primary Load

Global Effects

The following nominal impact loads shall be applied uniformly over a length of 3 m at the top of the traffic face of high level containment parapets only.

- a single horizontal transverse load of 500 kN
- a single horizontal longitudinal load of 100 kN
- a single vertical load of 175 kN

The associated nominal primary live load shall be for HA loading and accidental wheel loading as shown in Figure 6.2.8. These loads may be applied either separately or in combination.

e) Collision Load on Bridge Support

Nominal collision loads on supports over the highways are given in Table 6.2.5, together with their direction and height of application, and will be considered as acting horizontally on the bridge supports.

Table 6.2.5 Collision Loads on Supports of Bridge over Highways

	Load normal to Carriageway below	Load parallel to Carriageway below	Point of Application on Bridge Support
Loads transmitted from guard rail	150 kN	50 kN	- Any one bracket attachment point or, - 0.75 m above carriageway level for free standing fences
Residual load above guard rail	100 kN	100 kN	- At the most severe point between 1.0 m nd 3.0 m above carriageway level

Source: BS5400

No associated primary live load is required.

f) Loading for Fatigue Investigations

Loading for fatigue investigations shall be in accordance with BS5400 Part 10 in 1980.

The load factors γfL and γf3 shall be taken as 1.00.

9) Footway and Cycle Track Live Load

The nominal live load on elements supporting footways and cycle tracks only shall be taken as follows:

- for loaded lengths of 30 m and under, a uniformly distributed live load of 5.0 kN/m2,
- for loaded lengths in excess of 30 m, k x 5.0 kN/m2,

where,

k = (Nominal HA UDL for appropriate loaded length in kN/m) / 30 kN/m

Nominal load for pedestrian parapet shall be taken as 1.4 kN/m, applied at a height of 1.0 m above the footway or cycle track and acting horizontally.

10) Wind Load

The terrain surface characteristics upwind were classified as follows:

- Open Country : Open terrains with scattered obstructions having heights generally less than 10 m (flat open country, grasslands)
- Suburban : Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family or larger dwellings.
- City : Large city centres with at least 50% of the buildings having a height in excess of 2.1 m

The project bridge is over the River Nile, and there are only scattered obstructions windward of the bridge except for the railway bridge. It was categorized as “Open Country” in consideration of safety.

Wind load was assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all components based on the elevation taken perpendicular to the assumed wind direction.

For bridges or parts of bridges more than 10 m above low ground or water level, the design wind velocity, VDZ, was adjusted according to equations:

$$V_{DZ} = 2.5V_o\left(\frac{V_{10}}{V_B}\right)\ln\left(\frac{Z}{Z_o}\right)$$

where:

V_{DZ} = design wind velocity at design elevation, Z (km/h),

V_{10} = wind velocity at 10 m above low ground or above design water level (km/h), to be obtained from site specific wind survey. If no such result is obtained, 160 km/h may be used,

V_B = base wind velocity of 160 km/h at 10 m height,

Z = height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 10 m,

V_o = friction velocity, a meteorological wind characteristic taken as specified in Table 6.2.6, for various upwind surface characteristics (km/h),

Z_o = friction length of upstream fetch, a meteorological wind characteristic taken as specified in Table 6.2.6,

Table 6.2.6 Values of Vo and Zo for Various Upstream Surface Conditions

Condition	Open Country	Suburban	City
V_o (km/h)	13.2	17.6	19.3
Z_o (mm)	70	1,000	2,500

Source: Uganda Road Design Manual

a) Wind Pressure on Structures: WS

The direction of the design wind shall be assumed as horizontal. Design wind pressure, in MPa, was determined as follows:

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 = P_B\frac{{V_{DZ}}^2}{25,600}$$

where:

P_B = base wind pressure as specified in Table 6.2.7 (MPa),

Table 6.2.7 Base Pressure, P_B, Corresponding to V_B = 160 km/h

Superstructure Component	Windward Load, MPa	Leeward Load, MPa
Trusses, Columns and Arches	0.0024	0.0012
Beams	0.0024	N/A
Large Flat Surfaces	0.0019	N/A

Source: AASHTO LRFD 2007

The total wind loadings shall not considered for the detail design:

On truss and arch components:

- less than 4.4 N/mm in the plane of a windward chord,
- less than 2.2 N/mm in the plane of a leeward chord,

On beam or girder spans:

- less than 4.4 N/mm,

Loads from superstructures

The wind direction for the design shall be such that it produces the extreme force effect on the component under investigation. The transverse and longitudinal pressures shall be applied simultaneously.

Where the wind is not taken as normal to the structures, the base wind pressure, PB, for various angles of wind direction was taken as specified in Table 6.2.8.

Table 6.2.8 Base Wind Pressure, P_B, for Various Angles of Attack with V_B = 160 km/h

Skew Angle of Wind (degree)	Columns & Arches		Girders	
	Lateral (MPa)	Longitudinal (MPa)	Lateral (MPa)	Longitudinal (MPa)
0	0.0036	0	0.0024	0
15	0.0034	0.0006	0.0021	0.0003
30	0.0031	0.0013	0.0020	0.0006
45	0.0023	0.0020	0.0016	0.0008
60	0.0011	0.0024	0.0008	0.0009

Source: AASHTO LRFD 2007

Forces applied directly to the substructures

The transverse and longitudinal forces to be applied directly to the substructure will be calculated from an assumed base wind pressure of 0.0019 MPa.

For wind directions taken skewed to the substructures, this force will be resolved into components perpendicular to the end and front elevations of the substructure.

These forces will be applied simultaneously with those from the superstructure.

b) Wind Pressure on Vehicles: WL

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles will be represented by an interruptible, moving force of 1.46 N/mm acting normal to, and 1,800 mm above the roadway and to be transmitted to the structure.

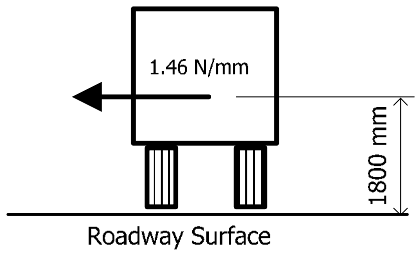


Figure 6.2.8 Wind Load on Vehicle

When wind on vehicles is not taken as normal to the structure, the components of normal and parallel forces applied to the live load were taken as specified in Table 6.2.9.

Table 6.2.9 Wind Components on Live Load

Skew Angle (degree)	Normal Component (N/mm)	Parallel Component (N/mm)
0	1.46	0.00
15	1.28	0.18
30	1.20	0.35
45	0.96	0.47
60	0.50	0.55

Source: AASHTO LRFD 2007

c) Vertical Wind Pressure

Vertical upward wind force of 9.6 x 10-4 times the width of the deck, including parapets and sidewalks, were considered to be a longitudinal line load.

This force will be applied only for limit states that do not involve wind or live load, and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge.

This lineal force will be applied to windward quarter-point at the deck width in conjunction with the horizontal wind loads.

d) Aero-elastic Instability

Aero-elastic force effects will be taken into account in the design of the bridges and structural components apt to be wind-sensitive.

It is said that bridges, decks and individual structural components are to be aero-elastically insensitive if length-to-width or length-to-depth ratios are under about 30.0.

Flexible bridges such as suspension bridges and cable stay bridges may require special studies based on wind tunnel information.

11) Temperature

a) Minimum and Maximum Effective Bridge Temperature (Uganda practice) Coefficient of thermal expansion/contraction

- Normal concrete : 12×10^{-6} / ℃
- Structural steel : 12×10^{-6} / ℃
- Temperature range : 15 ℃ - 35 ℃

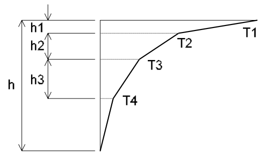
b) Temperature Difference

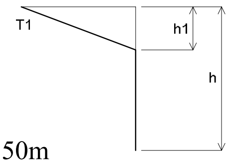
Effects of temperature difference within the superstructure were derived from the data in Table 6.2.10.

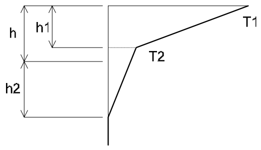
Positive temperature difference occurs when conditions such solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck.

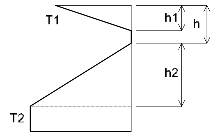
Temperature differences are sensitive to the thickness of surfacing, and the data assumes depths of 40 mm for Groups 1 and 2, and 100 mm for Groups 3 and 4. For other depths of surfacing, different values will apply.

