The United Republic of Tanzania
Ministry of Works, Transport and Communications
Tanzania National Roads Agency

SUPPLEMENTAL STUDY FOR IMPLEMENTATION OF ARUSHA-HOLILI ROAD IMPROVEMENT PROJECT IN THE UNITED REPUBLIC OF TANZANIA

FINAL REPORT

July 2016

JAPAN INTERNATIONAL COOPERATION AGENCY

International Development Center of Japan Inc.
Oriental Consultants Global Co., Ltd.

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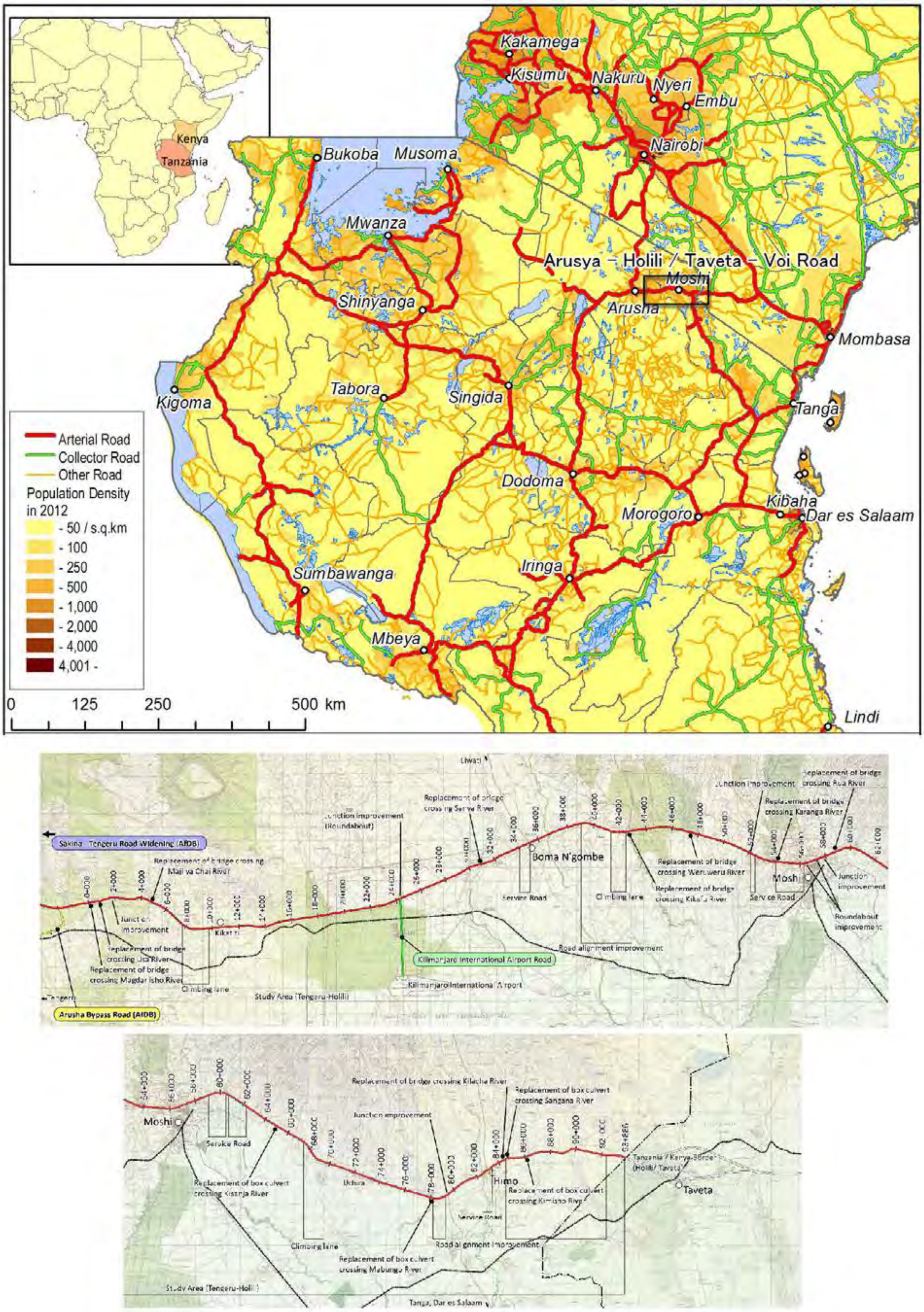
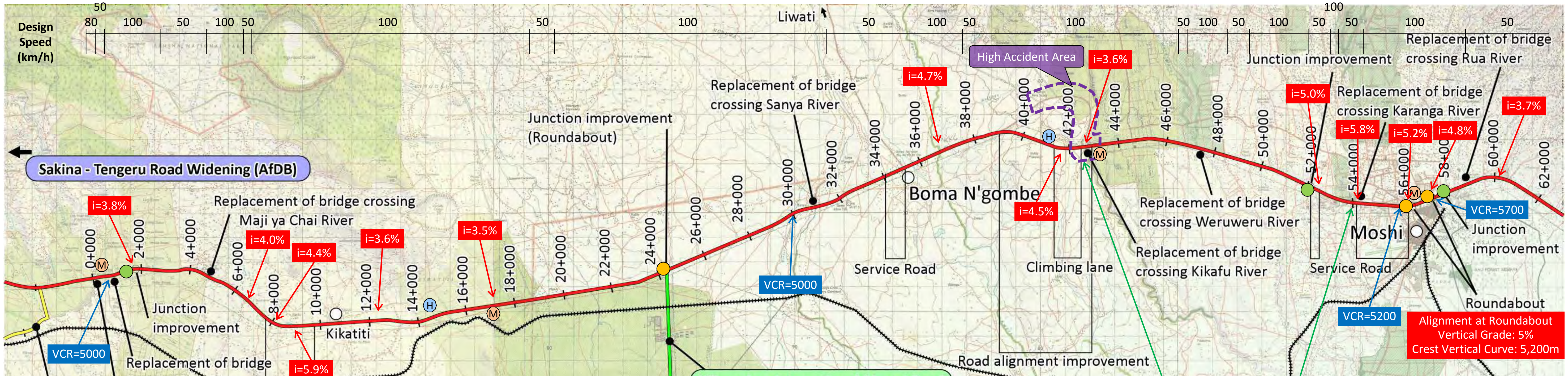
The following exchange rate is applied to the Study
1 USD = 2,192.1 TZS=109.9 JPY
(Exchange rate as of May 2016)



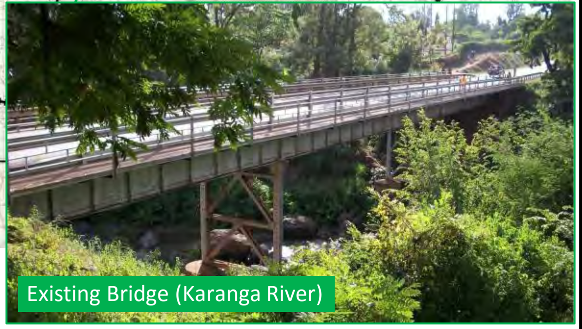
Note: A route map (shown in the middle and bottom), referred to from EAC (2011), shows Usa River and Holili section (93.9km) and the project also includes Tengeru and Usa River section (8.2km) and Kilimanjaro International Airport access road (5.7km).

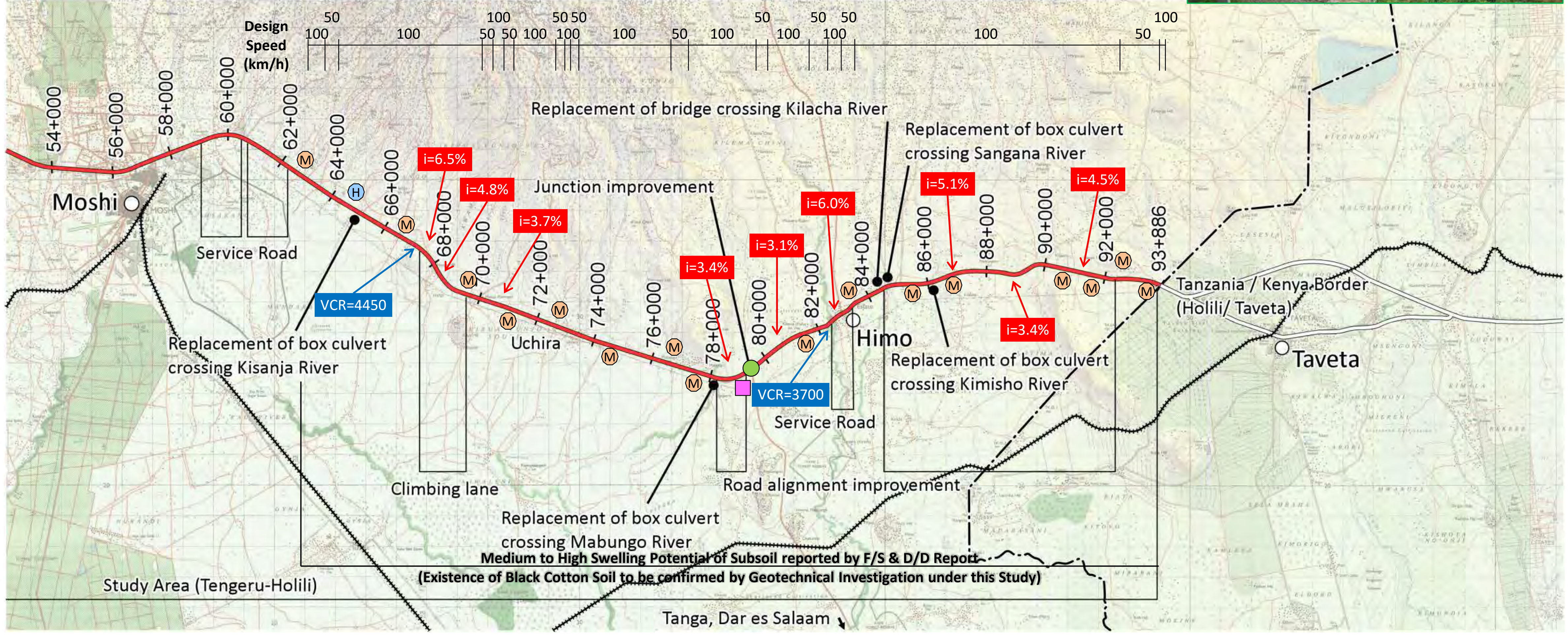
Project Location Map

Supplemental Study for Implementation of Arusha-Holili Road Improvement Project
Schematic Route Plan proposed by F/S




Existing Bridge (Kikafu River)


Existing Bridge (Karanga River)



LEGEND

- Njia Panda Weigh Station
- Roundabouts to be improved
- Other Major Intersections to be improved
- i=4.5% Vertical Grades not complying Geometric Design Standard in Tanzania
- VCR=5000 Crest Vertical Curve Radius not complying Geometric Design Standard in Tanzania
- H Swelling Potential (High)
- M Swelling Potential (Medium)

Source: F/S & D/D Report

Comparison of Geometric Design Standards for Vertical Alignment (Design Speed 100 km/h)

			Tanzania	Japan	AASHTO
Stopping Sight Distance		m	205	160	185
Max. Grade		-	3%	3% (6% in special exception case)	Flat: 3% Rolling: 4% Mountain: 6%
Min. Vertical Curve Radius	Crest	m	10,500	6,500 (preferable over 10,000)	5,200
	Sag	m	5,100	3,000 (preferable over 4,500)	4,500

Supplemental Study for Implementation of Arusha-Holili Road Improvement Project

Final Report

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Abbreviation

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
AfDB	African Development Bank
AHP	Analytical Hierarchy Process
BH	Borehole
BOD	Biochemical Oxygen Demand
BOQ	Bill of Quantity
BS	British Standards
CBR	California Bearing Ratio
CIF	Cost, Insurance and Freight
CO	Carbon Oxide
COD	Chemical Oxygen Demand
COI	Corridor of Impact
CSIR	Council for Scientific and Industrial Research
D/D	Detailed Engineering Design
dBA	Decibel
DBM	Dense Bituminous Macadam
DO	Dissolved Oxygen
DRC	Democratic Republic of Congo
DTM	Dense Tar Macadam
EAC	East African Community
EF	Equivalent Factors
EIA	Environmental Impact Assessment
EIRR	Economic Internal Rate of Return
EMA	Environmental Management Act
EMP	Environmental Management Program
ESAL	Estimation of Design Traffic Loading
ESIA	Environmental and Social Impact Assessment
F/S	Feasibility Study
FC	Foreign Currency
FOB	Free On Board
GCA	Game Controlled Area
GDP	Gross Domestic Product
GL	Ground Level
GOT	Government of Tanzania
GPS	Global Positioning System
GRDP	Gross Regional Domestic Product
HCM	Highway Capacity Manual
HDM	Highway Development and Management Tool
HGV	Heavy Goods Vehicles
HQ	Head Quarter
HRA	Hot Rolled Asphalt
HWL	High Water Level
IEE	Initial Environmental Evaluation
IR	Income Restoration
IRI	International Roughness Index
IUCN	International Union for Conservation of Nature and Natural Resources
JICA	Japan International Cooperation Agency
KIA	Kilimanjaro International Airport
kN	Kilo Newton
LC	Local Currency
LGAs	Local Government Agencies
LOS	Level of Service
LRFD	Load and Resistance Factor Design
MDG	Medium Development Goals
MGV	Medium Goods Vehicles
MOF	Ministry of Finance
MOWTC	Ministry of Work, Transport and Communication
MP	Master Plan
MPa	Mega Pascal
MR	Resilient Modulus
NA	Normal Traffic Loading
NACP	National HIV/AIDS Control Program
NB	Abnormal Traffic Loading
NEMC	National Environment Management Council
NEP	National Environment Policy
NGO	Non-Governmental Organization
NOx	Nitrogen Oxides
NPRS	National Poverty Reduction Strategy
NPV	Net Present Value
O&M	Operation and Maintenance
OD	Origin and Destination
ODA	Official Development Assistance
PAH	Project Affected Household
PAPs	Project Affected Persons
PC	Prestressed Concrete
PCU	Passenger Car Unit
PM	Particular Matter
PO-RALG	President's Office Regional Administration and Local Government
PPP	Public–Private Partnership
PSI	Present Serviceability Index
R/A	Roundabout
RAP	Resettlement Action Plan
RMR	Rock Mass Rating
RoW	Right of Way
RQD	Rock Quality Designation
SADC	Southern African Development Community
SCF	Standard Conversion Factor
SD	Deformed Bars
SMU	Social Management Unit
SN	Structural Number
SPT	Standard Penetration Test

Formalization Program TACAIDS Tanzania Commission for AIDS

TANROADs Tanzania National Roads Agency
TAWIRI Tanzania Wildlife Research Institute
TDS Total Dissolved Solids
TLC Traffic Load Classes
ToR Terms of Reference
TPBP Tanzania`s Property and Business
TSS Total Suspended Solids
TZS Tanzania Shilling
UAV Unmanned Aerial Vehicle
UCS Unconfined Compressive Strength
UTM Universal Transverse Mercator
VAT Value Added Tax
VEF Vehicle Equivalent Factors
VHGV Very Heavy Goods Vehicles
VOC Vehicle Operating Cost
WHO World Health Organization
WHO-GPA World Health Organization
Global Program on AIDS
WWF World Wide Fund for Nature

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CHAPTER 1 INTRODUCTION

1.1 Background of the Study

The Arusha – Holili/ Taveta – Voi Road is an international truck road that stretches from Arusha, the second largest city in Tanzania, to Voi, a city along the Northern Corridor, through Holili/ Taveta, the border to Kenya. The Arusha – Holili/ Taveta – Voi Road, when improved, will function as an alternative route to the Northern Corridor, connecting Mombasa to Nairobi as well as neighbouring inland countries. This will increase the use of the said corridor hence increasing capacity for international freight/passenger traffic. Currently, the unpaved road section between Taveta and Voi is being improved and which will provide a freight transport route connecting Mombasa Port, Tanzania and its neighbouring inland countries through Holili/ Taveta. In this regard, improvement of the Arusha – Holili road section is essential.