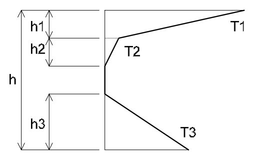
Table 6.2.10 Temperature Difference for Different Types of Construction

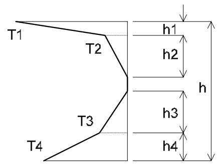
<table>
<tr><th rowspan="2">Group</th><th rowspan="2">Type of Constructon</th><th colspan="2">Temperature Difference ℃</th></tr>
<tr><th>Positive Temperature Difference</th><th>Negative Temperature Difference</th></tr>
<tr><td>1</td><td>Steel deck on steel box girders</td>
<td>

h1 = 0.10m, T1 = 24 ℃
h2 = 0.20m, T2 = 14 ℃
h3 = 0.30m, T3 = 8 ℃
T4 = 4 ℃</td>
<td>

h1 = 0.50m
T1 = 6 ℃</td></tr>
<tr><td>2</td><td>Steel deck on steel truss or plate girders</td><td colspan="2">Use differences as for Group 1</td></tr>
<tr><td>3</td><td>Concrete deck on steel box, truss or plate girders</td>
<td>h1 = 0.60h
h2 = 0.40m
T2 = 4 ℃


<table><tr><th>h (m)</th><th>T1 (℃)</th></tr><tr><td>0.2</td><td>13</td></tr><tr><td>0.3</td><td>16</td></tr></table></td>
<td>h1 = 0.60h
h2 = 0.40m
T2 = 8 ℃


<table><tr><th>h (m)</th><th>T1 (℃)</th></tr><tr><td>0.2</td><td>3.5</td></tr><tr><td>0.3</td><td>5.0</td></tr></table></td></tr>
<tr><td>4</td><td>Concrete slab or concrete deck on concrete beams or box girders</td>
<td>

h1 = 0.30h <= 0.15m,
h2 = 0.30h >= 0.10m, <= 0.25m
h3 = 0.30h <= 0.10m + surfacing depth
<table><tr><th>h (m)</th><th>T1 (℃)</th><th>T2</th><th>T3</th></tr>
<tr><td><= 0.2</td><td>8.5</td><td>3.5</td><td>0.5</td></tr>
<tr><td>0.4</td><td>12.0</td><td>3.0</td><td>1.5</td></tr>
<tr><td>0.6</td><td>13.0</td><td>3.0</td><td>2.0</td></tr>
<tr><td>>= 0.8</td><td>13.5</td><td>3.0</td><td>2.5</td></tr></table></td>
<td>

h1= h4 = 0.20h <= 0.25m
h2 = h3 = 0.25h <= 0.20m
<table><tr><th>h (m)</th><th>T1 (℃)</th><th>T2</th><th>T3</th><th>T4</th></tr>
<tr><td><= 0.2</td><td>2.0</td><td>0.5</td><td>0.5</td><td>1.5</td></tr>
<tr><td>0.4</td><td>4.5</td><td>1.4</td><td>1.0</td><td>3.5</td></tr>
<tr><td>0.6</td><td>6.5</td><td>1.8</td><td>1.5</td><td>5.0</td></tr>
<tr><td>0.8</td><td>7.6</td><td>1.7</td><td>1.5</td><td>6.0</td></tr>
<tr><td>1.0</td><td>8.0</td><td>1.5</td><td>1.5</td><td>6.3</td></tr>
<tr><td>>= 1.5</td><td>8.4</td><td>0.5</td><td>1.0</td><td>6.5</td></tr></table></td></tr>
</table>

Source: BS5400

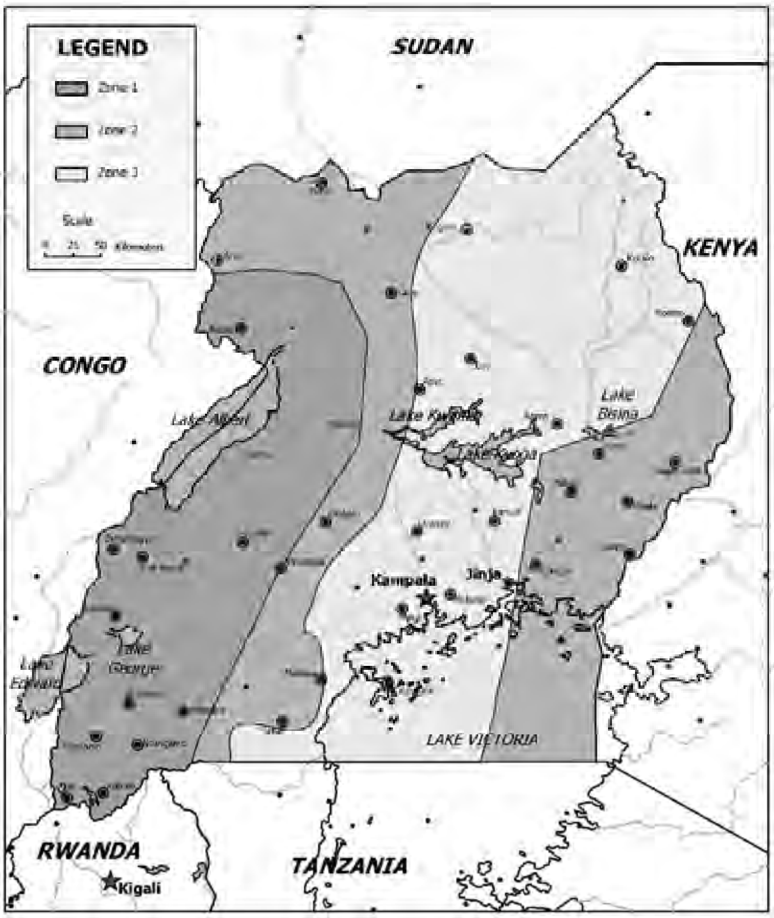
12) Earthquake

The provisions below shall be applicable to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 150 m. Suspension bridges, cable-stayed bridges, arch type and movable bridges may be subject to dynamic analysis.

No detailed seismic analysis is required for any single span bridge or for any bridge with Seismic Performance Category (SPC) 1 &2. For these bridges, the connections must be designed for specified forces and minimum support length requirements must be satisfied.

Seismic Zone

The project bridge is located in Jinja, which is classified as Seismic Zone 3 as shown in Figure 6.2.10.



Source: Uganda Road Design Manual

Figure 6.2.9 Seismic Map with Zone

Acceleration Coefficient, A

As the project bridge is located in Seismic Zone 3, acceleration coefficient A was taken as 0.05 from Table 6.2.11.

Table 6.2.11 Bedrock Acceleration Coefficient, A

Seismic Zone	1	2	3
A	0.15	0.07	0.05

Source: Uganda Road Design Manual

Importance Classification, IC

An Importance Classification (IC) shall be assigned for all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC), as follows:

- Essential bridges IC = I
- Other bridges IC = II

The project bridge is deemed to be an Essential Bridge with IC = I.

Seismic Performance Categories, SPC

Each bridge will be assigned with one of the three (3) Seismic Performance Categories (SPC): 1 to 3, based on Acceleration Coefficient (A) and Importance Classification (IC), as shown in Table 6.2.12.

Table 6.2.12 Seismic Performance Categories (SPC)

Acceleration Coefficient A					Importance Classification (IC) I (Essential Bridges)	Importance Classification (IC) II (Other Bridges)
		A	<=	0.05	**1**	1
0.05	<	A	<=	0.07	2	2
0.07	<	A	<=	0.15	3	3

Source: Uganda Road Design Manual

The Project bridge is classified as "IC = 1".

Site Effects

The effects of site conditions on bridge response was determined from a site coefficient, S, based on soil profile types defined as follows:

- Soil Profile Type I : Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 762 m/sec, or by other appropriate means of classification; or

  Stiff soil conditions where the soil depth is less than 60 m and the soil types overlying rock are stable deposits of sands, gravels, or still clays.
- Soil Profile Type II : A profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 60 m and

the soil types overlying rock are stable deposits of sands, gravels, or still clays.

- Soil Profile Type III : A profile with soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

The Site coefficient (S) approximates the effects of the site conditions on the elastic response coefficient or spectrum and is given in Table 6.2.13.

Table 6.2.13 Site Coefficient, S

	Soil Profile Type		
	I	II	III
Site Coefficient, S	1.00	1.20	1.50

Source: Uganda Road Design Manual

Seismic Design

As the result of discussion with UNRA, though the project bridge is classified as SPC 1, single mode or multimode spectral analysis shall be carried out on bridge structural members together with connections and minimum bearing support lengths.

13) Erection Load

For the ultimate limit state, erection loads was considered as follows:

Temporary Nominal Loads

The total weight of all temporary materials, plant and equipment to be used during erection was taken into account and accurately assessed to ensure that the loading is not underestimated.

Permanent Nominal Loads

All dead and superimposed dead loads affecting the structure at each stage of construction was taken into account.

Wind and Temperature Effect

Wind and temperature effects will be considered, as appropriate.

For the serviceability limit state, nothing will be done during erection that could cause damage to the permanent structures or alter its response in service from that considered in design.

14) Vessel Collision Load

Vessel collision load was not included in the Uganda Bridge Design Manual or British Standard. It was therefore determined with reference to AASHTO LRFD.

Though AASHTO LRFD indicates vessel collision loads by ship and barge, only collision load by ship will be considered.

a) Design Vessel

Though no navigable boats or vessels exist currently nor are expected in the future, vessel collision load shall be considered for structural stability and durability.

With reference to existing boats and vessels operating on Lake Victoria as shown in Table 6.2.1, the following boats/vessels can pass under the existing Railway Bridge in consideration of the 5.0 m high navigation clearance above design flood level:

- Passenger boats of 5 ton (DWT) at estimated speed of 12 knots
- Cargo boats of 300 ton (DWT) at estimated speed of 8 knots
- Patrol boats for military, police & fisheries of 3 ton (DWT) at estimated speed of 25 knots

b) Design Collision Velocity

For the purpose of project bridge design, boat / vessel speed will be used for the design collision velocity.

c) Ship Collision Force on Pier

Head-on ship collision impact force on a pier will be taken based on the following equation:

Ps = 1.2 * 10 5 V DWT 0.5

where:

P_s = equivalent static vessel impact force (N)

DWT = deadweight tonnage of vessel (Mg)

V = vessel impact velocity (m/s)

For each design vessel discussed above, vessel collision loads will be calculated as follows:

P_s = 1,660 (kN) for passenger boats;

P_s = 8,550 (kN) for cargo boats;

P_s = 2,675 (kN) for patrol boats for military, police & fisheries;

d) Application of Impact Force

For substructure design, equivalent static forces, parallel and normal to the centreline of the navigable channel, were separately applied as follows:

- 100% of the design impact force in a direction parallel to the centreline of the navigable channel, or
- 50% of the design impact force in a direction normal to the centreline of navigable channel

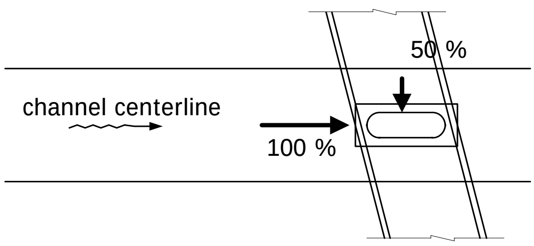


Figure 6.2.10 Load Application in Parallel and Normal Directions

For overall stability, the impact force to be applied will be a concentrated force on the substructure at mean annual high water level of the waterway, as shown in Figure 6.2.12.