The Feasibility Study and Detailed Engineering Design of the Multinational Arusha – Holili/ Taveta – Voi Road (hereinafter referred to as F/S or F/S report and D/D or D/D report) was carried out in 2011 by the East African Community (EAC). Following the said study, the road section between Mwatate and Voi (23.5 km) in Kenya is being improved by the Kenyan Government. The road section between Mwatate and Taveta (91.1 km) in Kenya, Sakina and Tengeru (14.1 km) and Arusha Bypass (42.4 km) are currently under implementation with financial assistance from the African Development Bank (AfDB).

Japan International Cooperation Agency (JICA) is currently considering improvement of the remaining Tengeru - Holili section (102.1 km) and access road to Kilimanjaro International Airport (5.7 km) as a loan project. This will follow the project approval process, to fulfil the requirements set by JICA's manuals and guidelines. Therefore, the Study aims at confirming the feasibility of the road improvement project between Tengeru and Holili and access road to Kilimanjaro International Airport.

1.2 Objectives and Contents of the Study

The objective of the Study is to confirm the viability of road improvement between Arusha (Tengeru) and Holili and the access road to Kilimanjaro International Airport, as stated above through revision works on F/S and D/D of the Multinational Arusha – Holili/ Taveta – Voi Road as well as conducting supplementary studies to fulfil requirements set to meet JICA's financial support criteria.

1.2.1 Study Area

The area of the Study covers Tanzania and neighbouring countries, which include Kenya, for the traffic study and proposed road alignment between Tengeru (Arusha Region) and Holili (Kilimanjaro Region) and access road to Kilimanjaro International Airport for the preliminary engineering and environment study.

1.2.2 Target Year

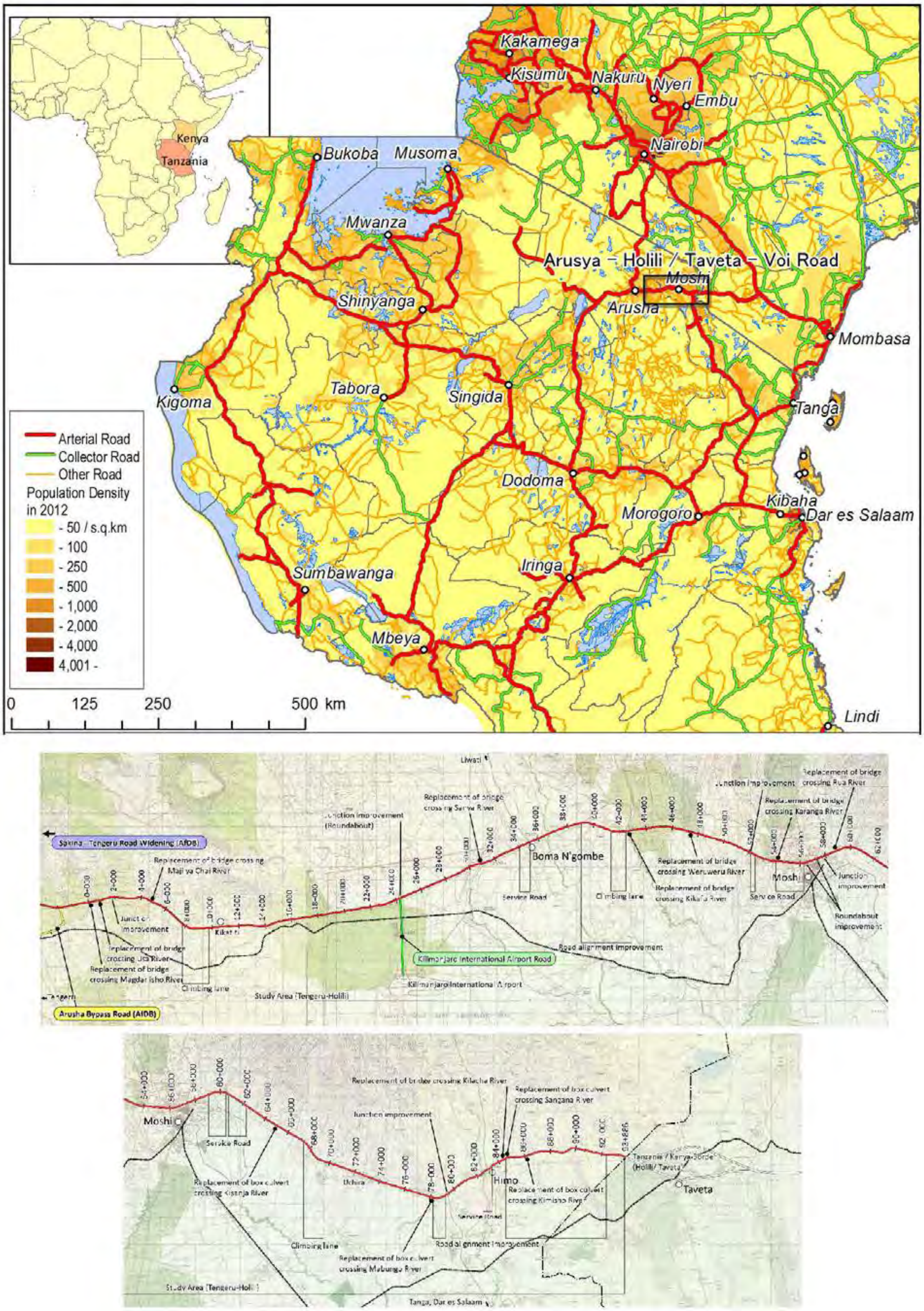
This Study tests the feasibility of the Project with the target year of 2035.

1.2.3 Executive and Implementing Agencies

Ministry of Works, Transport and Communications, TANROADs (HQ, Arusha Region and Kilimanjaro Region)

1.3 Structure of Final Report

This Final Report contains ten chapters. Chapter 2 overviews a road sector in Tanzania and justifies necessity and rational of the Project. Chapter 3 discusses and summarizes issues and findings through a preliminary review on the previous F/S and applies study approaches to confirm viability of the Project. Chapter 4 estimates future traffic demand of the project road, using available data obtained from empirical studies and traffic survey conducted in the Study. Chapter 5 explores an engineering study of alternative road and bridge plan, comparing those initially proposed in the previous F/S. It then suggests an optimum road design and bridge plan of new Kikafu Bridge, using a multi-criteria analysis. Chapter 6 then studies an implementation plan, considering packaging of the Project. Chapter 7 discusses a result of preliminary environmental study on the Project, identifying adverse impact derived from the Project. Chapter 8 then makes preliminary cost estimates of the Project. Chapter 9 justifies economic verification of the Project and Chapter 10 then concludes the Study and recommends toward a successful implementation of the Project.



Source: JICA Study Team

Figure 1.3.1 Study Area

CHAPTER 2 PROJECT RATIONAL

2.1 Road Sector in Tanzania at Glance

2.1.1 Road Network and Condition

The road network in Tanzania, totalling to 86,574 km (as of 2014), are classified into five road classes, namely, Trunk Roads, Regional Roads, District Roads, Feeder Roads and Urban Roads, by the road administer. Trunk Roads and Regional Roads are managed by TANROADS and other local roads are managed by Local Government Agencies (LGAs).

Table 2.1.1 2014 Road Network in Tanzania (Unit: km)

Road Class	Total	Administered by
Trunk Roads	12,203	TANROADS
Regional Roads	22,130	
Sub-total	34,333	
District Roads	25,113	LGAs
Feeder Roads	21,191	
Urban Roads	5,937	
Sub-total	52,241	
Grand Total	86,574	

Source: TANROADs and PO-RALG

The road condition of national roads marks significant differences between Trunk Roads and Regional Roads. The trunk road network has been the investment priority and, as a result, 54% of Trunk Roads have been upgraded to bitumen standard and 74% of these paved Trunk Roads are in good condition as of 2014. In contrast to Trunk Roads, 95% of Regional Roads are still left unpaved and only 28% of these unpaved Regional Roads are evaluated in good condition.

Table 2.1.2 2014 Road Network under TANROADS (Unit: km)

Road Class	Paved	Unpaved	Total	% of Paved
Trunk Roads	6,540	5,663	12,203	54%
Regional Roads	1,109	21,021	22,130	5%
Total	7,649	26,684	34,333	22%

Source: TANROADs

Table 2.1.3 2014 Condition of Roads under TANROADS

Road Class	Condition	2012	2013	2014
Trunk Road (Paved)	Good	67%	67%	74%
	Fair	25%	25%	19%
	Poor	8%	8%	7%
Trunk Road (Unpaved)	Good	35%	33%	34%
	Fair	53%	55%	34%
	Poor	12%	12%	32%
Regional Road (Paved)	Good	48%	48%	57%
	Fair	45%	45%	33%
	Poor	7%	7%	10%
Regional Road (Unpaved)	Good	29%	31%	28%
	Fair	53%	53%	58%
	Poor	18%	16%	14%

Source: TANROADs

2.1.2 Road Demand – Registered Vehicle

The following table tabulates the number of registered vehicle by vehicle type between 2010 and 2014. This illustrates that motorcycle had a rapid growth in numbers and reached around 860,000 by 2014, followed by light passenger vehicle which reached around 460,000 by the same period. The number of heavy vehicles and buses also shows a rapid increase of around 10 – 20% per annum.

Table 2.1.4 Registered Vehicle in Tanzania (Unit: nos)

Vehicle Type	2010	2011	2012	2013	2014	Average Growth Rate per Annum
Motorcycle (less than 3 wheels)	322,239	447,647	554,265	686,863	862,026	28%
Light Passenger Vehicle (Less than 12 persons)	266,586	301,734	337,381	390,457	460,765	15%
Heavy Load Vehicle (Over 3,500Kg)	60,653	66,513	72,613	79,658	89,757	10%
Light Load Vehicle (3,500Kg or Less)	57,253	62,515	67,419	74,543	85,398	11%
Motor Tricycle	6,477	11,677	19,031	32,505	55,149	71%
Heavy Passenger Vehicle (12 or more persons)	36,373	39,179	41,212	44,691	51,063	9%
Trailer	14,860	17,659	21,084	24,976	31,265	20%
Agricultural Tractor	10,563	11,976	13,827	15,190	17,295	13%
Construction Equipment	3,575	4,627	5,430	6,186	7,258	20%
Agricultural Trailer	165	230	367	856	904	59%
Others	689	1,101	1,542	1,908	3,306	49%
Total	779,433	964,858	1,134,171	1,357,833	1,664,186	21%

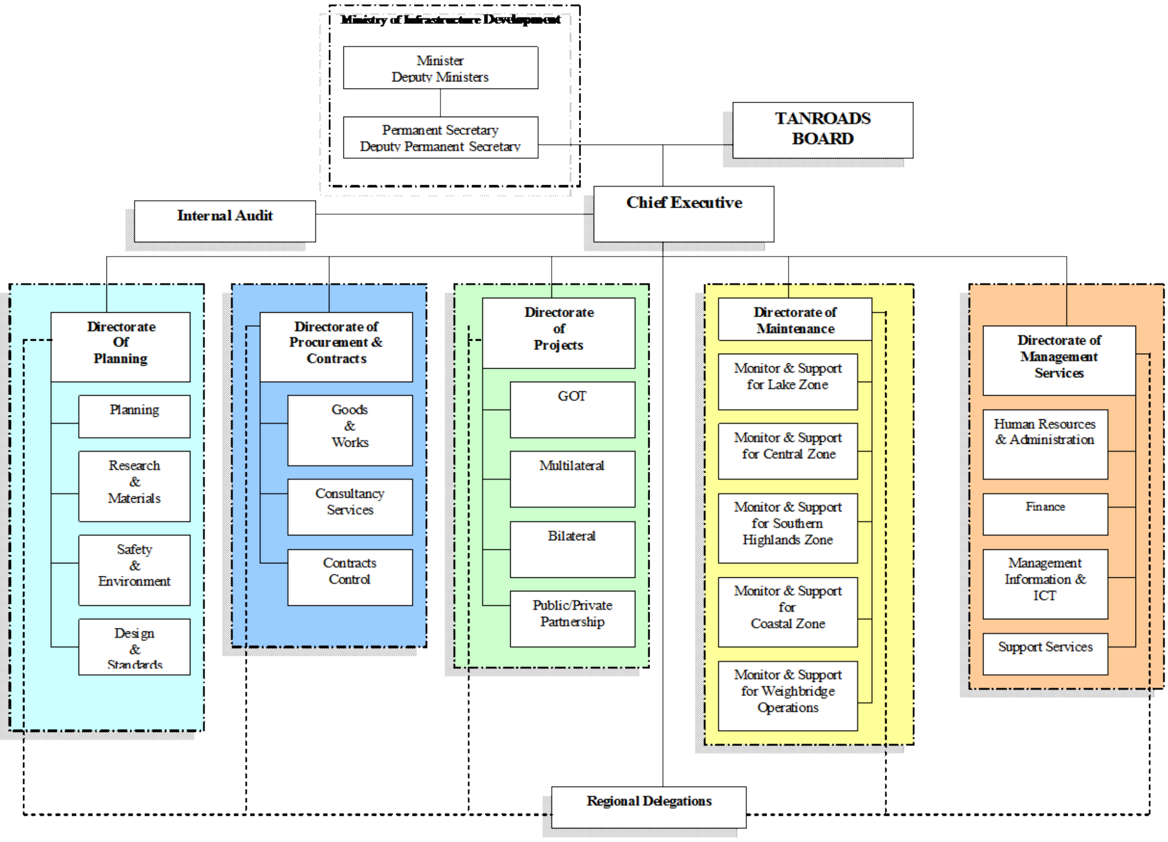
Source: Tanzania Revenue Authority

2.1.3 Road Administration

(1) Organizational Structure

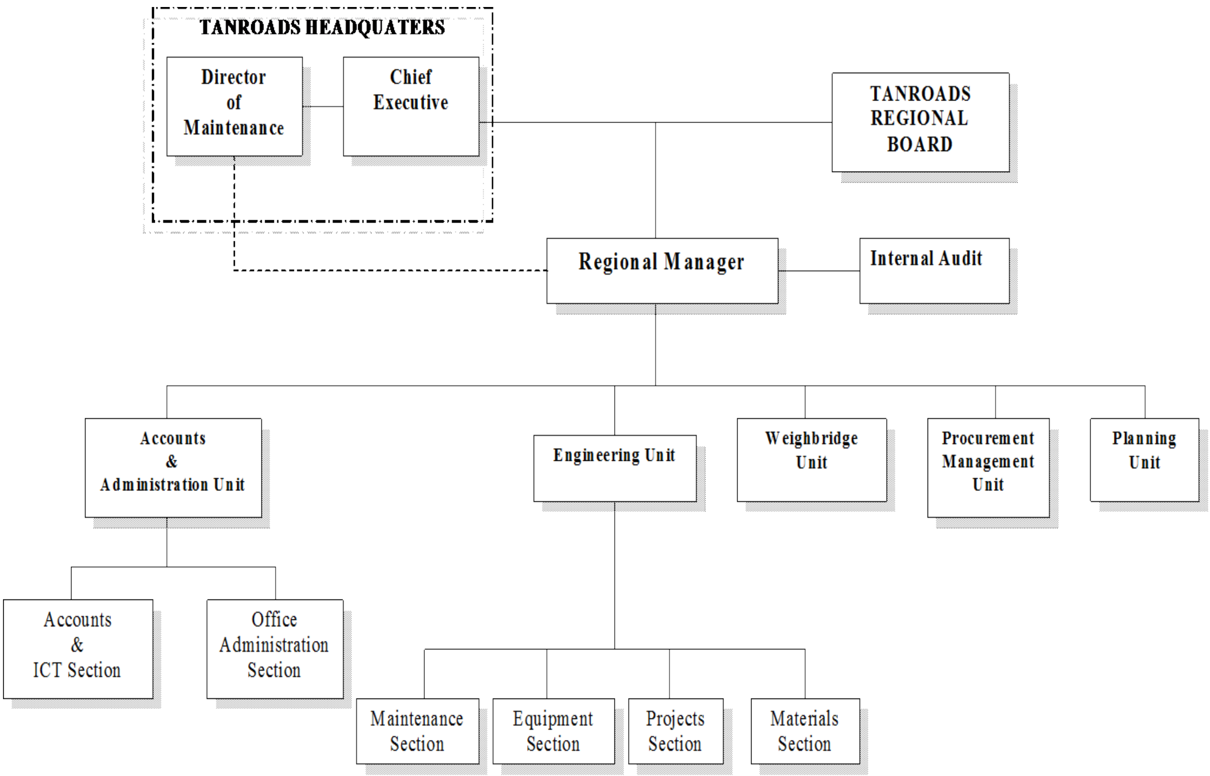
TANROADS was set up in July 2000 as a semi-autonomous agency under the Ministry of Works, Transport and Communications. It manages around 34,000 km of Trunk and Regional Roads, with 697 staff comprising of 492 technical skilled staff and 205 supporting staff (as of March, 2011).

The agency, headed by a Chief Executive, has six functional Directorates namely: Maintenance, Development, Planning, Procurement, Projects and Management Services. There are 21 regions managed by Regional Managers. The Regional Managers and heads of Legal and Internal Audit Units report directly to the Chief Executive.



Source: TANROADs

Figure 2.1.1 Organizational Structure in TANROADS HQ



Source: TANROADs

Figure 2.1.2 Organizational Structure in TANROADS Regional Office

(2) Project administered by TANROADS

There are a number of projects administered by TANROADS. In 2013/14 alone, 13 road development works contracts were signed with a total value of 674 billion TZS and a road length of 770 km. Over 1,000 road maintenance works for Trunk and Regional Roads are also managed by TANROADS with a total amount of 259 billion TZS.

Table 2.1.5 Projects administered by TANROADS

S/N	Descriptions	Unit	FY 2011/12	FY 2012/13	FY 2013/14
1	Road Development Works Contracts	No. of Contracts	5	11	13
		Value (Billion TZS)	407	311	674
		Km	NA	263	770
2	Road Maintenance Works Contracts	No. of Contracts	1,154	1,033	1,093
		Value (Billion TZS)	190	227	259
3	Supervision Contracts	No. of Contracts	5	3	13
		Value (Billion TZS)	12	2	27
		Km	287	469	621
4	Studies and Design Contracts	No. of Contracts	13	4	14
		Value (Billion TZS)	8	0	19
		Km	1,196	309	2,164
5	Procurement of Goods	No. of Contracts	117	7	9
		Value (Billion TZS)	34	1	1

Source: TANROADs

2.1.4 Road Financing

(1) Road-related Finance

The source of funds for road improvement and maintenance of the Trunk Roads and Reginal Roads are (i) Roads Fund, (ii) GOT (Ministry of Work, Transport and Communication: MOWTC) and Donor Funding. The following table illustrates financial performance of TANROADS and TANROADS budgets/expenses of nearly 1,000 billion TZS in recent years for road maintenance (by Roads Fund) and road improvement (by GOT and Donor Funding).

Table 2.1.6 2011/12 - 2013/14 Financial Performance in TANROADS (Unit: Billion TZS)

Source of Revenue	FY 2011/12		FY 2012/13		FY 2013/14	
	Budget	Actual	Budget	Actual	Budget	Actual
Roads Fund - Maintenance	259	206	268	274	315	263
Roads Fund - Development	19	16	25	25	29	23
MOWTC - Consolidated Funds for Development	610	653	280	450	431	356
MOWTC - Payroll Costs	13	9	13	9	9	11
Other Income	9	7	1	3	3	0
Direct Donor Funding	NA	NA	397	347	394	444
Total Income	910	891	984	1,108	1,181	1,097

Source: TANROADs Quarterly Progress Reports

(2) Road Maintenance Finance

The Roads Fund and Roads Fund Board functions to ensure collection of funds from selected sources and disbursement of funds to implementing agencies. Following the Roads and Fuel Tolls Act, nearly 70% of total Roads Fund is allocated for Trunk and Regional Roads and 30% for District, Feeder and Urban Roads.

The sources of the revenue for the Roads Fund are the fuel levy, transit charges and overloading fees. In the last five years, the amount of the Roads Fund increased sharply from 10% to 43% p.a., and reaches 640 billion TZS in 2013/14 and fuel levy contributes more than 96% to the Fund in the same year.

Table 2.1.7 2009/10 - 2013/14 Revenue of Roads Fund

Year	Fuel Levy		Transit Charges		Overloading Fees		Total	Growth Rate
	Billion TZS	Share	Billion TZS	Share	Billion TZS	Share	Billion TZS	% p.a.
2009/10	256.1	96%	3.7	1%	6.8	3%	266.6	
2010/11	314.8	97%	4.7	1%	6.3	2%	325.8	22%
2011/12	390.8	96%	5.8	1%	9.4	2%	406.0	25%
2012/13	434.5	97%	5.4	1%	7.9	2%	447.8	10%
2013/14	626.0	98%	6.5	1%	8.6	1%	641.1	43%

Source: Roads Fund Board

2.1.5 Road Maintenance

The following tables summarize maintenance performed by TANROADS between 2011/12 and 2013/14. Kilometre wise, majority of maintenance works are routine maintenance, followed by periodic maintenance and spot improvement works. In monetary terms, most of maintenance funds are spent for periodic maintenance, followed by routine maintenance and spot improvement works.

In 2011/12, TANROADS implemented nearly 100% of maintenance works as planned, however, percentage of the actual maintenance against planned maintenance works is dropping and 60% of planned maintenance works is completed in 2013/14.

Table 2.1.8 2011/12 - 2013/14 Maintenance Performance (1)

Description	Unit	FY 2011/12	FY 2012/13	FY 2013/14
Road Maintenance - Planned	Routine (km)	27,470	28,833	30,656
	Periodic (km)	2,571	3,449	4,041
	Spot Improvement (km)	514	932	1,091
	Total (km)	30,555	33,214	35,788
Road Maintenance - Actual	Total (km)	30,380	27,584	21,604
% covered		99%	83%	60%
Bridge Maintenance - Planned	Nos	2,324	2,417	2,577
Bridge Maintenance - Actual	Nos	2,345	2,007	1,544
% covered		101%	83%	60%

Source: TANROADS

Table 2.1.9 2011/12 - 2013/14 Maintenance Performance (2)

Description	Unit	FY 2011/12	FY 2012/13	FY 2013/14
Road Maintenance - Planned	Routine (Billion TZS)	39	53	58
	Periodic (Billion TZS)	91	130	155
	Spot Improvement (Billion TZS)	13	16	21
	Total (Billion TZS)	143	199	234
Road Maintenance - Actual	Routine (Billion TZS)	35	41	38
	Periodic (Billion TZS)	75	93	97
	Spot Improvement (Billion TZS)	11	13	14
	Total (Billion TZS)	121	147	149
% covered		85%	74%	64%

Source: TANROADs

In addition to the routine and periodic maintenance, backlog rehabilitation is also undergoing mainly in the Trunk Roads. These backlog rehabilitation works are funded by both GOT and Donor Funding.

Table 2.1.10 Backlog Maintenance as of June 2014

Project Name	Length (km)	Status as of June 2014	Financer
Iringa - Mfinga	69	Rehabilitation completed	DANIDA mixed credit financing
Kitumbi - Segerea - Tanga	120	Rehabilitation completed	DANIDA
Isaka - Lusahunga	132	Rehabilitation completed	GOT
Ushirombo - Lusahunga	110	50% in progress	GOT
Mwanza/Mara Boarder - Musoma	85	94% in progress	GOT
Arusha - Minjingu	103.3	99% in progress	IDA
Same - Mkumbara - Korogwe	NA	NA	NA
Korogwe - Mkumbara	76	87% in progress	IDA
Mkumbara - Same	96	42% in progress	IDA
Arusha - Namanga	104	Completed	ADB/JICA/GOT
Arusha - Moshi - Holili/Taveta	140	Sakina - Tengeru and Arusha Bypass in progress	ADB/GOT
Mafinga - Makambako - Igawa	142	In progress	IDA

Source: TANROADs

2.1.6 Road Maintenance Needs

Roads Fund Board, TANROADS and PO-RALG prepared a 5-year investment plan, projecting future maintenance needs and revenue to be collected. The following tables illustrate the projected amount of funds by funding source, available funds for maintenance and actual maintenance needs in the coming 5 years.

The Roads Funds expects to collect increasing amount of funds, mainly those from fuel levy, at an average growth rate of 13% per annum thus collect 1,340 billion TZS in 2018/19. TANROADS and PO-RALG estimate the future maintenance needs to reach 1,430 billion TZS in the same year. Although the maintenance needs, including backlog rehabilitation needs, exceeds the amount of the funds collected, the gap between revenue and future needs is expected to become marginal every year.

Table 2.1.11 Projection of Roads Fund Revenue (Unit: Billion TZS)

Source of Funds	2014/15	2015/16	2016/17	2017/18	2018/19
Fuel Levy	754.1	884.8	1,019.8	1,121.8	1,234.0
Transit Charges	7.3	8.4	9.8	11.4	13.2
Overloading Fees	7.8	7.8	8.1	8.4	8.9
Goods Vehicle License	14.8	17.8	21.5	26.0	31.6
Vehicle Inspection Fees	18.9	22.2	26.2	31.1	36.8
Motor Vehicle Insurance Levy	8.3	9.7	11.4	13.3	15.5
Total Revenue	811.2	950.8	1,096.7	1,211.9	1,340.0

Source: Roads Fund Board

Table 2.1.12 Projection of Maintenance Needs (Unit: Billion TZS)

Descriptions	2014/15	2015/16	2016/17	2017/18	2018/19
Total Revenue	811.2	950.8	1,096.7	1,211.9	1,340.0
Deduction by SUMATRA, Police, etc.	22.0	25.9	30.2	34.1	38.7
Total Expenditure for Road Agencies	789.2	924.9	1,066.5	1,177.8	1,301.4
Total Maintenance Needs (including Backlog)	1,282.8	1,361.1	1,249.2	1,326.0	1,413.2
Financial Deficit	-572.6	-528.7	-289.3	-266.0	-241.9

Source: Roads Fund Board

2.2 Road Development Policy

2.2.1 Vision 2025

Vision 2025 was launched in 1999 and its primary goal was that the country would have attained middle-income status, with per capita income of 3,000 USD, characterised by high levels of industrialization, competitiveness and quality livelihood. To achieve this goal, five critical development approaches were identified.

- Large investments in energy and transport infrastructure;
- Strategic investments in key industries, including agriculture, special economic zones, natural resource industries;
- Enhancing skills development;
- Significant improvement in business environment;
- Institutional reforms for effective implementation and monitoring/evaluation mechanism.

2.2.2 MUKUKUTA 2

In Vision 2025, the overarching goal of the development agenda is reduction of poverty. The strategy to achieve this goal is described in the MUKUKUTA. The MUKUUTA 2 is a 5-year phasing program covering 2010/11 to 2015/16. The MUKUKUTA 2 sets operational targets in various sectors including transport sector. The following table summarizes the operational targets of the road sub-sector and actual performance made by TANROADS between 2012 and 2014.

Table 2.2.1 Road Sector in MUKUKUTA 2

Operational Targets	Performance in 2012 to 2014
National (trunk and regional) roads. Different part of the country linked and connected to regional and global networks by efficient transport system	
1,000 km of paved roads rehabilitated	636 km of trunk roads rehabilitated to bitumen standards
15,000 km of national unpaved roads rehabilitated/graveled (3,000 km of trunk road and 12,000 km of regional roads)	456 km of trunk roads rehabilitated to gravel standards. 3,114 km of regional roads rehabilitated to gravel standards
30,000 km per year of national roads maintained	30,380 km in 2011/12 32,282 km in 2012/13 30,320 km in 2013/14
10 major bridges constructed	3 bridges completed. Other 7 in various stage of implementation.

Source: TANROADs

2.2.3 National Transport Policy

The National Transport Policy describes how the transport sector contributes to national goals and objectives set by the Vision 2025 and MUKUKUTA 2 and how it facilitates the development of national economy. The vision of the National Transport Policy is to have efficient and cost-effective domestic and international transport service to all segments of the population and sectors of the national economy. The goals of the national transport policy are focused on the following themes:

- Paving all trunk rods linking regional capitals to bitumen standards by 2018,

- Improving urban mobility and reducing congestion,
- Delivering safe and environmentally sustainable transport infrastructure and services,
- Providing efficient and effective transport corridors for international trade using national gateways.

The National Transport Policy identifies following four corridors as priority development corridors and Arusha – Himo section forms part of Tanga Development Corridor.

i) Mtwara Development Corridor (linking Mtwara with Northern Malawi, Mozambique and North Eastern Zambia);

ii) Dar es Salaam Development Corridor (or the TAZARA or Uhuru Corridor connecting Dar es Salaam Port with the Southern and Eastern Highland and Northern Zambia;

iii) Central Development Corridor (originating from the port of Dar es Salaam and taking the central line route extending to Eastern DRC and Burundi via Kigoma, and to Rwanda via Isaka Dry Port and to Uganda via Mwanza; and

iv) Tanga Development Corridor (connecting the Tanga Port with the Lake Victoria Region and Uganda) via Arusha.

2.2.4 Big Result Now

In 2013, the Big Results Now program was launched as one of the key steps in driving the Vision 2025. The Big Results Now program focuses on targeted priority projects that will have a large multiplier effects and enhance the achievement of Vision 2025. The Big Results Now program identifies four priority sectors, that includes (i) transport and logistics, (ii) energy, (iii) agriculture and (iv) real estates and hence promotes private and public funding for infrastructure development in these sectors.

Arusha – Holili project (described as Construction of Arusha –Moshi –Himo Junction Dual Carriageway (105km) in the Big Result Now) is one among the four priority projects in the road sub-sector[1].

Table 2.2.2 Priority Projects in Road Sub-sector in Big Result Now

Project Name	Implementer	Estimated Value (USD)
Construction of Dar es Salaam-Chalinze Express Way	TANROADS	$ 519 M
Construction of Arusha –Moshi –Himo Junction Dual Carriageway (105km)	TANROADS	$ 363 M
Upgrading Uvinza (Kigoma)- Ilunde- Malagarasi-Kaliua– Urambo(188km)	TANROADS	$ 150 M
Upgrading to Bitumen Standard for the Manyoni-Itigi-Tabora Road (254km)	TANROADS	$ 57 M

Source: Project Funding Opportunities (GOT)

[1] The Arusha- Holili project is described as follows in the Big Result Now. The road links two tourist and economically important towns of Arusha and Moshi, links the two towns to tourist attractions such as the Serengeti, Ngorongoro, Arusha and Kilimanjaro National Parks, links the rest of Tanzania to a neighboring country of Kenya, links the two towns to Dar es Salaam port and Tanga port. The existing road is a two lane road. The study in 2011 by ICT of India also recommended that this road is economically and financially viable for construction to a dual carriageway under PPP arrangement. The traffic volume based on year 2010 on this road ranges between 5,000 -7,500 AADT.