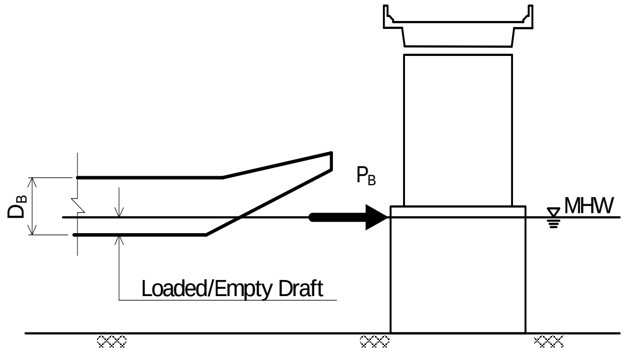


Figure 6.2.11 Load Application for Overall Stability

(6) Materials

1) Concrete

Minimum crushing strength for each structural element shall be, as a minimum, based on the design results, which will be determined during the design work based on BS5400 Part 4: Code of Practice for Design of Concrete Bridges.

Modulus of elasticity of concrete for short term loading is given in Table 6.2.14. The effect of creep for long term loading is normally allowed by using half the values in Table 6.2.14 for modulus of elasticity.

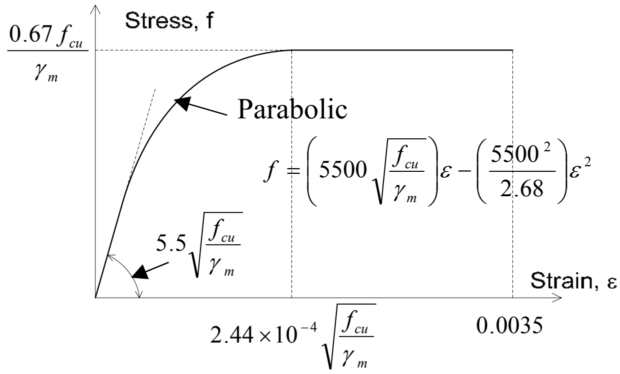
Table 6.2.14 Modulus of Elasticity of Concrete under Short Term Loading

Cube Strength of Concrete at Age Considered (MPa)	Modulus of Elasticity of Concrete, Ec (MPa)
20	25,000
25	26,000
30	28,000
40	31,000
50	34,000
60	36,000

Source: BS5400

Poisson's ratio may be taken as 0.20.

The design stress-strain curve may be taken as shown in Figure 6.2.13.



Source: BS5400

Figure 6.2.12 Short Term Design Stress-Strain Curve for Normal Weight of Concrete

2) Reinforcing Bar

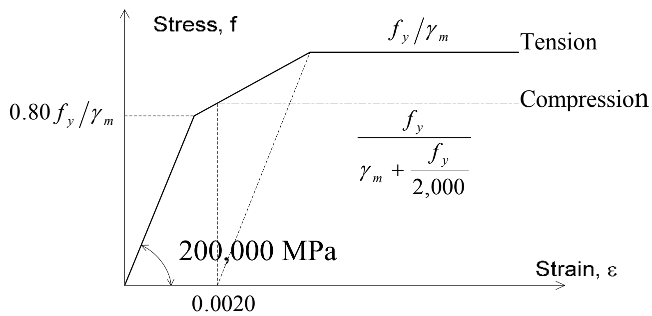
Reinforcing bars shall be of deformed bars (Type-2) in accordance with BS 4449. The properties and strength are summarized in Table 6.2.15.

Table 6.2.15 Properties and Strength of Reinforcing Bars

Nominal Diameter (mm)	Area (cm2)	Mass (kg/m)
8	0.503	0.395
10	0.785	0.616
12	1.131	0.888
16	2.011	1.579
20	3.142	2.466
25	4.909	3.854
32	8.042	6.313
40	12.566	9.864

Note: For Grde 250, only nominal diameter of 8, 10, 12 and 16 mm are preferred.
Source: BS5400

The design stress-strain relation is shown in Figure 6.2.14.



Source: BS5400

Figure 6.2.13 Short Term Design Stress-Strain Curve for Reinforcement

3) Prestressing Steel

Uncoated Seven-wire High Tensile Cold Drawn Low-relaxation strand for Prestressed Concrete in accordance with BS5896 shall be used with the following properties and strength shown in Table 6.2.16.

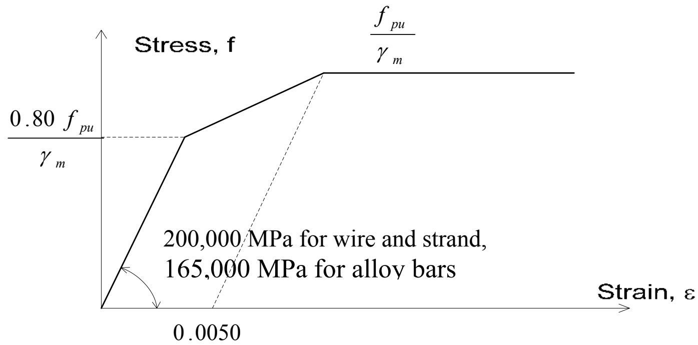
Strand may be 12.5 mm, and 15.2 mm in diameter may be used for cast-in-place post-tensioned members.

Table 6.2.16 Properties of Prestressing Strand

Nominal Diameter (mm)	Nominal Tensile Strength f_{pu} (MPa)	Nominal Steel Area (mm2)	Nominal Mass (g/m)	Specified Characteristic Breaking Load (kN)	Specified Characteristic 0.1% Proof Load (kN)	Load at 1% Elongation (kN)	Maximum Elongation at Max. Load (%)
15.2	1670	139	1,090	232	197	204	3.5
12.5	1770	93	730	165	139	144	3.5
11.0	1770	71	557	125	106	110	3.5
9.3	1770	52	408	92	78	81	3.5

Source: BS5400

Modulus of elasticity, Ep, for the short and long term periods may be taken from Figure 6.2.15 as the appropriate tangent modulus at zero loads.



Source: BS5400

Figure 6.2.14 Short Term Design Stress-Strain Curve for Reinforcement

4) Structural Steel

Structural steel shall be of "S" grades, no higher than S460 (S235, S275, S355, S420, S460) conforming to BS7668, BS EN 10025, BS EN 10113, BS EN 10137, BS EN 10155 or BS EN 10210.

The following properties for structural steel shall be assumed in the design:

- modulus of elasticity, E = 205,000 MPa
- shear modulus, G = 80,000 MPa
- Poisson's ratio, □ = 0.3

5) Material Coefficient

For the analysis of sections, the values of material coefficient shall be used as indicated in Tables 2.2.17 and 2.2.18.

Table 6.2.17 Material Coefficient for Concrete, Reinforcement & PC Strand γ_m

Material	Type of stress	Serviceability Limit		Ultimate Limit
		RC	PC	RC, PC
Concrete	Triangular or near-triangular compressive stress distribution	1.00	1.25	1.50
	Uniform or near-uniform compressive stress distribution	1.33	1.67	1.50
	Tension	Not applicable	1.25 pre-tensioned 1.55 post-tensioned	1.50
Reinforce-ment	Compression Tension	1.00	Not applicable	1.15
PC Strand	Tension	Not applicable	Not required (not to be checked)	1.15

Source: BS5400

Table 6.2.18 Material Coefficient for Structural Steel γ_m

Limit State	Structural component and behaviour	γ_m
Ultimate Limit State	Strength of longitudinal stiffeners	1.20 (fibre in compression) 1.05 (fibre in tension)
	Buckling resistance of stiffeners	1.20
	Fasteners in tension	1.20
	Fasteners in shear	1.10
	Friction capacity of HSFG bolts	1.30
	Welds	1.20
	Others	1.05
Serviceability Limit State	Friction capacity of HSFG bolts	1.20
	Others	1.00

Source: BS5400

6.2.2 Preliminary Bridge Design

(1) Introduction

Preliminary bridge designs were carried out following the concept of the structures selected by the study on Selection of Optimum Solution to cross River Nile at Jinja.

1. Prestressed concrete three-span cable-stayed bridge, 290 m main span, 100m left side span and 135m right side span, total bridge length 525 m.
2. The 100m long semi-underground structure on left side span. Inverted Y shape Pylons.
4. Spread footing for foundations on land (P1).
5. Cast in Place Concrete Pile with Steel Casing for the foundation on works on the island (P2).

(2) River Characteristics

The River Nile at the New Nile Bridge would be quite stable for the construction of a new bridge, since there have been no changes in the alignment of the banks due to erosion or deposition in the last 45 years, and it is characterized by amphibolites rocks that are the basis for the formation of the islands on the right bank side of the river.

Therefore, it is unlikely that erosion will ever occur for New Nile Bridge.

1) Longitudinal Profile

As shown in Figure 6.2.16 of the longitudinal profile between the Nalubaale dam and the Source of Nile, the average riverbed slope is 1/200.

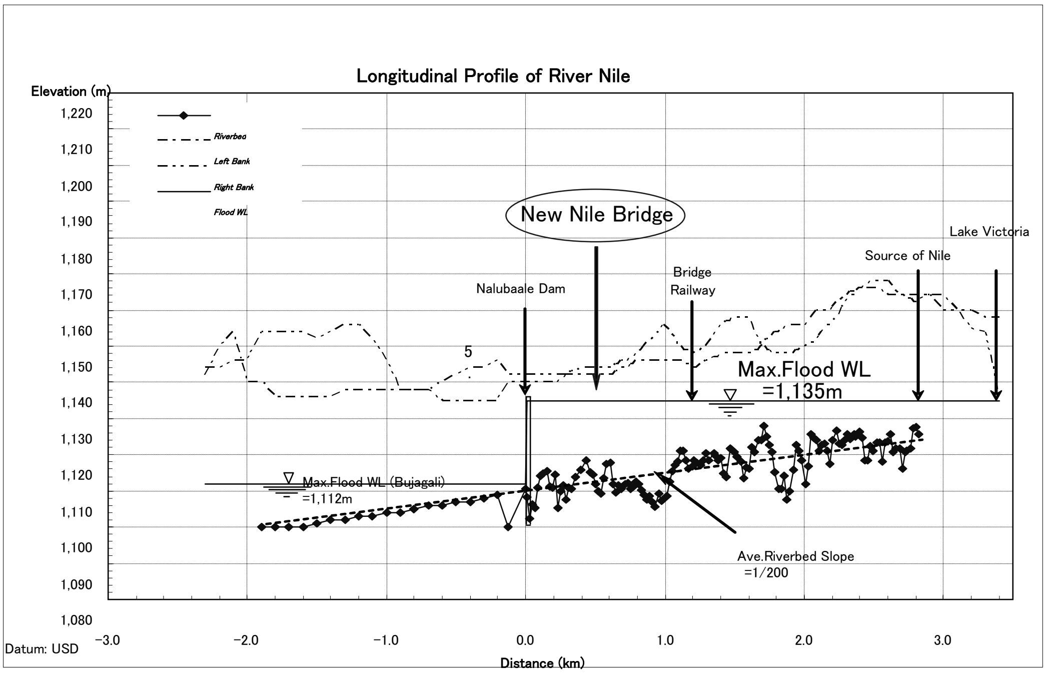
2) Flood Water Level

The maximum water depth during flood at the Bridge Location was estimated at 25.5m as shown Table 6.2.19, based on flood water levels defined in the dam operation rules (Figure 6.2.17).

Table 6.2.19 Flood Water Level at Bridge Location

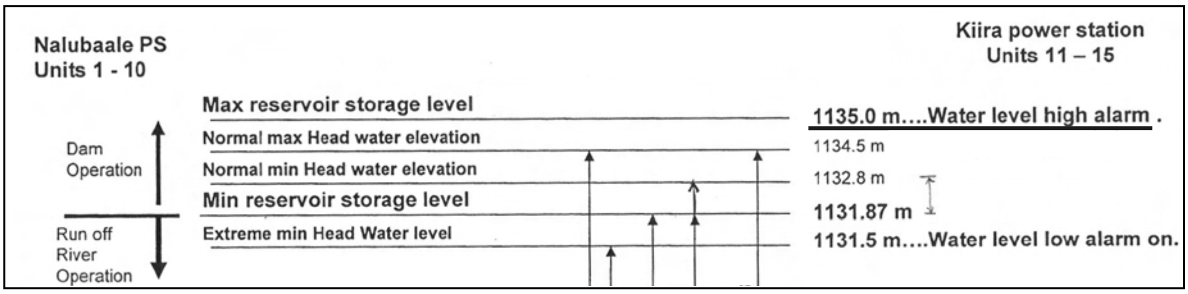
Flood Water Level	1135.0 m
Riverbed Level	1109.5 m
Flood Water Depth	25.5 m

Source: JICA Study Team



Source: (1) Flood Discharge: by Emergency Preparedness Plan (EPP) in 2003
(2) Flood Levels: by Nalubaale Dam Operation Regulation, Eskom

Figure 6.2.15 Longitudinal Profile



Source: Nalubaale Dam Operation Regulation, Eskom

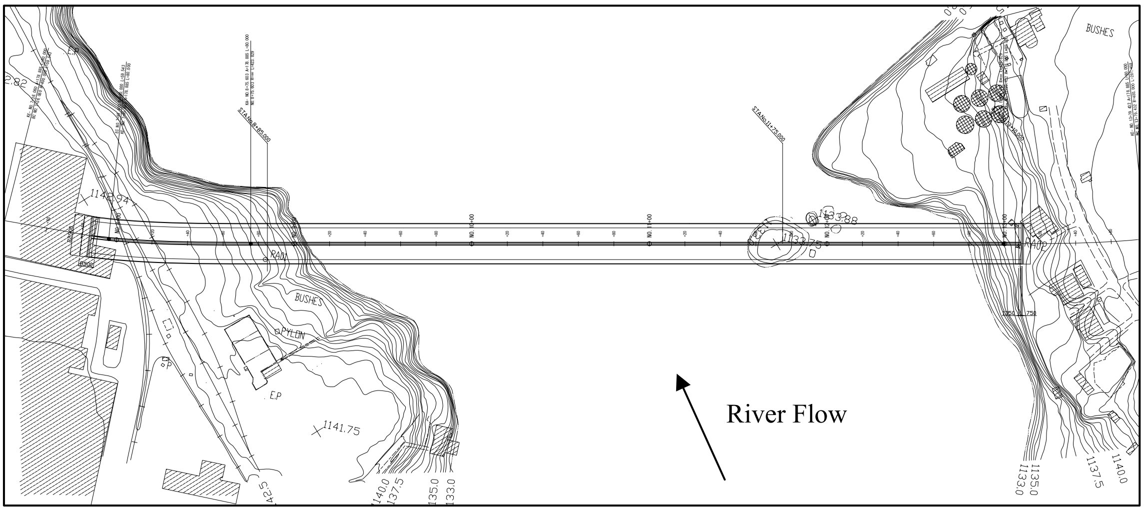
Figure 6.2.16 Turbine Operation Hydraulic Conditions

(3) Geological Conditions

1) Drilling and Trial Pit

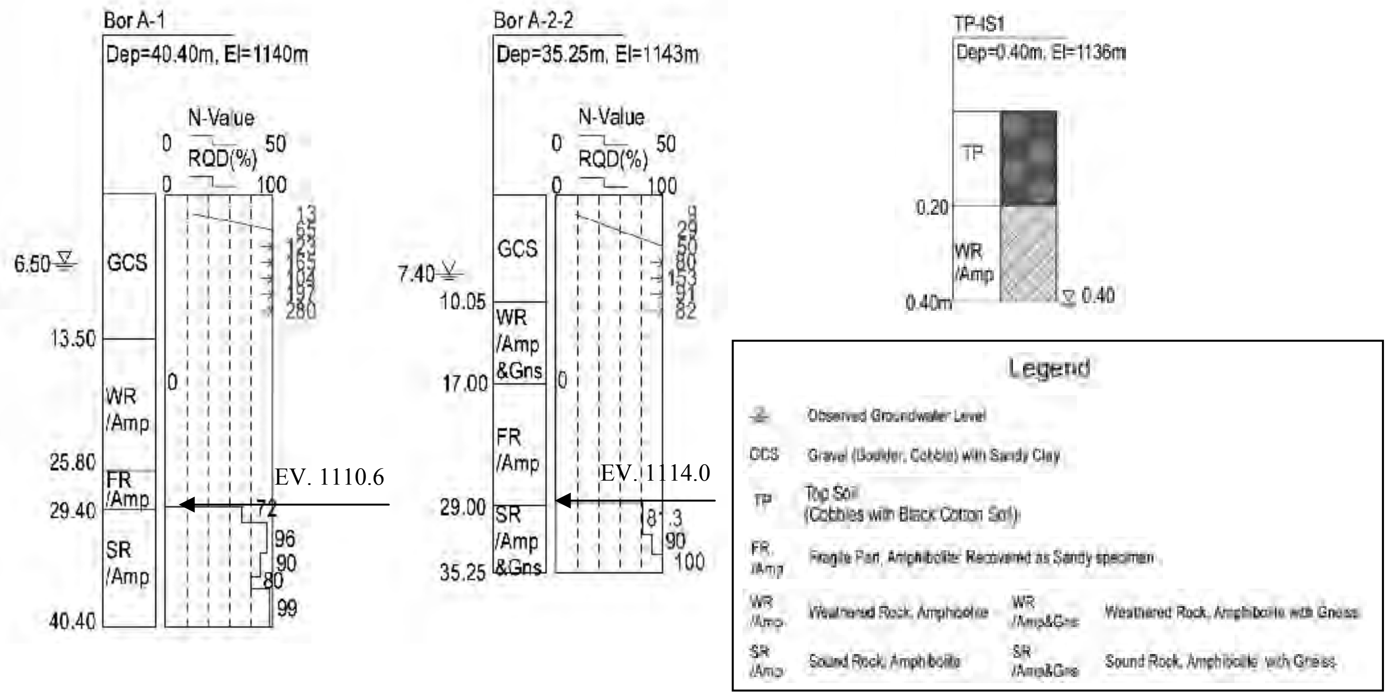
Figure 6.2.18 shows the location of the 2 trial drilling executed on both sides of the river (Bor.A-1 & Bor.A-2-2). A trial pit was excavated on the island in order to confirm the geological condition of the river Nile.

Figure 6.2.19 shows the columnar sections which summarises the results of the drilling and trial pit on the island.



Source: JICA Study Team

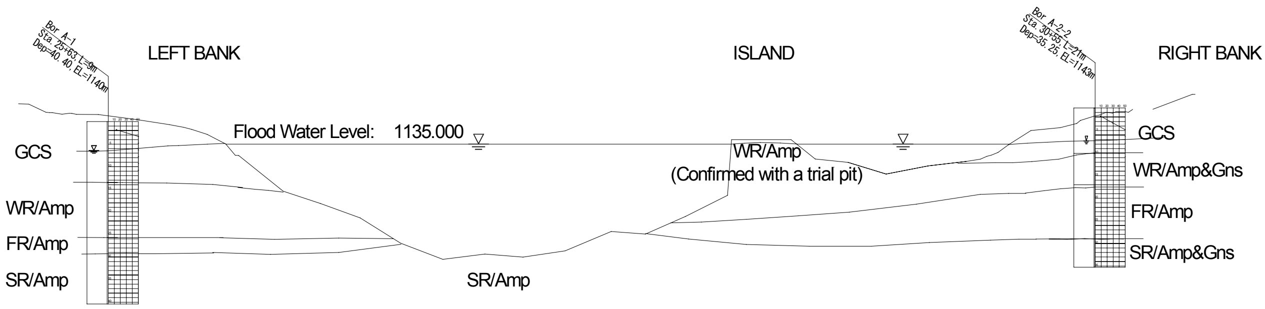
Figure 6.2.17 Location of Drilling and Trial Pit



Source: JICA Study Team

Figure 6.2.18 Boring Logs and Trial Pit

Figure 6.2.20 shows the assumed geological profiles at the Bridge Location based on the drilling and trial pit results.