Table 2.2.3 Priority Projects in Rail Sub-sector in Big Result Now

Project Name	Implementer	Estimated Value (USD)
Construction of standard gauge railway line Dar es Salaam-Kigoma(1263km) on reserved land	RAHCO	$ 3.78 B
Construction of standard gauge railway line from Taborato Mwanza on reserved land	RAHCO	$ 1.14 B
Construction of standard gauge railway line from Uvinza–Musongati	RAHCO	$ 600 M
Procurement of 63 high horsepower locomotives	TRL	$ 206 M
Procurement of 2,234 goods wagons	TRL	$ 171 M

Source: Project Funding Opportunities (GOT)

Table 2.2.4 Priority Projects in Port Sub-sector in Big Result Now

Project Name	Implementer	Estimated Value (USD)
Development of Kisarawe Cargo Freight Station	Tanzania Ports Authority	$ 120 M
Upgrading of Dar es Salaam Port Berths 1-7, Construction of Roll-on/Roll-off ("RoRo") Berth, Dredging and widening of Port Entrance Channel and Improvement of Bulk Cargo Handling System including Port Layout	Tanzania Ports Authority	$ 500 M
Expansion and Modernization of Kigoma Port	Tanzania Ports Authority	$ 300 M
Modernization of Mwanza South Port	Tanzania Ports Authority	$ 400 M
Development of New Container Terminal Berths 13-14	Tanzania Ports Authority	$ 400 M

Source: Project Funding Opportunities (GOT)

CHAPTER 3 REVIEW OF PREVIOUS F/S

3.1 Summary Result of Preliminary Review Work on Previous F/S

Prior to implementation of the Study, the Study Team reviewed available reports and materials, including that of the previous F/S report and identified several critical issues, particularly those related to the traffic demand study, preliminary design and project evaluation, in order to fulfil the requirements set by the JICA's manuals/guidelines and to have the Project achieve JICA's loan support. Accordingly, the Study Team suggests the following approaches to ensure the quality of the study result meeting the JICA's loan requirements.

3.1.1 Traffic Survey and Demand Forecast

The Study Team, through the preliminary review of previous F/S report, identified technical issues in traffic survey and demand forecast, explored in the F/S report. For instance, in previous F/S, improvement of several roundabouts along the project road was proposed. However, traffic survey to validate the proposed design of those roundabouts was not carried out.

Regarding traffic demand forecast, the Study Team identified several deficiencies, while reviewing the F/S report. For instance, in the previous F/S report, a single future traffic demand growth rate was applied to the entire study area and encompassed all types of vehicles. It was assumed that the growth rate of heavy vehicles such as trucks and trailers in Arusha and Kilimanjaro would be different because of disparity in population density and economic activities. Therefore, this assumption in the accuracy of future demand forecast in previous F/S report in middle and long term may not be reliable.

Diversion traffic induced by the proposed road improvement project from Dar es Salaam to Mombasa Seaports was estimated through road accessibility to those two seaports. Dwell time and other factors that influence route choice of sea-borne cargo were not considered. Furthermore, induced traffic demand generated by existing large-scale development plan along the project road was not accounted for in demand forecast.

To address the aforementioned deficiencies, the Study Team estimates traffic demand to achieve sufficient accuracy of future traffic demand for the preliminary design and project evaluation, which necessitates supplementary traffic surveys and carry out demand forecast by traffic zone and type of vehicle based on traffic survey and existing traffic data.

3.1.2 Road Design

The Study Team reviewed the road design of the previous F/S and identified the following pending issues.

(1) Setting of the design speed is unsubstantiated; the design speed changes from 100 km/h to 50 km/h frequently within a short section, which deviates from the actual traveling speed and is

therefore not desirable in terms of securing safety.

(2) There are multiple sections whose (vertical) grades do not comply with the design standard of Tanzania and that have steep grades without climbing lanes, which may cause accidents due to speed drop and when overtaking/bypassing heavy vehicles.

(3) For the vertical alignment near new Kikafu Bridge the grades of the approach section are proposed for in the range of 3.5% to 4.5% to limit the bridge length to about 100 meters. Considering the past major accidents, it is recommended to set flatter grades at/near proposed new Kikafu Bridge and to take appropriate measures to cope with design-related issues.

(1) Design Standard

The Study Team conducts design review on the basis of the Tanzanian road design standards, “Road Geometric Design Manual (2011)” and “Pavement and Material Design” while referring collaterally to the Japanese standards and AASHTO and other foreign standards.

(2) Width Composition

The Study Team reviews the width composition optimum for each section and proposes based on the width composition planned in F/S while referring to the land use along the route, demand for parking/stopping on the roadside and demand of short-trip traffic.

(3) Horizontal Alignment

The basis on which the speed of each section is established is clarified because the setting of the design speed governs the geometric design standard including the horizontal alignment. The preliminary review work on the F/S found that the horizontal alignment of the target section is gentle enough to ensure a satisfactory traveling performance in most sections.

(4) Vertical Alignment

83% of the target sections are expected to undertake widening of the existing roads, with the vertical alignment following the grades and vertical curve of the existing roads. However, the existing roads are not designed for the design speed of 100 km/h and certain sections include those with steep grades not appropriate for the set design speed and even those without sufficient sight distance.

For roads concerned, the ratio of large vehicles is rather high at about 20%. Speed decrease of large vehicles in the sections with steep grades may not only hinders smooth traffic flow but may also lead to overtaking/passing possibly resulting in collision with an oncoming vehicle. On the basis of the traveling performance and drivers’ characteristics of large vehicles running on the road concerned as well as the cargo loading state by direction, review is made for each section in terms of the feasibility of improving the vertical alignment to the flatter grades and providing the climbing lane.

(5) Intersection

Major intersections where the design proposed by F/S and D/D are reviewed. For the crossing method at each intersection, the at-grade intersection and roundabout are compared based on the traffic demand by direction and the design speed of the section concerned.

(6) Pavement Design

The pavement design of Tanzania specifies the pavement composition while using the chart based on the cumulative axle load and ground condition. Application of the design approach of AASHTO, etc. is reviewed as required while taking into account the application limit of cumulative axle load (50×10^6) of the standard.

3.1.3 Bridge Design

The Study Team reviewed the bridge design presented in previous F/S report and identified the following technical issues related to the bridge design.

(1) For applicable standards and design conditions, the Study Team confirms the approach with the authorities concerned because of revisions, made to the standards in 2011.

(2) The bridge type and structural data were uniformly established according to the bridge size, and the priority among them and the rationale for decision of the bridge type are unclear. Accordingly, the Study Team performs the technical verification (selection of the bridge type, etc.) to prove relevance to meet the requirements set by JICA's guidelines and manuals.

(3) Issues and subjects in terms of the design and construction plans are identified; for example, the reference data necessary for appraisal of the JICA loan project is insufficient, including those for the project process, review of the construction method for estimation of the work costs and for project schedules. The Study Team undertakes a supplementary study and surveys as follows to deal with these technical issues.

(1) Design Standard and Design Conditions

In Tanzania, the design standard was revised in 2011. The design standard and conditions is established after confirmation with the authority concerned based on review of the design standard in F/S.

(2) Verification of Technical Adequacy

The technical review, particularly that on the new Kikafu Bridge, is done and alternative bridge plans should be proposed in consideration with site conditions (topography, geology, rivers, construction, peripheral environment, etc.).

Table 3.1.1 List of Bridge Proposed in F/S

No.	Bridge position	Crossing river	Bridge type	Bridge length	Angle
1	0+200	Magdarisho River	Reinforced-concrete simple bridge (integral bridge)	10.6m	20.9
2	0+936	Usa River	Pre-stressed concrete simple girder bridge (both end fixed)	25.0m	Right angle
3	4+783	Maji Ya Chai River	Reinforced-concrete simple bridge (integral bridge)	10.6m	Right angle
4	30+988	Sanya River	Reinforced-concrete simple bridge (integral bridge)	20.6m	Right angle
5	42+613	Kikafu River	Steel box girder bridge	100.0m	Right angle
6	47+359	Weruweru River	3-pan continuous reinforced-concrete bridge	50.0m	Right angle
7	54+243	Karanga River	Reinforced-concrete simple box girder bridge (pedestrian bridge)	65.0m	Right angle
8	58+920	Rua River	Reinforced-concrete simple bridge (integral bridge)	20.6m	Right angle
9	65+012	Kisanja River	Reinforced-concrete simple bridge (integral bridge)	10.6m	Right angle
10	78+175	Mabungo River	Reinforced-concrete simple bridge (integral bridge)	17.2m	25.4
11	84+636	Kilacha River	Reinforced-concrete simple bridge (integral bridge)	10.6m	Right angle
12	84+936	Sangana River	Reinforced-concrete simple bridge (integral bridge)	12.8m	33.8
13	86+100	Kimisho River	3-span continuous reinforced-concrete bridge	41.8m	Right angle

Source: JICA Study Team

(3) Construction Method and Project Schedules

The construction plan and the project schedules is proposed, which are necessary for estimation of the project costs and preparation of the project process. Specifically for the new Kikafu Bridge exceeding 100 m in length an adequate bridge construction plan is reviewed by taking into account locally-available equipment and materials (temporary materials, such as cantilever noses, movable forms, etc.) due to the complexity of the construction work.

3.1.4 Project Evaluation

The Study Team identified the following technical issues in the project evaluation methodology discovered in the previous F/S report.

(1) The Vehicle Operating Cost (VOC) saving and travel time reduction due to the implementation of project were calculated and indicated as quantitative effect derived from the Project. However, the accuracy of future demand forecast in previous F/S report in middle and long term may not be reliable as described above.

(2) In the previous F/S report, qualitative effects derived from the Project are not estimated, apart from its input in economic analysis on the Project. In the Study, the Study Team carries out economic analysis and project evaluation, following JICA's guideline.

CHAPTER 4 TRAFFIC DEMAND FORECAST

4.1 Traffic Survey

4.1.1 Outline of Traffic Survey

To update existing traffic volume data at road section and junction, traffic count survey was performed in October 2015. Traffic survey consists of; (i) traffic count survey at road sections, and (ii) traffic count survey at junctions and roundabouts.

The objectives of traffic count survey at road sections are to obtain current volume of traffic demand to consider necessity of expansion to four lanes and for the calibration of estimated traffic demand in 2015 by origin-destination and vehicle type. Thus, traffic count at road sections was carried out at five sites as shown in following table. Basically, traffic count survey was performed for 12 hours (6:00 – 18:00) except Section B near Momera Road Junction. At Section B, traffic survey was carried out for 24 hours to prepare expansion factor for 12 hours traffic volume to 24 hours traffic volume.

Table 4.1.1 Road Section Traffic Count Survey Location

No	Name	Survey period	Survey day
A	Makumira University College	6:00 – 18:00 (12h)	15th October 2015
B	Near Momera Rd. Junction	6:00 – 6:00 (24h)	15th October 2015
C	Between Usa River and Kikatiti	6:00 – 18:00 (12h)	15th October 2015
D	Near Kingori-Leguruki Rd. Jct.	6:00 – 18:00 (12h)	15th October 2015
E	Near Kikafu Bridge	6:00 – 18:00 (12h)	15th October 2015

Source: JICA Study Team

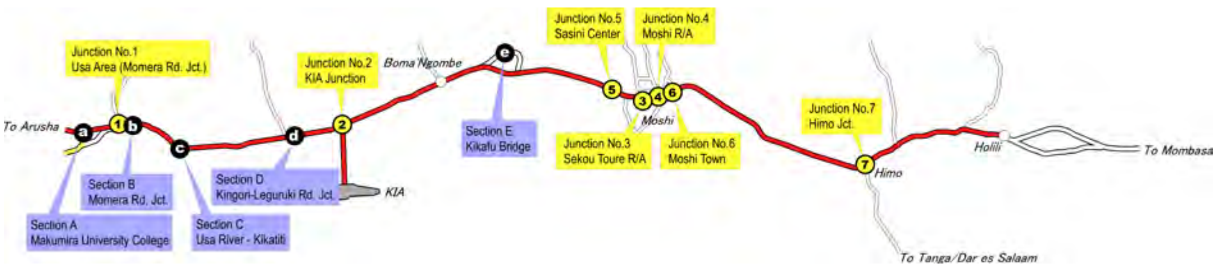
Traffic count survey at junctions and roundabouts is carried out to consider adequacy of existing junction/roundabout improvement plan proposed by F/S against future traffic demand. Thus, traffic count survey at junction is carried out at following junctions/roundabouts.

Table 4.1.2 Junction Traffic Count Survey Location

No	Junction	Type of junction	Survey period	Survey day
1	Usa Area (Momera Rd. Junction)	3 legs junction	6:00 – 18:00 (12h)	15th October 2015
2	KIA Junction	4 legs junction	6:00 – 18:00 (12h)	13th October 2015
3	Sekou Toure Way Roundabout	5 legs roundabout	6:00 – 18:00 (12h)	13th October 2015

No	Junction	Type of junction	Survey period	Survey day
4	Moshi Roundabout	4 legs roundabout	6:00 – 18:00 (12h)	12th October 2015
5	Sasini Center (Safari Farm Road Junction)	4 legs junction	6:00 – 18:00 (12h)	15th October 2015
6	Moshi Town (near Anglican Church Majengo)	3 legs x 2 junctions	6:00 – 18:00 (12h)	12th October 2015
7	Himo Junction	3 legs junction	6:00 – 18:00 (12h)	12th October 2015

Source: JICA Study Team



Source: JICA Study Team

Figure 4.1.1 Traffic Survey Location

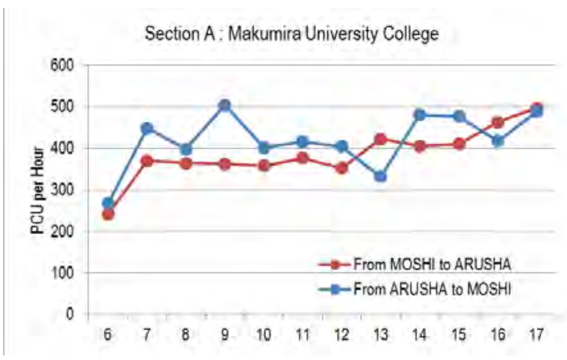
Vehicle classification of traffic count survey was of the following seven categories.

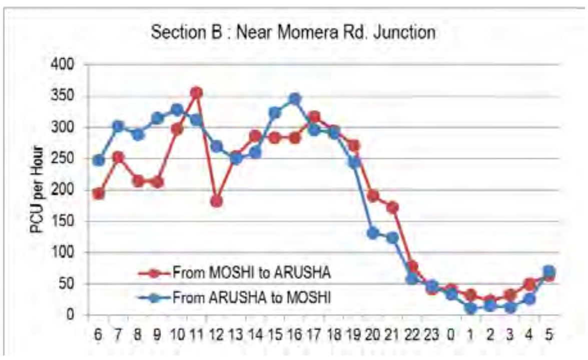
- Motor cycle, Bajaj
- Passenger cars (including 4WD, pickup-truck),
- Daladala, Mini Bus,
- Large Bus,
- 2 Axles Rigid Truck,
- 3 and more Axles Rigid Truck, and
- Semi / Full Trailers.