Source: JICA Study Team

Figure 6.2.19 Assumed Geological Profiles

2) Design Parameters

Table 6.2.20 organizes the assumed design parameters.

Table 6.2.20 Assumed Design Parameters

Symbols for Strata	Rock (Soil) Type		γ Mg/m^3	C kN/m²	φ °	Remarks
CS	Sandy Clay		1.8	50	-	Average values from NSPT
GCS	Gravel (Boulder Cobble) with Sandy Clay		2.1	-	35	Assuming Inter Rocking Gravelly Layer
				120	-	Assuming Matrix Supported Gravelly Layer, Average values from NSPT
Lat	Lateritic Soil		2.0	110	-	Average values from NSPT
Lat/Sap	Lateritic Soil with Sapprolitic Layer		2.0	200	-	Average values from NSPT
FR/Amp	Fragile	Amphibolite	2.6	50	8	RMR10 (BIENIAWSKI) qu<5Mpa (Supposed)
HWR/Amp	Highly Weathered		2.6	50	8	RMR10 (BIENIAWSKI) qu<5Mpa (Supposed)
WR/Amp	Weathered		3.0	195	22	RMR39 (BIENIAWSKI), qu=24Mpa (Average Valuses from Lab Test)
SR/Amp	Sound Rock		3.0	265	29	RMR53 (BIENIAWSKI) qu=50Mpa (Average Valuses from Lab Test)

Source: JICA Study Team

From the Geological profile of Kiira Canal and Geological Investigation results of the JICA Study Team, it can be assumed that sound rock exist intermittently at elevations approximately between EV.1110 and 1120 m.

Based on the laboratory test results, it can be concluded that the weathered rock will be adequate as bearing layer.

3) Bridge Foundation Types

On Land (Abutments and P1)

Appropriate bearing layers of weathered amphibolite rock are found at a shallow depth so that the foundation of P1 can be set directly on the rock. As for the abutment foundation, it can be set in the GCS layer since the reaction force of the girder end is small as compared to pylon portion.

A spread foundation is one possible type for foundation in these cases.

On the Island (P2)

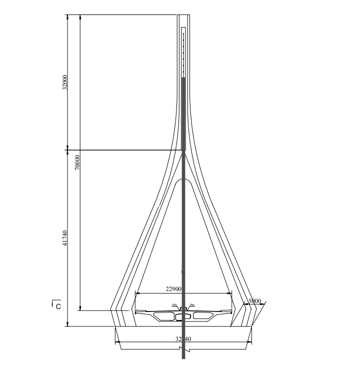
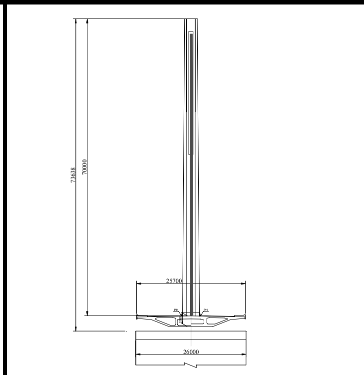
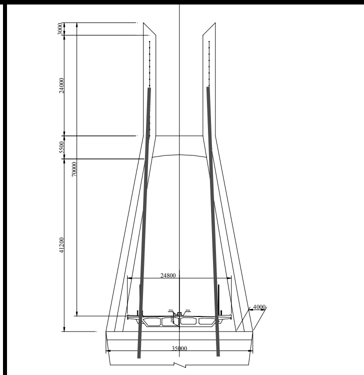
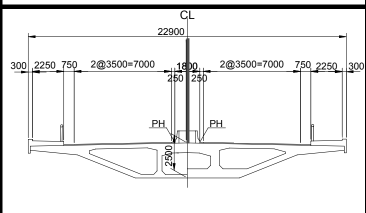
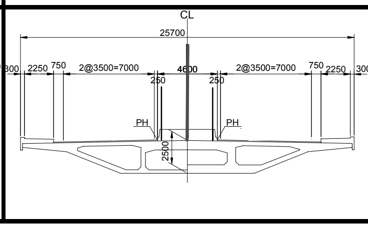
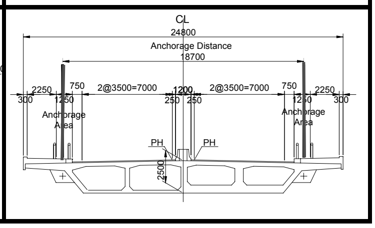
The Bridge foundation on the island is proposed to be cast-in-place concrete piles. It can be constructed without temporary cofferdam from a stable jetty and temporary platform.

(4) Basic Configurations of New Nile Bridge

The comparison of basic configurations was carried out prior to the preliminary design.

Table 6.2.21 and Figure 6.2.21 show the features of the 3 alternatives and general view respectively. Comparison was carried out in various aspects of the bridge based on these 3 alternatives.

Table 6.2.21 Characteristics of 3 Alternatives

	Alterative 1	Alternative 2	Alternative 3
Front View of Pylon a			
Pylon Shape	Inverted Y Shape	I Shape	H Shape
Stay Cable	Single Plane	Single Plane	Double Planes
Cross-section of Girder			

Source: JICA Study Team

Alternative 1 (Inverted Y Shape Pylon & Single Plane Stay Cable)

Front View

Alternative 2 (I Shape Pylon & Single Plane Stay Cable)

Front View

Alternative 3 (H Shape Pylon & Double Plane Stay Cable)

Front View

Source: JICA Study Team

Figure 6.2.20 General View of Each Alternatives

1) Comparison of the Bridges based on the User's Feeling

Table 6.2.22 shows the characteristics of feeling for bridge users.

Table 6.2.22 Characteristics of Feeling for Bridge Users

	Alternative 1	Alternative 2	Alternative 3
Front View of Pylon			
Running Surface	Open feeling	Open feeling	Confined feeling
Outside View of Cable	Simple	Simple	Interlace

Source: JICA Study Team

2) Comparison of the Design Aspects (Structural Characteristics)

<u>**Applicability of the 3 Alternatives**</u>

- First cable-stayed bridge with single plane cable and I shape pylon was constructed in Germany (1963, 172m)
- Since then, many single plane cable-stayed bridges with I type or inverted Y type pylon were constructed in the world.
- Longest cable-stayed bridge with single plane cable is:
  - Steel Bridge (Inverted Y type pylon with 510m span) was constructed in Japan 1994.
  - PC Bridge (I type pylon with 435m span length) was constructed in Vietnam 2006.
- <u>**Any types of cable and pylon are applicable to the project bridge, since the main span length is 290m.**</u>

The history of several major cable-stayed bridges with single plane cable is listed in Table 6.2.23.

Table 6.2.23 History of Major Cable-stayed Bridges with Single Plane Cable

No.	Name	Carriageway	Steel/PC	Pylon Type	Main Span Length	Completion Year	Country
1	Norderelbe	6	Steel	I	127	1963	Germany
2	Friendrich-Ebert	6 + footway	Steel	I	280	1967	Germany
3	Papineau-Leblanc	6	Steel	I	240	1969	Spain
4	Arakawa	4	Steel	I	160	1970	Japan
5	Duisburg-Neuenkamp	6 + footway	Steel	I	350	1970	Germany
6	Brotonne	4	PC	I	320	1977	France
7	Rama IX	6	Combined	I	450	1987	Thailand
8	Sunshine Skyway	4	PC	I	365.8	1988	USA
9	Tsurumi Tsubasa	6	Steel	IY	510	1994	Japan
10	Rama VIII	4 + footway	Combined	IY	300	2001	Thailand
11	Baichay	4	PC	I	435	2006	Vietnam (Japan Aid)
12	Yabekawa	4	PC	IY	261	2009	Japan

Note: IY; Inverted Y shape pylon, I; I shape pylon
Source: JICA Study Team

Other aspects of the 3 Alternatives

Table 6.2.24 shows the other structural aspects for the 3 alternatives.

Table 6.2.24 Structural Characteristics

	Alternative 1	Alternative 2	Alternative 3
Wind Resistant Stability of Girder	- Girder dimensions are examined by static wind load. - In general, a concrete girder has an advantage of dynamic resistant characteristics because of huge dead weight. - Girder of single plane cable has enough torsion rigidity		
Stability of Pylon (Seismic and wind resistance in lateral direction)	**Slightly better than Alternative 2 because of higher rigidity.**	Slightly poorer than others.	**Slightly better than Alternative 2 because of higher rigidity.**
Dia. & Nos. of Cable	72 Cables Ave. ϕ 200mm	72 Cables Ave. ϕ 200mm	144 Cables Ave. ϕ 150mm
Girder Width	**Narrowest (22.9m)**	Widest (25.7m)	Medium (24.8m)
Substructure Width	32.3 m	**26.0 m**	35.0 m

Note : The color column shows an advantage in comparison with the other alternatives
Source : JICA Study Team

3) Comarison of the Constractability

Table 6.2.25 shows the constructability for the 3 alternatives.