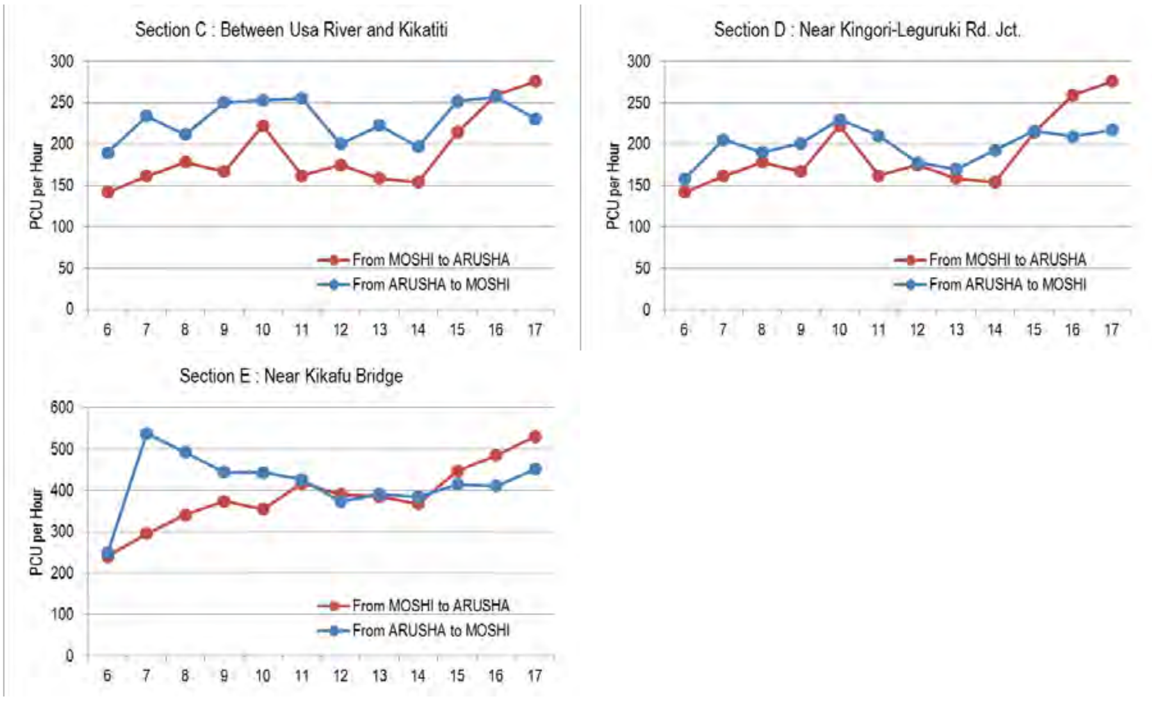
4.1.2 Summary Results of Traffic Survey

(1) Road Sections

The following figures show a summary of traffic count survey at surveyed road sections.







Source: JICA Study Team

Figure 4.1.2 Hourly Fluctuation and Traffic Volume

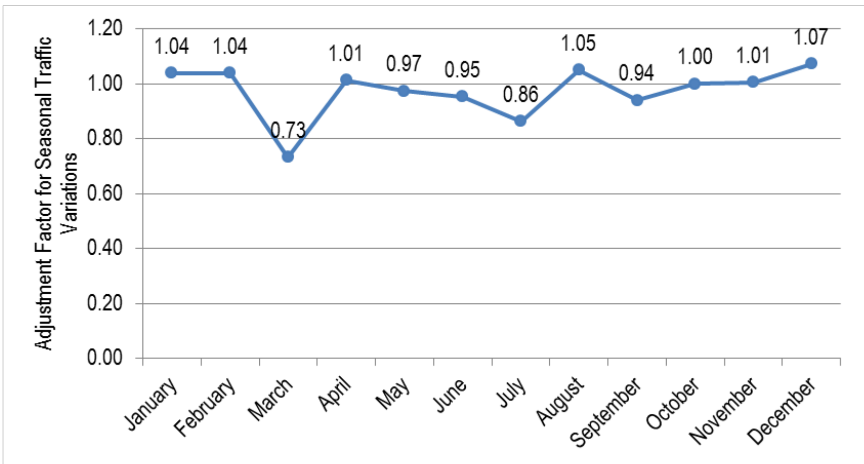
Based on the traffic count survey at Section B, expansion factor to expand 12 hours traffic volume to 24 hours are calculated as shown in the following table.

Table 4.1.3 Traffic Volume at Section B

	Motor-cycle	Passenger Car	Small Bus	Large Bus	2 Axles Truck	3 Axles Truck	Trailers
Traffic Volume (veh./12 hours 6:00-18:00)	775	2,608	1,550	209	315	185	238
Traffic Volume (veh./12 hours 18:00-6:00)	277	1,067	356	63	138	66	118
Expansion Factor for 6:00 – 18:00 to 24 hours	1.36	1.41	1.23	1.30	1.44	1.36	1.50

Source: JICA Study Team

Traffic volume is fluctuated by season. According to the F/S, adjustment factor of October to annual average daily traffic (AADT) is 1.00.



Source: Traffic Survey and Analysis Report of F/S in 2011

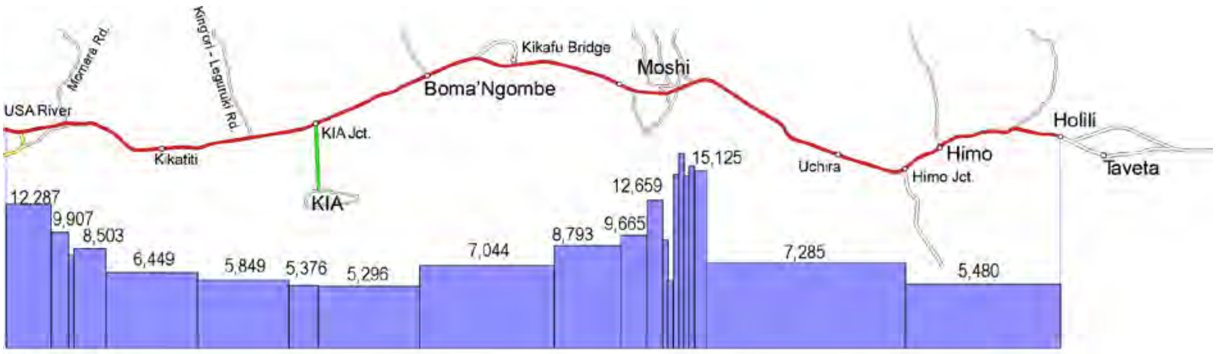
Figure 4.1.3 Adjustment Factors for Seasonal Traffic Variations

The table and figure below show AADT in 2015 along Arusha-Holili Road based on the traffic survey including inflow traffic volume at junctions/roundabouts.

Table 4.1.4 Average Daily Traffic Volume by Section along Arusha - Holili Road

	Motor-cycle	Passenger Cars	Small Buses	Large Bus	2 Axles Truck	3+ Axles Truck	Trailer	Total (PCU/ day)	Total (PCU/ day exc. MC)
Section A	707	5,408	4,164	460	1,025	401	829	12,994	12,287
Usa River (West of Momera Jct.)	851	4,551	2,855	400	915	462	724	10,758	9,907
Usa River (East of Momera Jct.)	664	3,751	2,591	402	999	481	727	9,615	8,951
Section B	526	3,675	2,478	435	680	452	783	9,029	8,503
Section C	404	2,734	1,453	435	742	430	655	6,853	6,449
Section D	227	2,607	1,124	469	537	415	698	6,077	5,849
KIA Junction (West of KIA)	253	2,286	1,181	419	608	244	638	5,630	5,376
KIA Junction (East of KIA)	150	2,559	966	373	554	222	622	5,446	5,296
Section E	329	3,254	1,591	462	688	365	684	7,373	7,044
Sasini Center (West of Jct)	508	3,948	2,216	552	822	508	747	9,301	8,793
Sasini Center (East of Jct)	735	4,458	2,497	554	884	528	744	10,400	9,665
Sekou Toure Way R/A (West)	1,885	6,515	4,621	404	465	187	467	14,544	12,659
Sekou Toure Way R/A (East)	1,696	5,684	2,439	135	331	176	464	10,924	9,229
Moshi R/A (West of R/A)	1,319	4,248	546	39	399	264	349	7,163	5,845
Moshi R/A (East of R/A)	2,122	6,958	6,519	367	397	270	349	16,983	14,860
Moshi Town West (West)	2,825	9,128	5,045	441	977	579	477	19,473	16,648
Moshi Town West (East)	2,580	7,787	5,115	408	662	405	362	17,320	14,740
Moshi Town East (West)	2,430	8,318	4,893	452	1,199	234	487	18,014	15,584
Moshi Town East (East)	2,138	7,909	4,852	452	1,195	230	487	17,263	15,125
Himo Jct (West)	502	2,193	2,497	454	1,439	232	471	7,786	7,285
Himo Jct (East)	883	1,973	2,387	48	820	115	138	6,363	5,480

Source: JICA Study Team

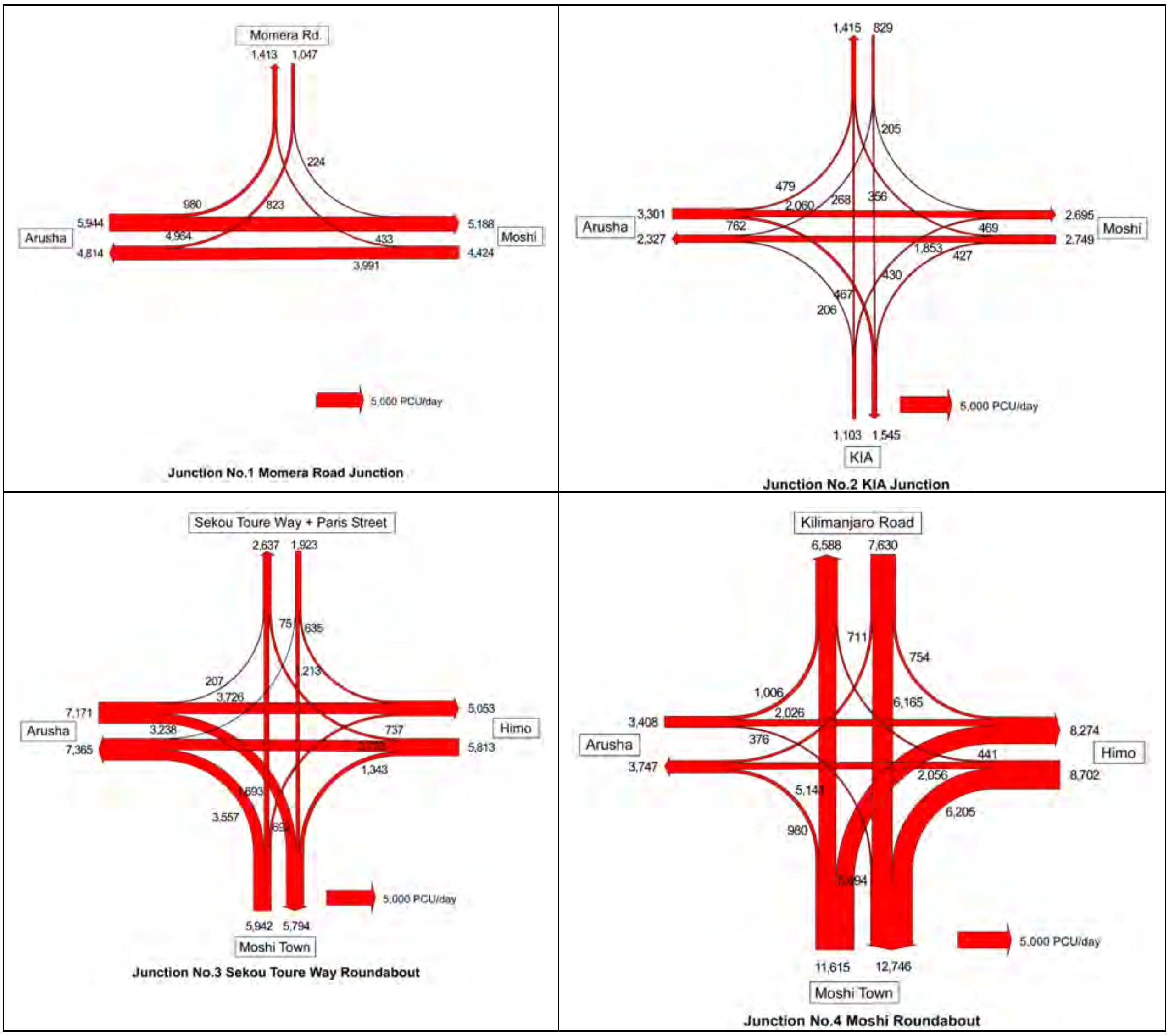


unit: PCU/day
Source: JICA Study Team
Note: motorcycles are not included.

Figure 4.1.4 AADT in 2015 Based on Traffic Survey (PCU/day)

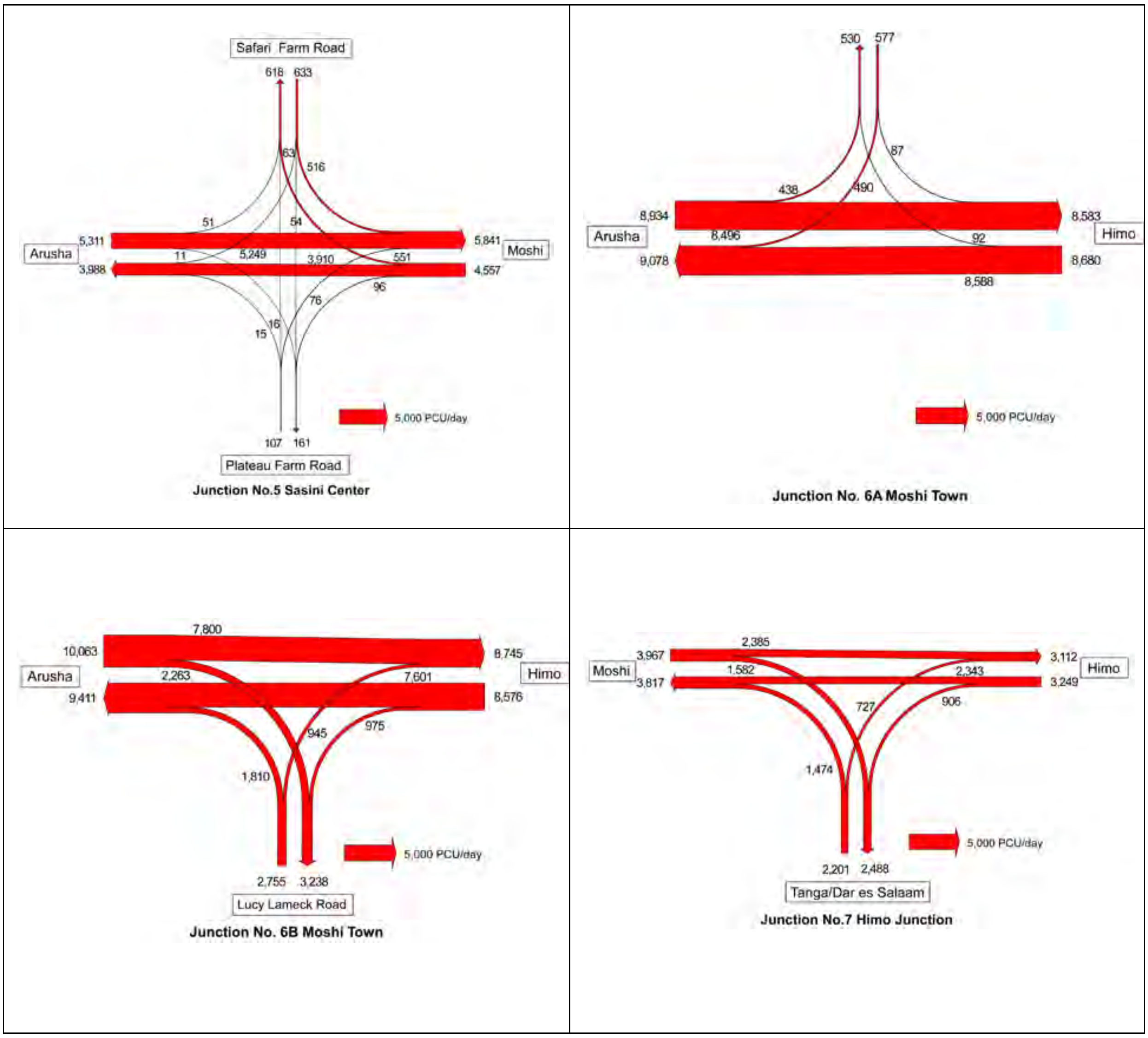
(2) Junctions and Roundabouts

The following figures show the traffic volume at junctions/roundabouts in 2015 expanded to daily traffic volume.



Source: JICA Study Team

Figure 4.1.5 AADT in 2015 at Junctions/Roundabouts



Source: JICA Study Team

Figure 4.1.6 AADT in 2015 at Junctions/Roundabouts (cont'd)

The Highway Capacity Manual (HCM) indicates the methodology for calculation of capacity of non-signalized two-way stop-controlled intersection and roundabout based on gap acceptance model.

Potential capacity of each minor traffic stream; which is traffic flow from minor road, is computed by the formula below.

$$C_{p,x} = V_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3600}}{1 - e^{-v_{c,x} t_{f,x}/3600}}$$

Where,

$C_{p,x}$: potential capacity of minor movement x (vehicle /hour),
$V_{c,x}$: conflicting flow rate for movement x (vehicle/hour),
$t_{c,x}$: critical gap (the minimum time that allows junction entry for one minor stream vehicle) for minor movement x, and
$t_{f,x}$: follow-up time (the time between the departure of one vehicle from the minor street

and the departure of the next under a continuous queue condition) for minor movement.

Table 4.1.5 Base Critical Gaps and Follow-up Time for two-way stop-controlled intersections

Vehicle Movement	Base Criterial Gap (sec)		Base Follow-up Time (sec)
	Two-lane Major Street	Four-lane Major Street	
Right turn from major road	4.1	4.1	2.2
Left turn from minor road	6.2	6.9	3.3
Through traffic on minor road	6.5	6.5	4.0
Right turn from minor road	7.1	7.5	3.5

Source: HCM 2000

Note: Movement is converted to left-hand traffic in Tanzania.

Where several movements share the same lane and cannot stop side-by-side at the stop line, shared-lane capacity is computed by the following formula.

$$C_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}} \right)}$$

Where,

C_{SH}: capacity of the shared lane (vehicle /hour),

v_y : flow rate of the y movement in the subject shared lane (vehicle /hour), and

$C_{m,y}$: movement capacity of the y movement in the subject shared lane (vehicle /hour).

On the other hand, capacity of approach of roundabout is computed by following formula.

$$C_a = V_c \frac{e^{-v_c t_c / 3600}}{1 - e^{-v_c t_f / 3600}}$$

Where,

C_a: approach capacity (vehicle /hour),

V_c: conflicting circulating traffic (vehicle/hour),

t_c: critical gap (sec), and

t_f: follow-up time (sec).

Table 4.1.6 Critical Gap and Follow-up Time for Roundabouts

	Critical Gap (sec)	Follow-up time (sec)
Upper bound	4.1	2.6
Lower bound	4.6	3.1

Source: HCM 2000

Based on the methodology for the calculation of potential capacity for un-signalized junctions and roundabouts by HCM, current volume capacity ratio of each approach to junctions in peak hour is calculated as shown in the following table.

Currently, Moshi R/A is almost saturated in evening peak, because of heavy traffic volume between Himo (East) and Moshi Town (South), Kilimanjaro Road (North) and Moshi Town. Other junctions and R/As are less saturated.

Table 4.1.7 Inflow Traffic Volume and Capacity in Peak Hour

No.	Junction Name And Peak Hour	Approach from	Inflow traffic in peak hour (PCU/hour)	Potential Capacity (PCU)	Volume/Capacity
1	Momera Rd. Jct. 16:00-17:00	North	102	307	0.33
		Moshi	344	1,977	0.17
		Arusha	452	1,964	0.23
2	KIA Jct 17:00-18:00	North	79	559	0.14
		Moshi	250	1,106	0.23
		South	80	478	0.17
		Arusha	242	1,223	0.20
3	Sekou Toure Way R/A 17:00-18:00	Sekou Toure Way	90	871	0.10
		Paris St.	2	824	0.00
		Himo	482	1,063	0.45
		Moshi Town	561	1,036	0.54
		Arusha	521	1,106	0.47
4	Moshi R/A 17:00-18:00	North	469	807	0.58
		Himo	659	975	0.68
		South	1,143	1,138	1.00
		Arusha	228	559	0.41
5	Sasini Centre 16:00-17:00	North	33	533	0.06
		Himo	334	856	0.39
		South	1	247	0.00
		Arusha	481	1,233	0.39
6A	Moshi Town 17:00-18:00	North	34	122	0.28
		Himo	580	1,993	0.29
		Arusha	779	1,990	0.39
6B	Moshi Town 7:00-8:00	Himo	820	1,885	0.44
		South	185	471	0.39
		Arusha	664	1,872	0.35
7	Himo Jct. 15:00-16:00	Himo	238	1,929	0.12
		South	181	521	0.35
		Arusha	280	1,884	0.15

Source: JICA Study Team

4.2 Traffic Demand Forecast

4.2.1 Methodology of Traffic Demand Forecast

Traffic demand for the Arusha-Holili Road is forecasted by traffic assignment of current and forecasted future vehicular trip by origin and destination (OD) assigned on the road network. Current vehicular OD is estimated based on existing traffic survey conducted by relevant empirical studies and traffic information provided by TANROADS and traffic survey performed in October 2015 by the Study. Future OD is forecasted by GDP/GRDP growth rate by traffic zone and elasticity of traffic volume to GDP/GRDP growth.

Existing traffic survey results and information applied to demand forecast in the Study is summarized in the following table.

Table 4.2.1 Existing Traffic Survey Results and Information applied to the Study

Study	Year of Traffic Survey	Data and Information	Remarks
Arusha – Holili / Taveta - Voi Feasibility Study (2012, AfDB)	2010	AADT by survey station Current origin – destination table	11 vehicle classes incl. motorcycle. Combined into single OD table excl. motorcycle.
Comprehensive Transport and Trade System Development Master Plan in the United Republic of Tanzania (2014, JICA)	2011	Traffic volume and road side OD interview of cargo trucks at relevant 6 stations (Namanga, Holili, Usa River, Hidaru, Dar es Salaam Port)	Cargo trucks (6 classes) only.
Axle load data at weigh stations (TANROADS)	2014	Axles load by type axles and OD at weigh station in Arusha (Longido, Makuyuni), Kilimanjaro (Nija panda)	Axles load data of Arusha exclude non-overload trucks. Data of Kilimanjaro include empty trucks and heavy bus.

Source: JICA Study Team

The methodology and approaches of traffic demand forecast in the Study is summarized below.

(i) Existing vehicular OD table prepared by F/S associates with the following technical issues; (i) traffic analysis zone system does not follow the administration boundary, (ii) single aggregated OD table which includes passenger cars, buses and trucks is prepared and single growth rate is applied to estimate the future OD table. Therefore, existing OD table is developed by six vehicle classes (passenger cars, small bus, large bus, light truck, medium truck and trailers) and recombined based on the traffic analysis zone system (30 zones including neighbouring countries) based on the current administration boundary.

(ii) In the F/S, elasticity of traffic demand is defined as 1.01 (- 2019), 0.92 (2020 - 2029) and 0.73 (2030 -) without consideration of the vehicle type. Therefore, modified OD table (six vehicle type, 30 zone system) in 2010 is updated to 2014 by traffic growth rate by zone. Traffic growth rate is estimated by periodical traffic volume growth provided by TANROADS and GDP growth of Tanzania as shown in the following table.

Table 4.2.2 Traffic Demand Elasticity to GDP/GRDP Growth

	Passenger Cars	Buses	Trucks/Trailers
Elasticity of Traffic Demand to GDP/GRDP	1.493	1.042	0.903

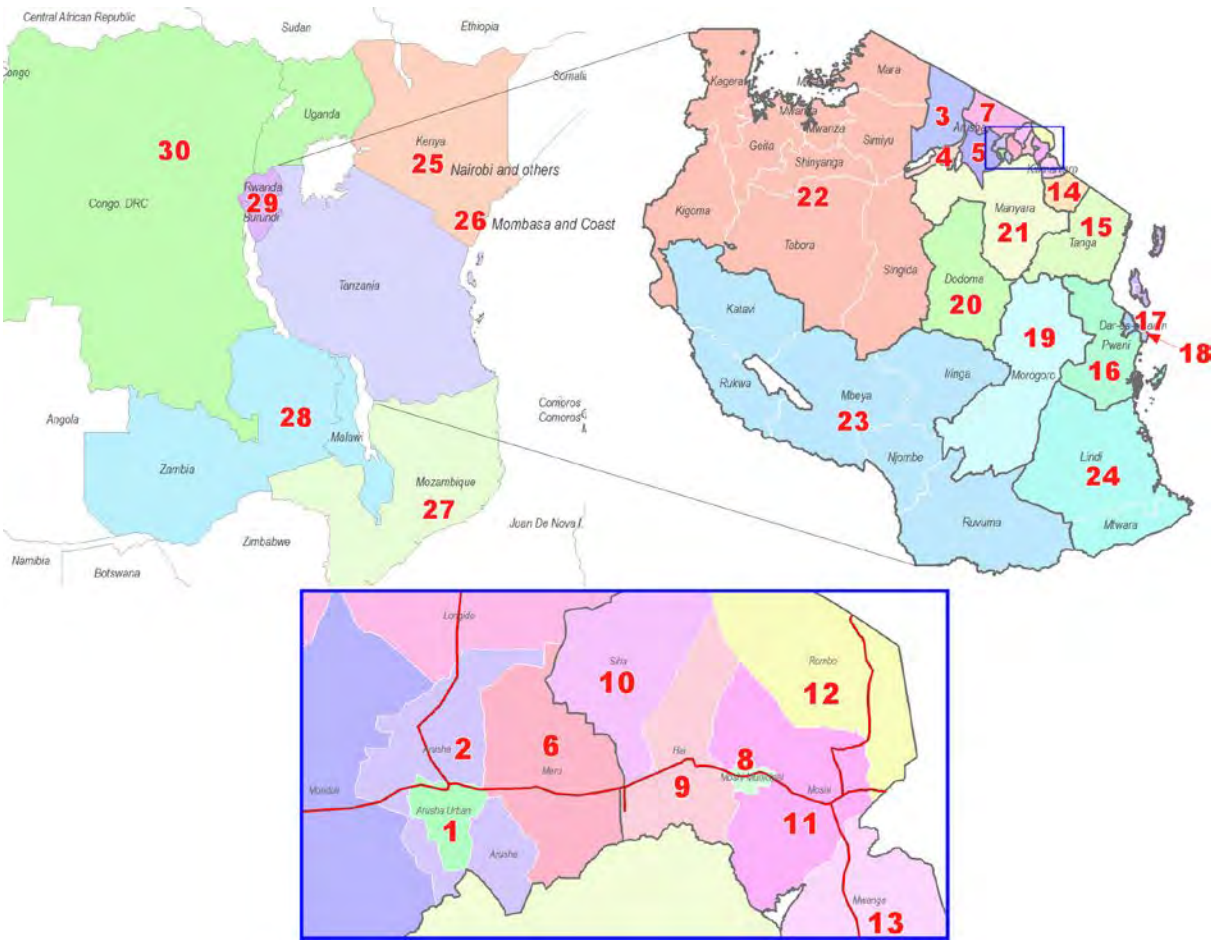
Source: JICA Study Team

Table 4.2.3 Zone System for Arusha – Holili Road Study

Zone No.	Region, Country	Zone Name
1	Arusha	Arusha Urban
2	Arusha	Ngorongoro
3	Arusha	Karatu
4	Arusha	Monduli
5	Arusha	Meru
6	Arusha	Arusha
7	Arusha	Longido
8	Kilimanjaro	Moshi Municipal
9	Kilimanjaro	Hai
10	Kilimanjaro	Siha
11	Kilimanjaro	Moshi
12	Kilimanjaro	Rombo
13	Kilimanjaro	Mwanga
14	Kilimanjaro	Same
15	Tanga	Tanga

Zone No.	Region, Country	Zone Name
16	Pwani	Pwani
17	Dar es Salaam	Dar es Salaam
18	Dar es Salaam	Dar es Salaam Port
19	Morogoro	Morogoro
20	Dodoma	Dodoma
21	Manyara	Manyara
22	Mara, Simiyu, Singida, Tabora, Geita, Shinyanga, Mwanza, Kagera,Kigoma	Tanzania West
23	Katavi, Rukwa, Mbeya, Iringa, Ruvuma, Njombe	Tanzania South
24	Lindi, Mtwara	Mtwara
25	Kenya	Nairobi
26	Kenya	Mombasa
27	Mozambique	Mozambique
28	Malawi, Zambia	Malawi, Zambia
29	Rwanda, Brundi	Rwanda, Brundi
30	Uganda, DRC	Uganda, DRC

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.1 Zone System for Arusha – Holili Road Study

(iii) Based on the results of roadside truck OD interview survey in 2011 carried out by JICA Master Plan (MP)[1], truck OD tables (light truck, heavy truck, and trailers) are re-built in accordance with 30 zone system and updated to 2014 by the elasticity of traffic demand.

(iv) Two projected OD tables in 2014, namely, F/S-based OD consists of six vehicle classes and MP-based OD consists of three types of trucks are integrated to single OD table. Vehicular trips by OD of passenger cars and buses are F/S-based, truck trip is average of F/S-based OD and MP-based OD.

(v) Based on the axles load data provided by TANROADS, truck OD through weigh stations in 2014 is estimated. Truck trip through weigh stations in Arusha and Kilimanjaro in F/S and MP-based OD is replaced by weigh station based truck OD.

(vi) Modified OD in 2014 is updated to 2015 by the elasticity of traffic demand and estimated GDP/GRDP growth, and current 2015 vehicular OD is estimated through calibration with the results of traffic survey carried out in October 2015.

(vii) Future traffic demand, namely, zonal vehicle trip generation in 2025 and 2035 are estimated by forecasted GDP/GRDP growth rate[2] by zone and elasticity of traffic demand. Vehicle OD in 2025 and 2035 is estimated by a present pattern method using the current OD in 2015 and future traffic generation in 2025 and 2035.

[1] JICA (2014) Comprehensive Transport and Trade System Development Master Plan in the United Republic of Tanzania

[2] GDP growth rate in Tanzania is projected at 6.2% (2015) 5.6% (2025), 5.2% (2035) and 4.9% (2045) per annum, based on a cross country study in the selected countries.

(viii) Current and forecasted future traffic demand is assigned on the road network and computed growth rate of traffic demand by road link. Future traffic demand by road link is calculated by the results of traffic survey in 2015 and traffic growth based on the traffic assignment.

4.2.2 Results of Demand Forecast

The result of traffic demand forecast along the project road is tabulated by the road network link with the target year of 2025 and 2035. The traffic volume along the project road is projected to increase at around 6% to 7% per annum between 2015 and 2035. A large number of traffic is projected at/near Usa River and Moshi Town where the projected traffic demand exceeds 20,000 PCU/day in 2025 and 40,000 PCU/day in 2035[3].