Table 6.2.25 Constructability for the 3 Alternatives

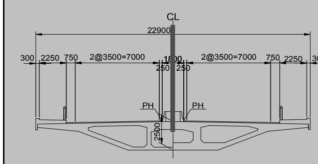
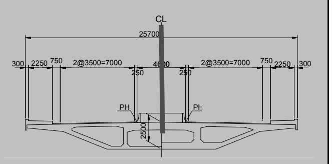
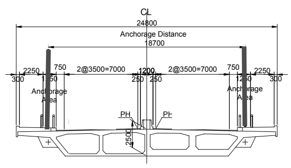
	Alternative 1	Alternative 2	Alternative 3
Girder Erection	- Cantilever erection can be applied. - One segment of girder will be around 4m long and it takes 2 weeks to erect.		
Pylon Erection	A little difficult than Alternative 2 (because of slanted pylon legs)	**Moderate (because of vertical leg)**	A little difficult than Alternative 2 (because of slanted pylon legs)
Cable Erection (Constructability)	**A little easier than Alternative 3 (because of less number of cables, although each cable diameter becomes larger than Alternative 3)**		A little difficult than the others, although each cable diameter is smaller.
Construction Period	3.5 years		
Construction Cost	**1.00**	1.02	1.03

Note : The color column shows an advantage in comparison with the other alternatives
Source : JICA Study Team

4) Comparison of the **Maintenance** aspects

Table 6.2.26 shows the maintenance aspects for the 3 alternatives.

Table 6.2.26 Maintenance Aspect for 3 Alternatives

	Alternative 1	Alternative 2	Alternative 3
Maintenance Work	**- Number of cables and anchors is half of Alterative 3.** **- Access and maintenance of stay cable anchors are easy because they are inside girder.** 		- Number of cables and anchors is twice as many as the others. - Access and maintenance of stay cable anchors are not easy because they are located on the underside of footway. 
Maintenance Cost (100 Years)	**1.00**	**1.00**	1.03

Note : The color column shows an advantage in comparison with the other alternatives
Source : JICA Study Team

5) Comparison of the **Safety** aspects

Table 6.2.27 shows the safety aspects for 3 alternatives.

Table 6.2.27 Safety Aspect for 3 Alternatives

	Alternative 1	Alternative 2	Alternative 3
Users (Vehicles)	- Vehicles are guarded from falling by concrete wall or guardrail. - It must not be expected that the falling of vehicles is protected by stay-cables.		
Users (Pedestrians)	Pedestrians can be protected from collision of vehicle by guardrail.		Pedestrians can be protected by concrete wall.
Protection of the Cables against vehicle collision	Cables are protected by concrete wall along median		Cables are protected by concrete wall between carriageway and footway
Protection of Cable from fire	Cables should be tubed into steel pipe on the bottom part of cables (around 10m from the road surface) against fire due to accident.		
Protection of cable from Vandalism	**It is impossible for pedestrians to access cables because stay cables are anchored inside the box girder under median.**		It is impossible for pedestrians to access stay cables.

Note : The color column shows an advantage in comparison with the other alternatives
Source : JICA Study Team

6) Comparison of the **Security** aspects

Table 6.2.28 shows the security aspects for the 3 alternatives.

Table 6.2.28 Security Aspect for the 3 Alternatives

	Alternative 1	Alternative 2	Alternative 3
Entire Bridge	Bridge will be guarded by army.		
Cables	It is not easy for anyone to access cables because they are installed inside median.		It is easy for pedestrian to access cables because they are installed along the footway.
Cable Anchors	Doors or hatches to entre box girder or pylon where cable anchors are installed should be checked every morning and evening.		

Note : The color column shows an advantage in comparison with the other alternatives
Source : JICA Study Team

7) Evaluation summary and recommendation

Table 6.2.29 shows the evaluation summary and recommendation for the 3 Alternatives. The construction and maintenance cost of Alternative 1 is lower than the other Alternatives, and it has some advantages in safety and security aspects in comparison with Alternative 3.

Therefore, Alternative 1 (Inverted Y Shape Pylon & Single Plane Stay Cable) should be selected as the basic configuration for the New Nile Bridge.

As the result of selection of basic configuration for the New Nile Bridge, the Steering Committee, which was supported by the Technical Committee Meeting held on 15th May, 2009, agreed that Alternative 1 (Inverted Y Shape Pylon & Single Plane Stay Cable) was the preferable basic configuration for the New Nile Bridge.

Table 6.2.29 Evaluation Summary and Recommendation

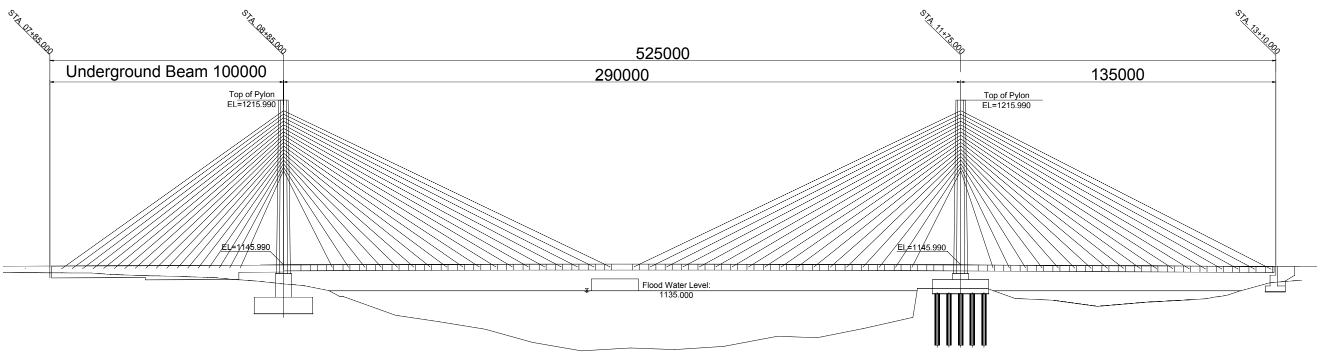
		Alternative 1	Alternative 2	Alternative 3
1) Bridge User's feeling	Driver's view	Open feeling		Confined feeling
	Outside view of cable	Simple		Interlace
2) Design (Structural characteristics)	Applicability	Applicable		
	Wind resistance	Possible		
	Stability of pylon	**Slightly better**	Slightly poorer	**Slightly better**
	Diameter/number of cable	72 Cables and Anchors Ave. φ200mm	72 Cables and Anchors Ave. φ200mm	144 Cables and Anchors Ave. φ150mm
	Width of girder	**Narrowest (22.9m)**	Widest (25.7m)	Medium (24.8m)
	Substructure width	34.0m	**26.0m**	35.0m
3) Construction (Constructability)	Girder erection	Same method		
	Pylon erection	A little difficult	**A little easier**	A little difficult
	Cable erection	**A little better**		A little poorer
	Construction period	3.5 years		
	Construction cost	**1.00**	1.02	1.03
4) Maintenance	Maintenance work	Easy to access		Difficult to access
	Maintenance cost	**1.00**	**1.00**	1.03
5) Safety	Users (vehicles)	Guarded by concrete wall or guardrail		
	Users (pedestrians)	Protected by guardrail		By concrete wall
	Protection of cable against vehicle collision	Protected by concrete wall with sufficient rigidity		
	Cable Protection from fire	Cables in 10m high are tubed into the steel pipe		
	Cable Protection from vandalism	**Impossible for pedestrians to access**		Possible for pedestrians to access
6) Security	Entire bridge	Bridge will be guarded by army		
	Cables	**Not easy for anybody to access**		Easy to access
	Cable anchors	Doors or hatches to enter box birder or pylon should be checked every morning and evening		
Recommendation		**Most Recommendable**	-	-

Source : JICA Study Team

(5) Preliminary Bridge Design

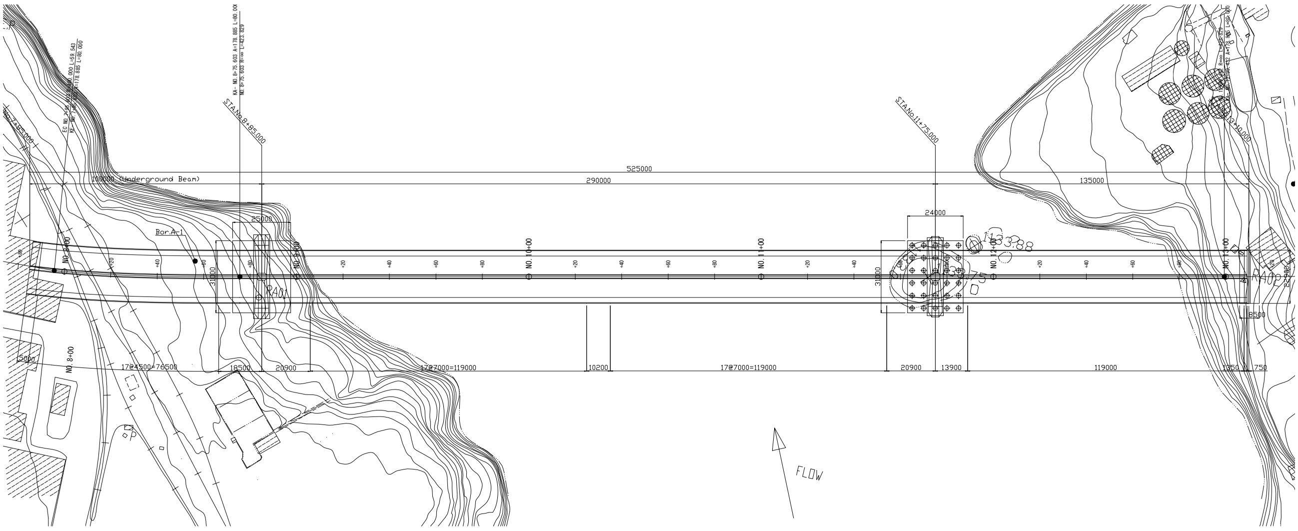
1) Bridge Layout

Three-Span continuous span arrangement, comprising of a main span of 290 m, left side span of 100 m and right side span of 135 m, with total bridge length of 525 m, has been adopted for the preliminary design for this study. P1 pylon will be located on the left bank and P2 Pylon will be located on the small island near the right bank. The profile and the plan of the bridge are shown in Figure 6.2.22 and Figure 6.2.23 respectively.