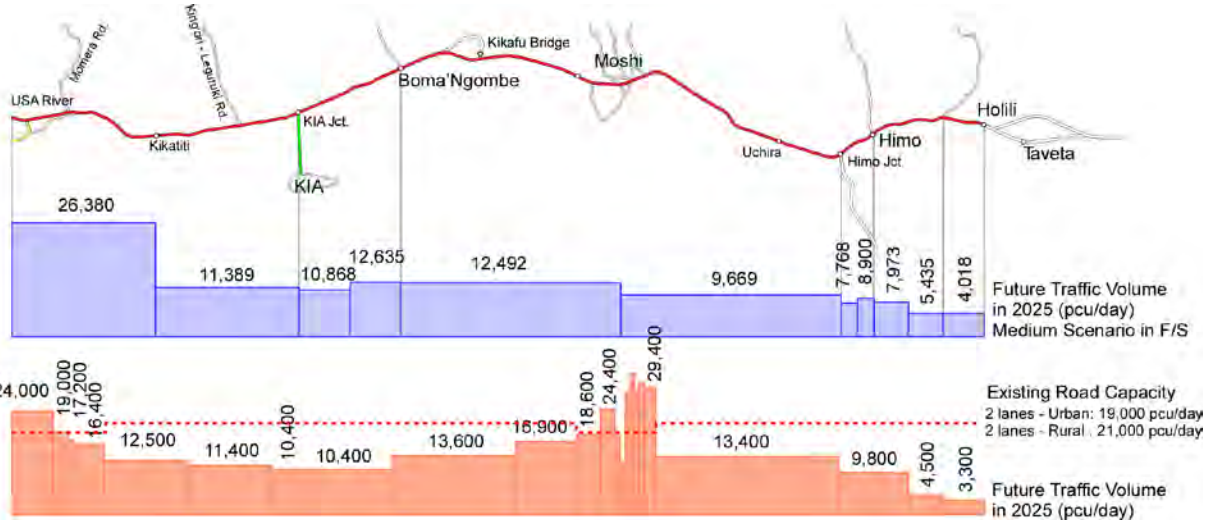
Table 4.2.4 Future Traffic Demand by Section (both direction, PCU/day)

Survey Station	2015	Est. 2025	2015 – 2025 Growth Rate (p.a.)	Est. 2035	2025 – 2035 Growth Rate (p.a.)
Section A	12,287	24,000	6.9%	45,800	6.7%
Usa River (West of Momera Jct.)	9,907	19,200	6.8%	36,400	6.6%
Usa River (East of Momera Jct.)	8,951	17,200	6.7%	32,200	6.5%
Section B	8,503	16,400	6.8%	30,800	6.5%
Section C	6,449	12,500	6.8%	23,500	6.5%
Section D	5,849	11,400	6.9%	21,700	6.6%
KIA Junction (West of KIA)	5,376	10,400	6.8%	19,600	6.5%
KIA Junction (East of KIA)	5,296	10,400	7.0%	20,000	6.8%
Section E	7,044	13,600	6.8%	25,600	6.5%
Sasini Center (West of Jct)	8,793	16,900	6.8%	31,700	6.5%
Sasini Center (East of Jct)	9,665	18,600	6.8%	35,000	6.5%
Sekou Toure Way R/A (West)	12,659	24,400	6.8%	46,000	6.5%
Sekou Toure Way R/A (East)	9,229	18,400	7.1%	35,600	6.8%
Moshi R/A (West of R/A)	5,845	12,200	7.6%	24,200	7.1%
Moshi R/A (East of R/A)	14,860	28,200	6.6%	52,300	6.4%
Moshi Town West (West)	16,648	32,700	7.0%	62,800	6.7%
Moshi Town West (East)	14,740	28,700	6.9%	54,700	6.7%
Moshi Town East (West)	15,584	30,400	6.9%	58,200	6.7%
Moshi Town East (East)	15,125	29,400	6.9%	56,100	6.7%
Himo Jct (West)	7,285	13,400	6.3%	24,000	6.0%
Himo Jct (East)	5,480	9,800	6.0%	16,800	5.5%
Mwika Jct (West)	2,306	4,500	6.9%	8,400	6.4%
Mwika Jct (East)	1,685	3,300	7.0%	6,100	6.3%
KIA Access	2,648	5,200	6.9%	9,900	6.7%

Note: The above figures exclude motorcycles.
Source: JICA Study Team

The following figures compare 2025 projected traffic volume by section and indicates that two forecasts provide similar traffic volume in most road sections, whereas the larger traffic volume is projected in/around Moshi by the Study.

[3] Conversion Rate of Passenger Car Unit applied to F/S is also applied to this Study: Motorcycle (0.5), Passenger Cars (1.0), Small Buses (1.3), Large Bus (1.6), 2 Axles Truck (1.5), 3+ Axles Truck (1.8), Trailer (2.2).



Source: JICA Study Team

Figure 4.2.2 Future Traffic Demand in 2025 (PCU/day)

4.2.3 Generalized Traffic Demand

(1) Generalized Traffic Demand by Development

In Arusha and Kilimanjaro Region, there are two large-scale development projects, namely, (i) Usa River Development (Usa River Satellite City) and (ii) Safari City.

As shown in the following figure, Safari City is located on the west of Arusha city centre, therefore, influence of the project to Arusha-Holili Road is considered minimal. However, Usa River Development, located in the southern Usa, might have a considerable impact to Arusha-Holili Road and therefore the vehicular trip generated by the project is projected in the Study.

Source: JICA Study Team

Figure 4.2.3 Location of Development Project

Usa River Development is the large-scale residential-commercial development as its features are summarized in the following table. The implementation of the project will start in 2016 and implementation period is 5 – 10 years, according to the National Housing Corporation.

Table 4.2.5 Features of Usa River Development

		Units	Gross Leasable Area (sq.m)	Total Built up Area (sq.m)	Plot Area (sq.m)
Residential	Villa	196	82,653	82,653	250,269
	Apartments	3,253	357,242	420,285	239,400 (Mix use building)
Commercial	Office	-	111,571	138,693	
	Retail, Traditional Market	-	41,660	41,660	
Covered Parking		-	-	16,056	
Leisure, Restaurant, Hotel Apartments & Hospitality		-	40,820	43,316	62,056
Furnished Res. Apartment		379	48,095	56,582	20,208
Sports		-	5,664	5,664	14,160
Medical		-	10,470	10,470	6,980
Education		-	21,672	28,004	45,978
Multi-purpose & Conference		-	11,880	11,880	7,920
Community Facilities		-	1,866	3,116	14,129
Green Area & Parks		-	-	-	171,300
Utility		-	-	5,881	11,762
Infrastructure-ROW-Open space		-	-	-	365,767
Total		3,828	733,593	864,260	1,209,929

Source: National Housing Company

Based on the projected population of the project, the traffic generated by the Usa River Development is estimated at around 13,800 vehicles per day as shown in the following table. Assuming half the vehicular trips are made in an internal zone (within Usa River Development), there is a high possibility that around 7,000 vehicles will use part of Arusha – Holili Road, which has a significant impact on the Arusha – Holili Road.

Table 4.2.6 Projected Population and Traffic Generation

	Project Population	Est. Traffic generation	Remarks
Residence	19,140	3,828	1 vehicle per unit
Staff	12,000	3,372	3.6 passenger per vehicle
Visitors	23,660	6,648	3.6 passenger per vehicle
Total	54,800	13,848	

Source: JICA Study Team

Note: Average vehicle occupancy 3.6 is an average of occupancy of passenger cars by F/S in 2011.

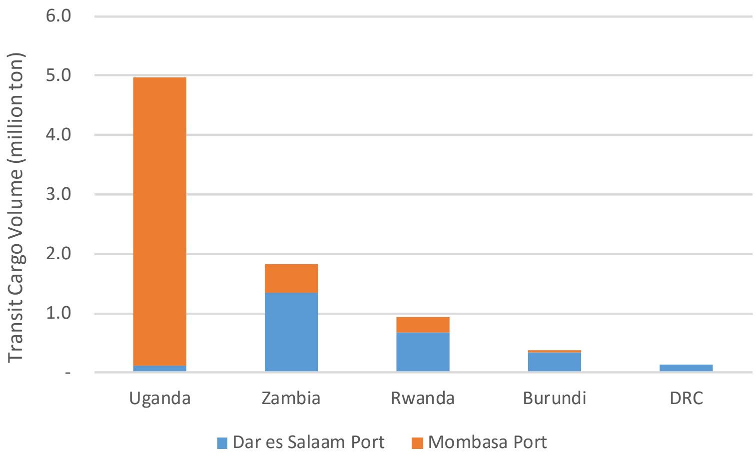
(2) Diverted Truck Traffic from Dar es Salaam Port

Both Dar es Salaam Port in Tanzania and Mombasa Port in Kenya are the gateways of the international freight for Tanzania and Kenya as well as land-locked countries such as Uganda, Zambia, Rwanda, Burundi and DRC. The following table summarized transit cargos handed in Dar es Salaam Port and Mombasa Port and indicates that Dar es Salaam Port mainly handles the transit cargos to/from Zambia, Rwanda, Burundi and DRC, whereas Mombasa Port handles majority of transit cargos to/from Uganda.

Table 4.2.7 Transit Cargo through Dar es Salaam Port and Mombasa Port in 2012

Country	Dar es Salaam Port (ton)		Mombasa Port (ton)		Total (ton)
Uganda	122,318	2%	4,850,000	98%	4,972,318
Zambia	1,352,457	74%	482,358	26%	1,834,815
Rwanda	670,634	72%	260,238	28%	930,872
Burundi	346,149	90%	39,160	10%	385,309
DRC	141,536	100%	0	0%	141,536

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.4 Transit Cargo through Dar es Salaam Port and Mombasa Port in 2012

Based on the share of transit cargo between two ports and estimated time from two ports to land-locked countries, the following logit model is developed.

$$P_{DSM} = \frac{\exp(U_{DSM})}{\exp(U_{DSM}) + \exp(U_{MBA})}$$

$$P_{MBA} = 1 - P_{DSM}$$

Where,

P_{DSM}: Probability of Dar es Salaam Port,

P_{MBA}: Probability of Mombasa Port,

U_{DSM}: Utility function of Dar es Salaam Port, and

U_{MBA}: Utility function of Mombasa.

Utility functions are estimated by the regression analysis based on the transit cargo in 2012 handled at two ports as follow,

$$U_{DSM} = -0.314 \times T_{DSM} + 0.613 \qquad (R^2=0.802)$$

$$U_{MBA} = -0.314 \times T_{MBA}$$

Where,

T_{DSM}: Travel time from Dar es Salaam Port to destination in hour including cross-border,

T_{MBA}: Travel time from Mombasa Port to destination in hour including cross-border, and

Time required to through cross-border at Holili/Taveta is 3.7 hours which is the average time based on the results of road side truck interview survey in the previous MP.

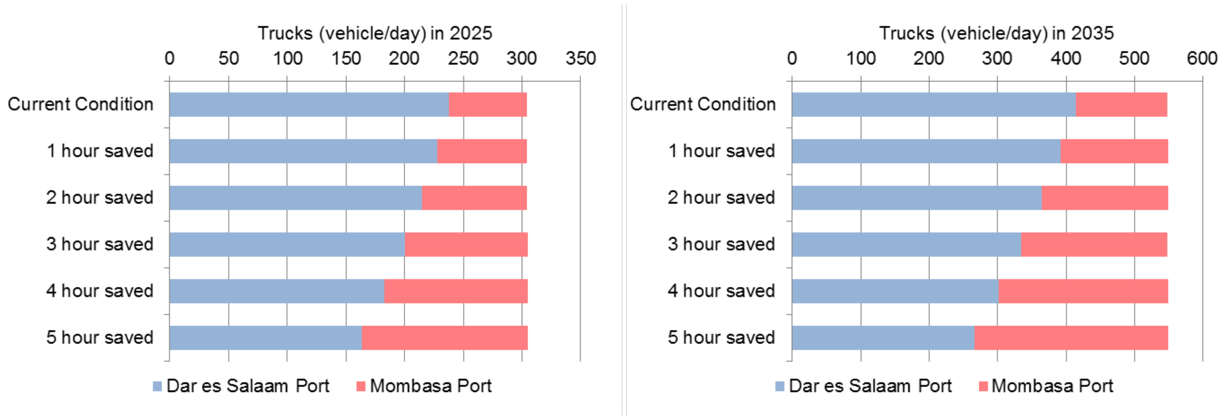
By estimated model and zonal impedance, namely, travel time between origin and destination, proportion through Dar es Salaam or Mombasa Ports are calculated by OD pair. Based on estimated proportion of two ports by OD pair and forecasted truck trip relevant to two ports through Arusha-Holili Road by OD, truck traffic volume relevant to two ports are computed.

To estimated diverted traffic from Dar es Salaam Port to Mombasa Port, five cases by travel time reduction between Mombasa Port and Holili are defined as shown in the following table. Suppose travel time between Mombasa Port and Holili including border crossing time is reduced to three hours, the transit cargo volume equivalent to 38 trucks in 2025 and that of 80 trucks in 2035 is expected to divert from Dar es Salaam port to Mombasa Port.

Table 4.2.8 Forecasted Diverted Trucks by Time Saving between Mombasa Port and Holili

Travel Time between Mombasa Port – Holili	2025 (trucks veh./day)				2035 (trucks veh./day)			
	Dar es Salaam Port		Mombasa Port		Dar es Salaam Port		Mombasa Port	
Current Condition	238	(78%)	66	(22%)	414	(75%)	134	(25%)
1 hour saved	228	(75%)	76	(25%)	392	(71%)	157	(29%)
2 hour saved	215	(71%)	89	(29%)	365	(67%)	184	(33%)
3 hour saved	200	(66%)	105	(34%)	334	(61%)	214	(39%)
4 hour saved	183	(60%)	122	(40%)	301	(55%)	248	(45%)
5 hour saved	164	(54%)	141	(46%)	266	(48%)	283	(52%)

Source: JICA Study Team
Note: Travel time includes waiting time for border crossing.



Source: JICA Study Team

Figure 4.2.5 Forecasted Diverted Trucks by Time Saving between Mombasa Port and Holili

CHAPTER 5 STUDY OF ALTERNATIVE ROAD AND BRIDGE PLAN

5.1 Supplemental Natural Condition Surveys

5.1.1 Topographical Survey

Topographical survey was conducted in October 2015. Main objective is to capture terrain features around the area of the Kikafu River for an alternative study on its highway and bridge plan.

The survey work used the Unmanned Aerial Vehicle (UAV) (i.e., Drone) in mapping. The UAV Mapping uses a GPS guided aircraft flying in approximately 300m above the ground level to take aerial photos of the survey area. The advantage of this technology is provision of land cover information with higher resolution and reasonable costs compared to that of the conventional survey and enhances the accuracy of survey results.