Source: JICA Study Team

Figure 6.2.21 Profile of the New Nile Bridge



Source: JICA Study Team

Figure 6.2.22 Plan of the New Nile Bridge

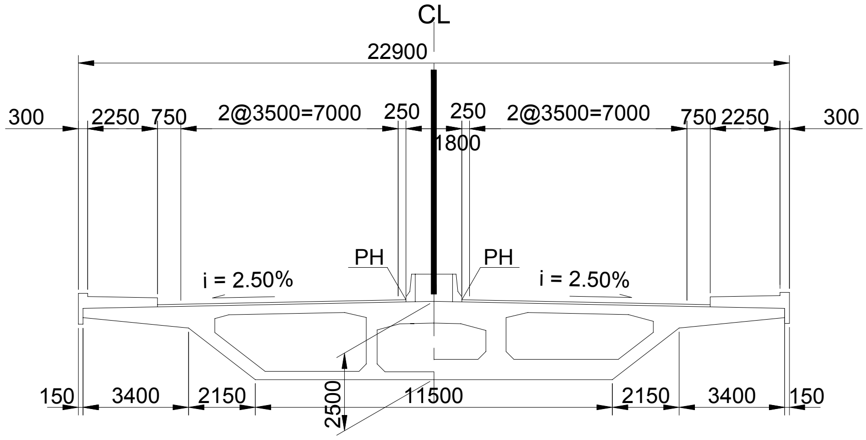
2) Superstructure

The span arrangement is shown in Figure 6.2.22. The main span of 290m has been decided upon so as not to provide piers in the river (due to the very deep depth and hard riverbed).

The left side span was set at 100m considering the horizontal alignment of radius of 400m (R=400). And the right side with a span of 135m was considered to balance the main span..

<u>Structural Dimensions</u>

The typical cross-section of the bridge is shown in Figure 6.2.24. The girder depth is 2.5m. Three-cell box girder is adopted.



Source: JICA Study Team

Figure 6.2.23 Typical Cross Section of the New Nile Bridge

Bridge Deck

Because of the deck slab is relatively wide, the slab was designed as Prestressed concrete structure.

The cables should be spaced at 7.0 m interval and the length of one construction segment of the bridge deck was established at 4.0 m (non stay-cable anchor segment) and 3.0 m (anchor segment). The bridge deck will be constructed by the cantilever method from each pylon towards the centre span and side span to balance the operation. The bridge deck could be constructed by cast-in-place concrete or pre-cast segments, and the cast-in-place concrete method will be selected by utilizing commonly used construction equipment. Deck and piers will connected by rubber bearing.

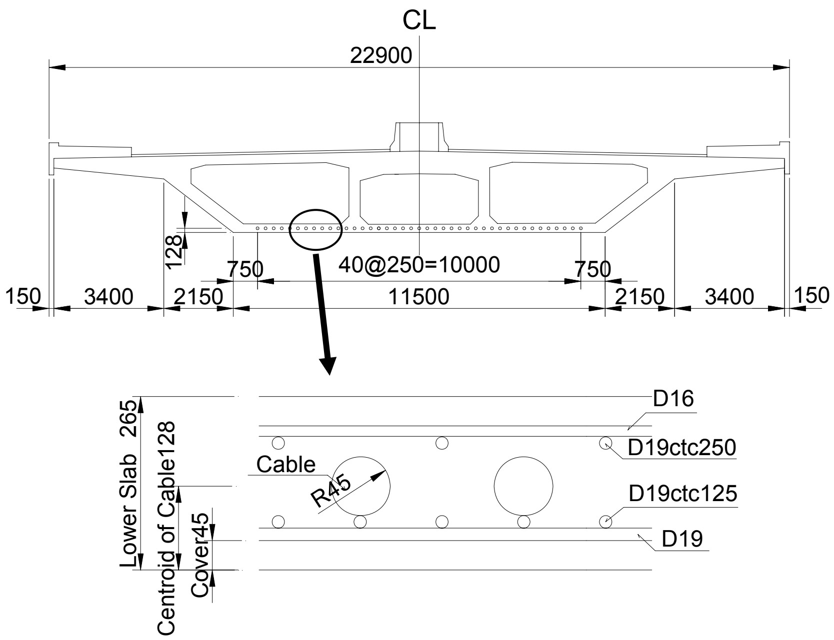
PC Cable Arrangement for Girder

Based on the result of the calculations, the numbers of PC cables for the girder are estimated as follows:

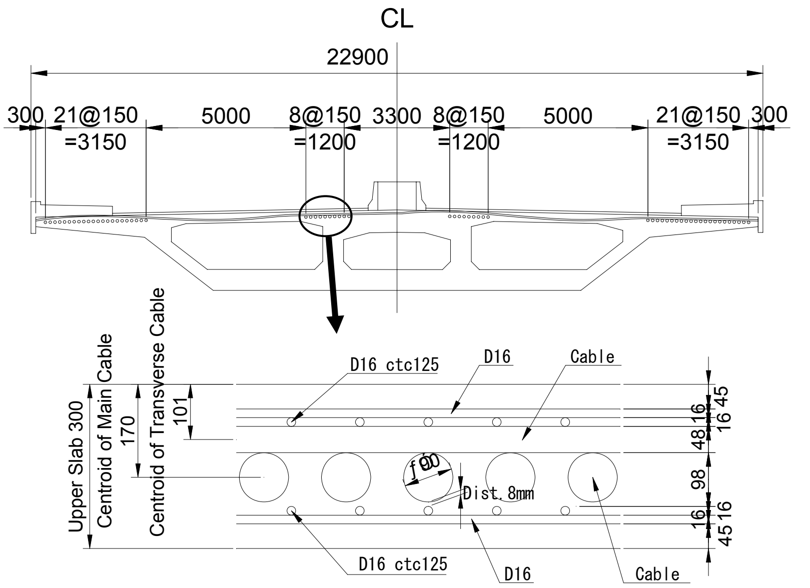
- Centre Span : 41 nos. (lower slab)
- Supprting Portion (P1 & P2) : 62 nos. (Upper slab)

Figure 6.2.25 shows the possible arrangement of the PC Cables for the girder.

<Centre Span>



< Supprting Portion (P1 & P2) >



Source: JICA Study Team

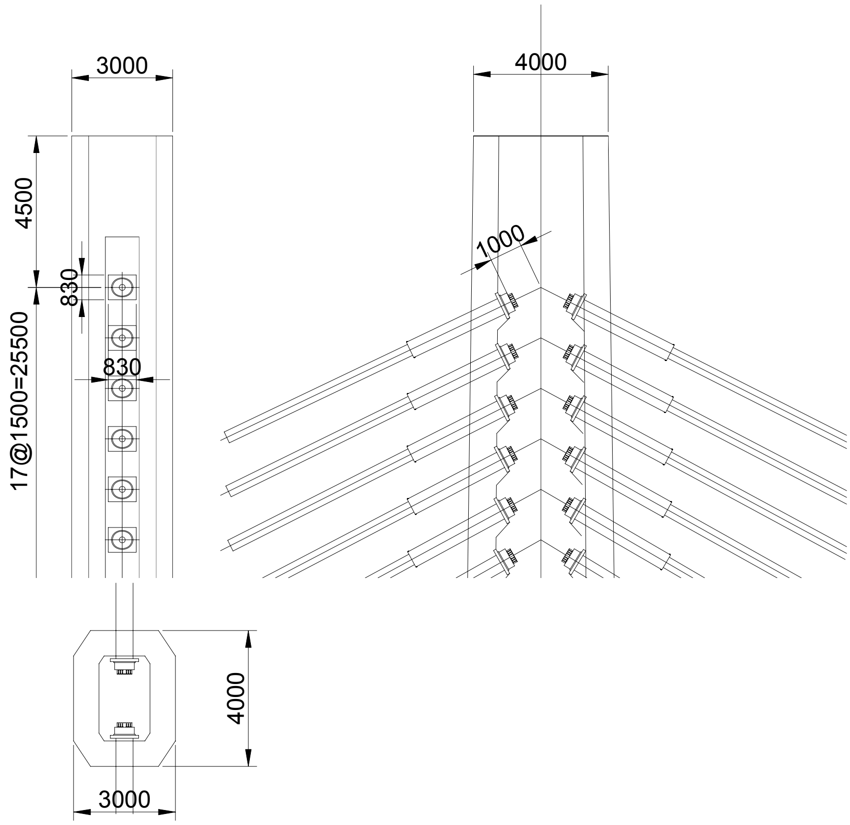
Figure 6.2.24 PC Cable Arrangement for Girder

Stay Cables and Anchors

There are two types of stay cable, the Multi Strand (MS type) and Parallel Wired Strand (PWS) type applicable to cable-stayed bridges. Multi Strand Cable consists of a bundle of parallel mono strands coated by grease for protection from friction and sheathed in a high density polyethylene (HDPE) tube. In addition, bundle of the multi strand cable are encased in high density polyethylene duct and a compound for corrosion prevention is filled in the duct. Equipment for installing the stay cable during construction at site is rather simple.

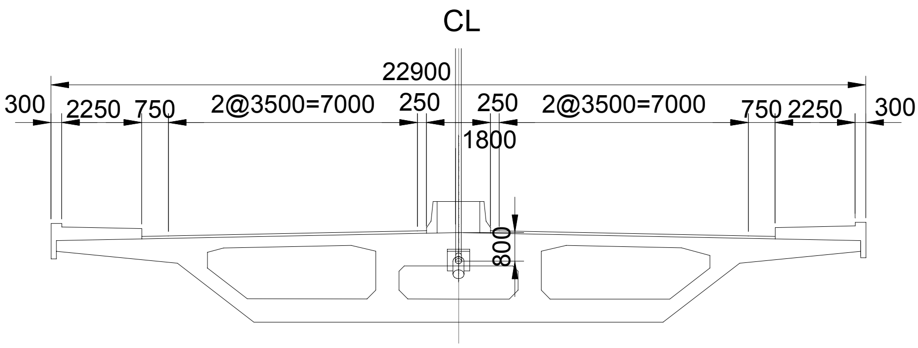
The Parallel Wire Strand will fabricate in the factory. Bundle of galvanized wire is rolled up by filament tape and sheathed in the HDPE tube. Unit of the Parallel Wire Strand stay cable is heavier than MS type cable and equipment to install the cables at the construction site will be heavier than the equipment for MS type. The MS type of stay cable is preferable to adopt for prestressed concrete cable-stayed bridge for construction in remote areas far from the factory. Therefore, the MS type of stay cable will be applied for the New Nile Bridge.

Figure 6.2.26 and Figure 6.2.27 show the anchor of the stay cable for the Pylon and girder respectively.



Source: JICA Study Team

Figure 6.2.25 Anchor of Stay Cable for Pylon

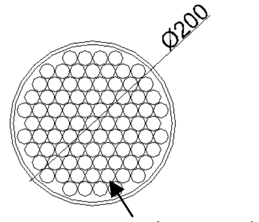


Source: JICA Study Team

Figure 6.2.26 Anchor of Stay Cable for Girder

The number of strand for the stay cable is shown in Table 6.2.30.

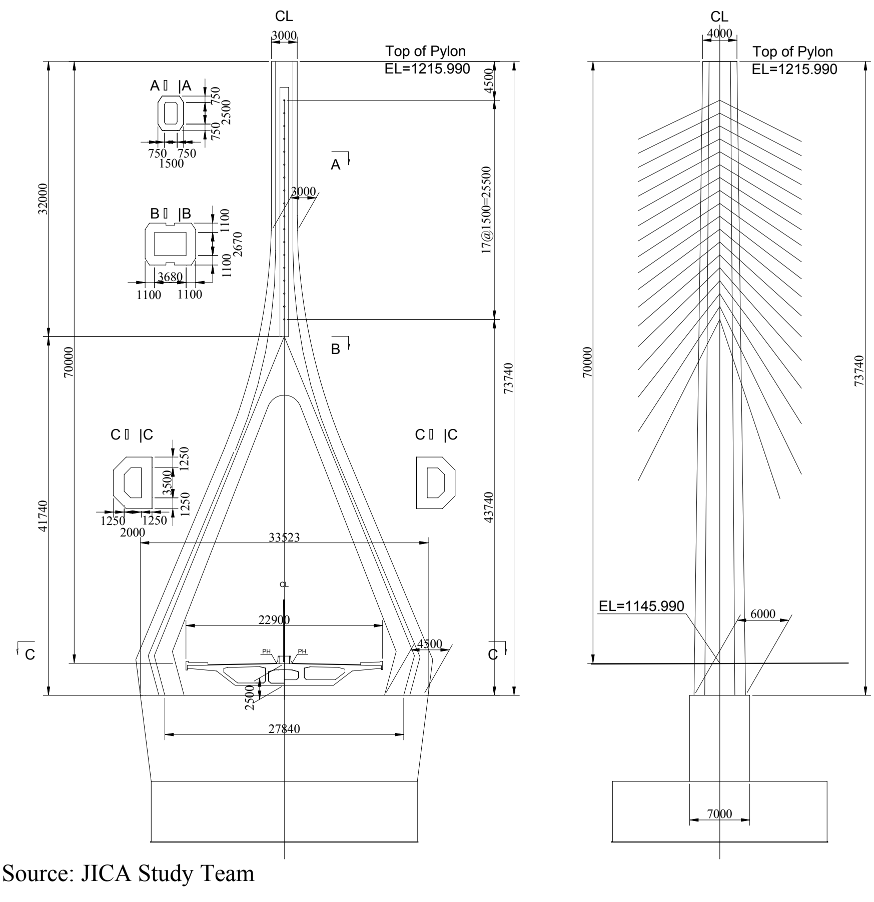
Table 6.2.30 Number of Strand for Stay Cable

	Number of Strand	Area (sq. mm.)	Remark
Maximum	97	13,400	Outside (long) cables
Minimum	59	8,120	Inside (short) cables
Average	82  The diameter is different slightly by the cable suppliers.	11,300	-

Source: JICA Study Team

Pylons

The concrete inverted Y shape pylon will be adopted for the New Nile Bridge, because this type of pylon would achieve a preferable visual impact for drivers and cost efficiency. Arrangement of the stay cables will be the semi-fan pattern type suited for the construction of cable anchors in concrete pylons. The height of the pylons at approximately 73.7 m from the surface of the pile cap, and 70 m from the road surface will adhere to aviation limitation of Jinja Airfield. A general view of the pylon is shown in Figure 6.2.28.



Source: JICA Study Team

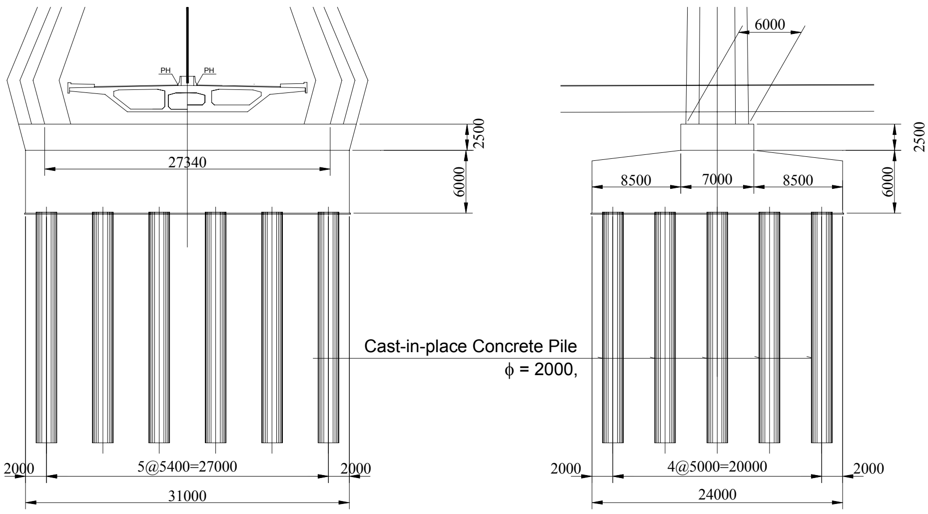
Figure 6.2.27 General View of Pylon (P1)

3) Substructure

Foundations

Based on the result of the geological and hydrological investigated conducted, appropriate bearing layers consisting of weathered amphibolite rock are found at a shallow depth on the left bank, so that foundation of P1 can be set directly on the rock. Dimension of P1 spread footing will be 25 m x 31 m x 7 m.

On the other hand, cast-in-place concrete piles partly encased with steel are considered appropriate for the foundations of P2 pylons based on the geological information and method of at the small island. The pile cap is set above normal water level to avoid underwater construction. Figure 6.2.29 shows the pile foundation of P2.



Source: JICA Study Team

Figure 6.2.28 Arrangement of Pile for P2

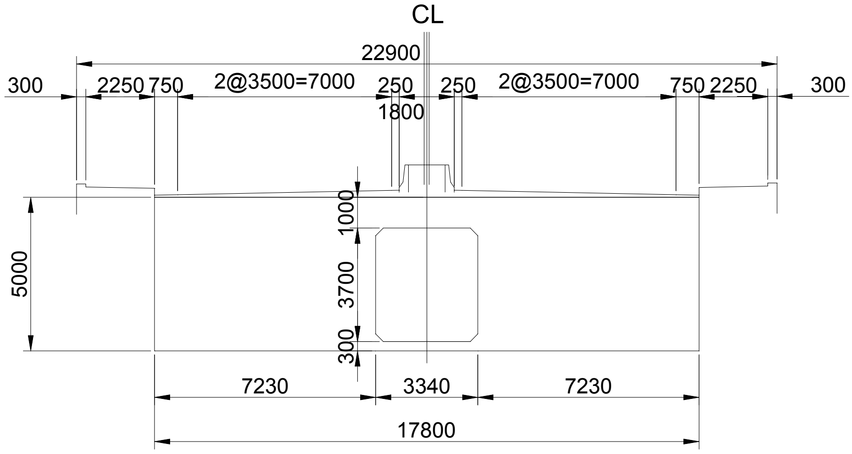
Semi-underground Beam

Since the left side 100m span is located on land, a semi-underground beam will be adopted. to balance the design bending moment for the weight of 3 m high underground beam. However, the average height of the semi-underground beam was set to 5 m because of the need it will be installed on GCS layer with relatively high N value.

Additional detail survey (ground elevation) and geological investigation of this section is recommended to be carried out during the detailed design stage.

The inner side of the semi-underground beam was designed as hollow for the anchor setting and for maintenance.

Figure 6.2.30 shows the typical cross section of the semi-underground beam.



Source: JICA Study Team

Figure 6.2.29 Typical Cross Section of Semi-underground Beam

4) Miscellaneous Facilities

Facilities for the Traffic Safety

Facilities for the safety measures against traffic accidents should be provided by the project. The major safety facilities to be provided are:

a) Lighting facilities on the bridge

To illuminate the carriageway of the bridge, the provision of lighting facilities on conventional galvanized steel pole is necessary..

b) Aircraft warning light

The pylon of the cable-stayed bridge is 70 m high which is under the aerodrome limitation for Jinja Airfield, but aircraft warning light should be installed on top of the pylon as warning lights to aircrafts flying at low level.

c) Lightning rod (conductor)

To prevent the stay cable and other bridge members from being struck by lightning, it is inevitable to provide lightning rod on top of the pylon.

Deck Drainage

The drainage system should have a sufficient capacity to drain surface water on the bridge decks efficiently. Vertical drain pipes space at appropriate intervals will be installed between the carriageway and walkway with catch-basins at each side of the bridge to drain surface water from the bridge deck.

Miscellaneous Utilities

Utilities such as water supply pipe, electrical cable and fiber optical cable are commonly placed to the deck of the bridge for the crossing of a river. However, the future plan for such facilities is not yet structured. Therefore, this will be considered for the next stage of the detailed engineering design, in consultation with concerned agencies.