Figure 5.1.1 A Scene from UAV Survey

(1) Specifications of the Survey

Table 5.1.1 Datum

Datum Name:	ARC 1960
Ellipsoid Name:	Clarke 1880
Semi-major Axis:	6,378,249.145m

Source: JICA Study Team

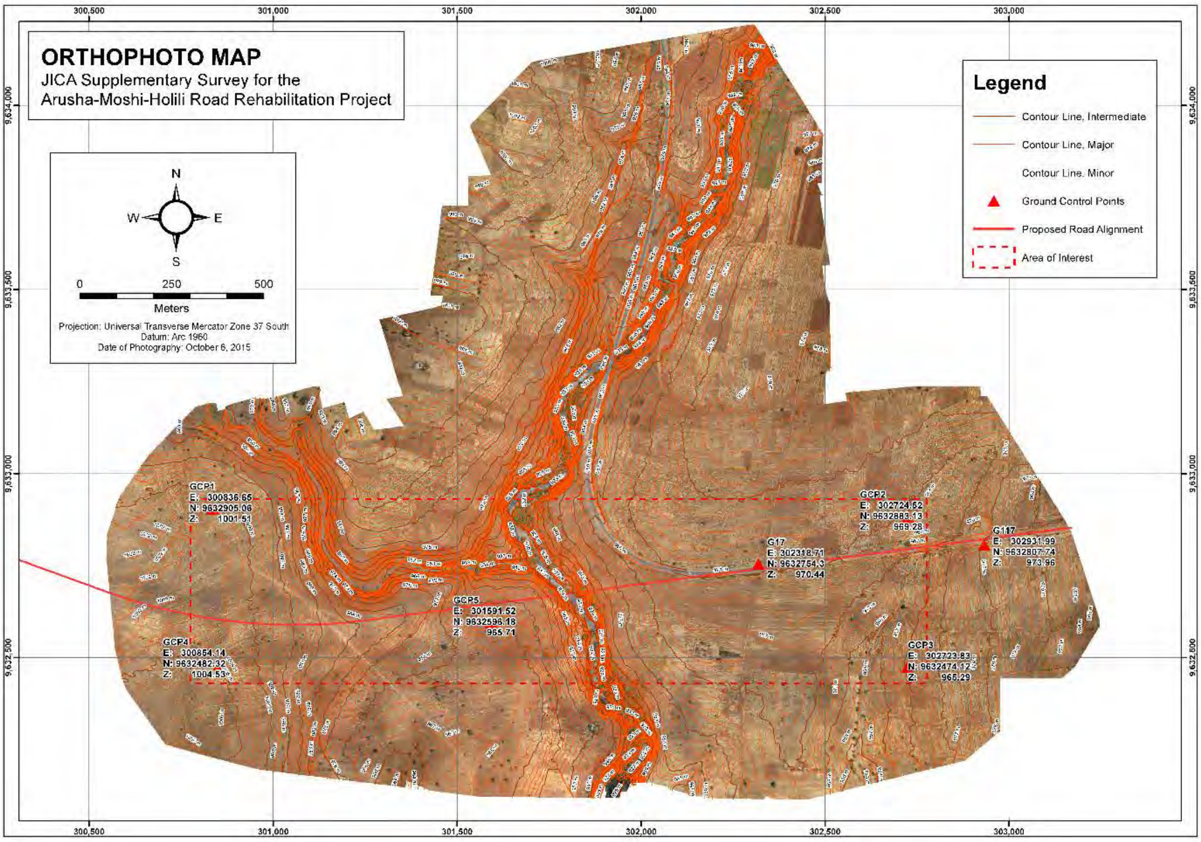
Table 5.1.2 Projection

Projection Type:	Universal Transverse Mercator (UTM)
Zone No.:	37S
Latitude of Origin:	00° 00′ 00.00000″ N
Central Meridian:	39° 00′ 00.00000″ E
Scale Factor:	0.9996
False Easting:	500,000.000m
False Northing:	0.000m

Source: JICA Study Team

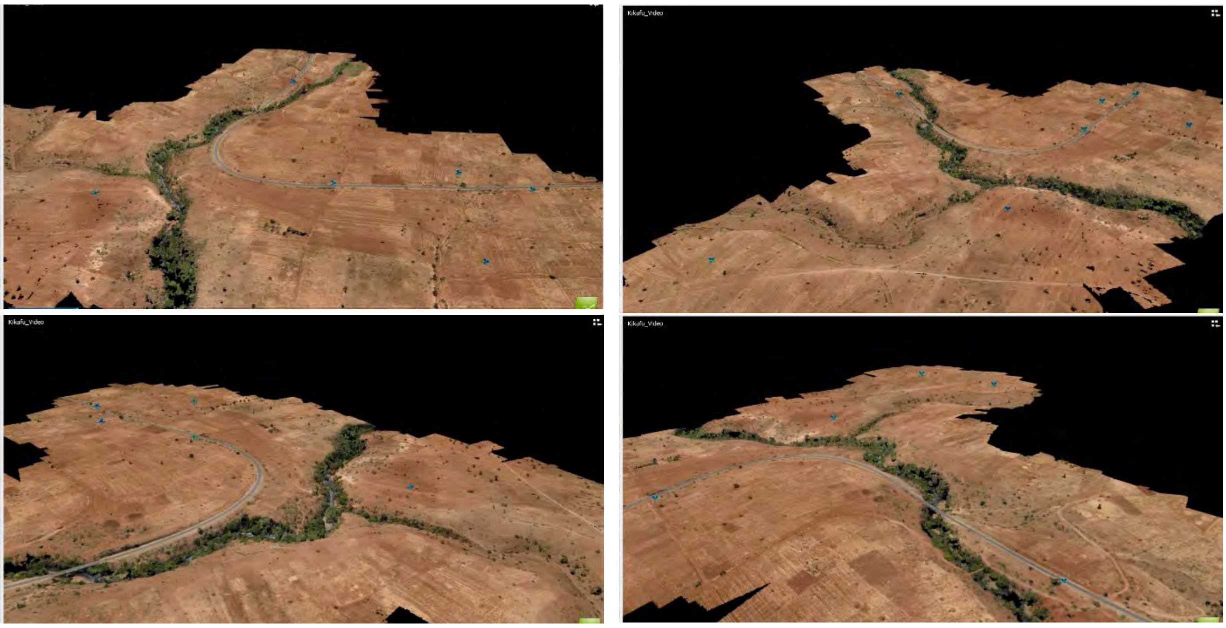
(2) Survey Results and Land Analysis

Orthophoto map with contours and 3 dimensional image video were attained as a result of the survey.



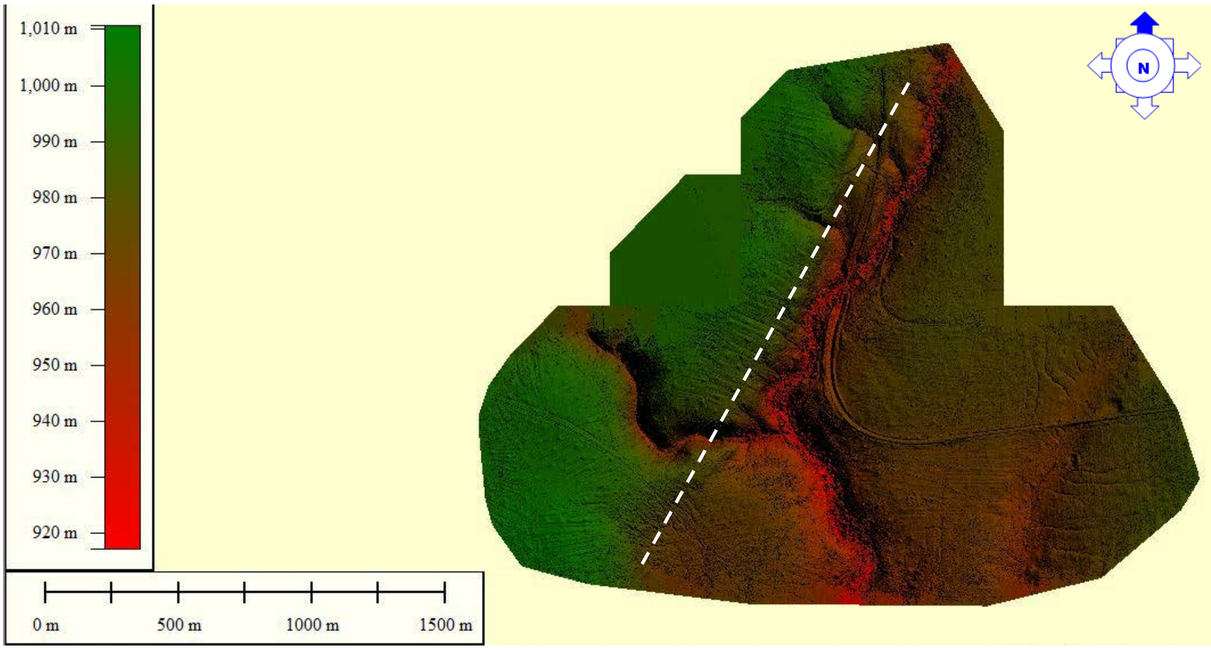
Source: JICA Study Team

Figure 5.1.2 Orthophoto Map



Source: JICA Study Team

Figure 5.1.3 Screenshots of Image Video



Source: JICA Study Team

Figure 5.1.4 Slope-Thematic Map

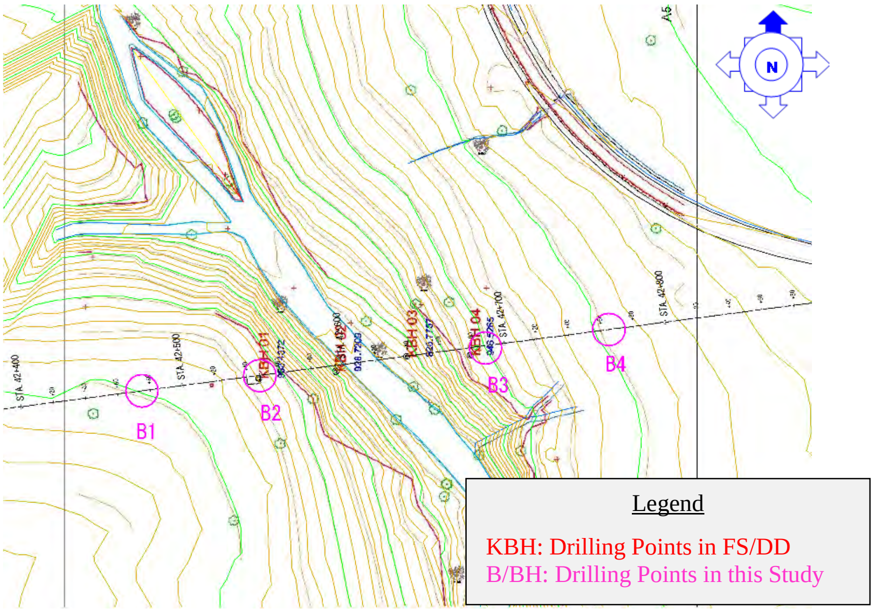
Figure 5.1.4 confirms a line running south to north along the river as was supposed to be a fault created by the past earthquake.

5.1.2 Geotechnical Investigation

(1) Geotechnical Investigation in F/S and D/D

Drilling of four (4) boreholes was carried out in the F/S and D/D of which positions were shown in the report, however coordinates of the drilling points were not available in the report.

The Study Team superimposes the points on the topographical map as shown in Figure 5.1.5.



Source: JICA Study Team

Figure 5.1.5 Drilling Points in F/S and D/D and the Study

In addition to the drillings, the Point Load Tests and Unconfined Compressive Strength (UCS) Tests were carried out in F/S and D/D.

(2) Scope of Works of the Field Investigation

Geotechnical investigation was carried out in September and October 2015. The objective of the geotechnical investigation was to determine geotechnical features for the proposed structural foundations and any associated infrastructure.

Table 5.1.3 Exploratory Drilling Locations and Depth

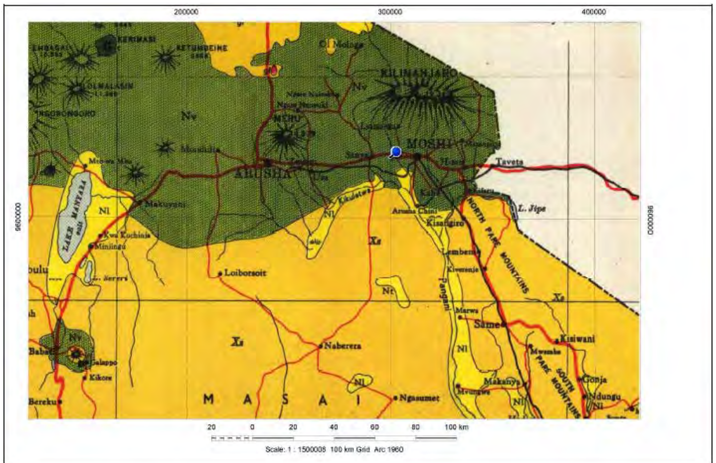
Borehole ID	Final depth (mbgl: meter below ground level)	Elevation (m)	Coordinates	
			Easting	Northing
BH1	30.0	959.81	301659.4	9632655
BH2	30.0	953.41	301732	9632664
BH3	30.0	950.26	301871	9632681
BH4	27.0	960.72	301946.9	9632692

Source: JICA Study Team

As shown in Figure 5.1.5, the BH2 and BH3 are planned to be same positioned with KBH01 and KBH04 so as to compare and confirm the geological structures.

(3) Geological Features

According to the geological map of Tanganyika, 1959 the proposed new Kikafu Bridge is located in the upper reaches of the River Panagani Drainage basin. The Kikafu River is a tributary of the Pangani River. Its source is in Mount Kilimanjaro and being a young river at the site, it has incised deeply into the volcanic deposits spread in the plains beyond the foot of the mountain. The River generally drains South East towards the Indian Ocean, and at the site, the river incised a valley. The area is in a plateau that has successive cycles of lava flows and volcanic deposits associated with the formation of Mount Kilimanjaro, and the rivers flowing within. The map above illustrates the general geology of the project are.



Key :
N1 : Recent Lacustrine deposits including sand, silt, limestone, volcanic tuff. Post Karoo NEOGENE formations
Nv : Alkaline Volcanics, olivine basalt,alkali basalt, trachyte, nephelinite and pyroclasts. Post Karoo NEOGENE formations.
Xs: Crstalline Limestone, Marble, Schist, Gneiss, Quartzites, Amphibolites and migmatites - USAGARAN system.

Source: Geological Map of Tanganyika, 1959

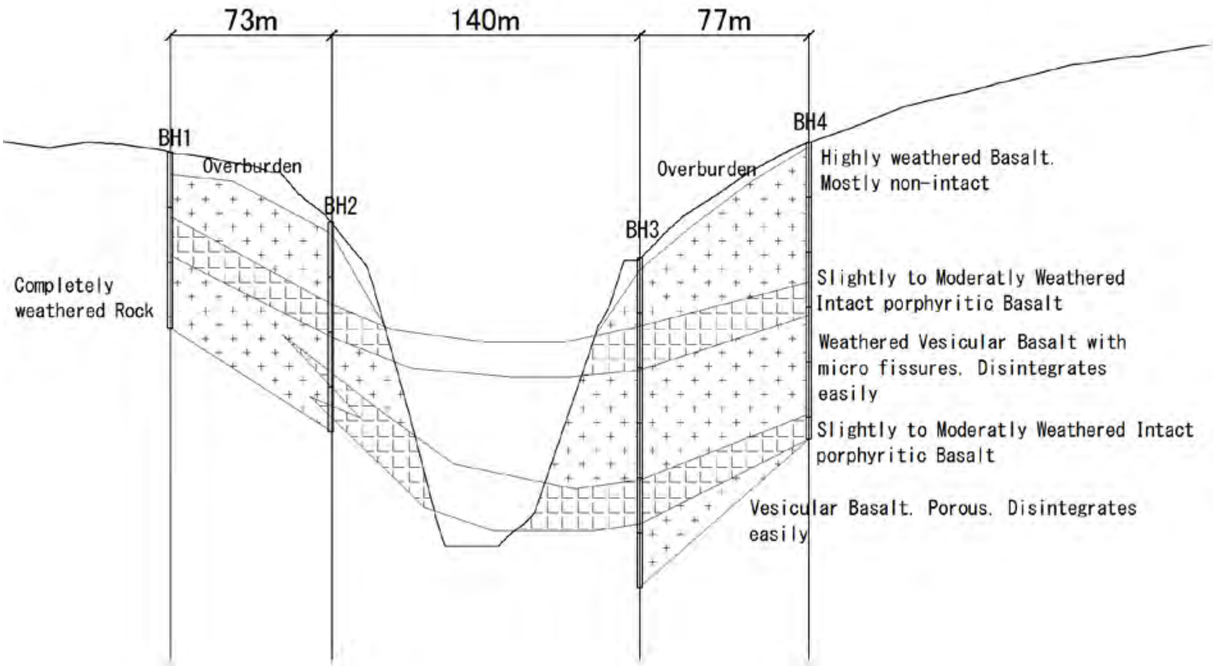
Figure 5.1.6 Geological Map of Tanganyika, 1959

(4) Drilling Borehole Results

A sum of four (4) boreholes were drilled to depths ranging from 27.0m to 30.0m from the surface. The drill met weathered rock showing Rock Quality Designation (RQD) ranging 30 to 80 continued up to 30m depth. The samples were generally highly weathered coarse gained rock. SPT value 70-80 was achieved at 3m depth below and SPT was refused due to hardness of rock, thereafter. Each boring log are attached in the separate survey report.

(5) Geological Profile

A geological profile was prepared by developing individual drilling results, as shown in the following figure.

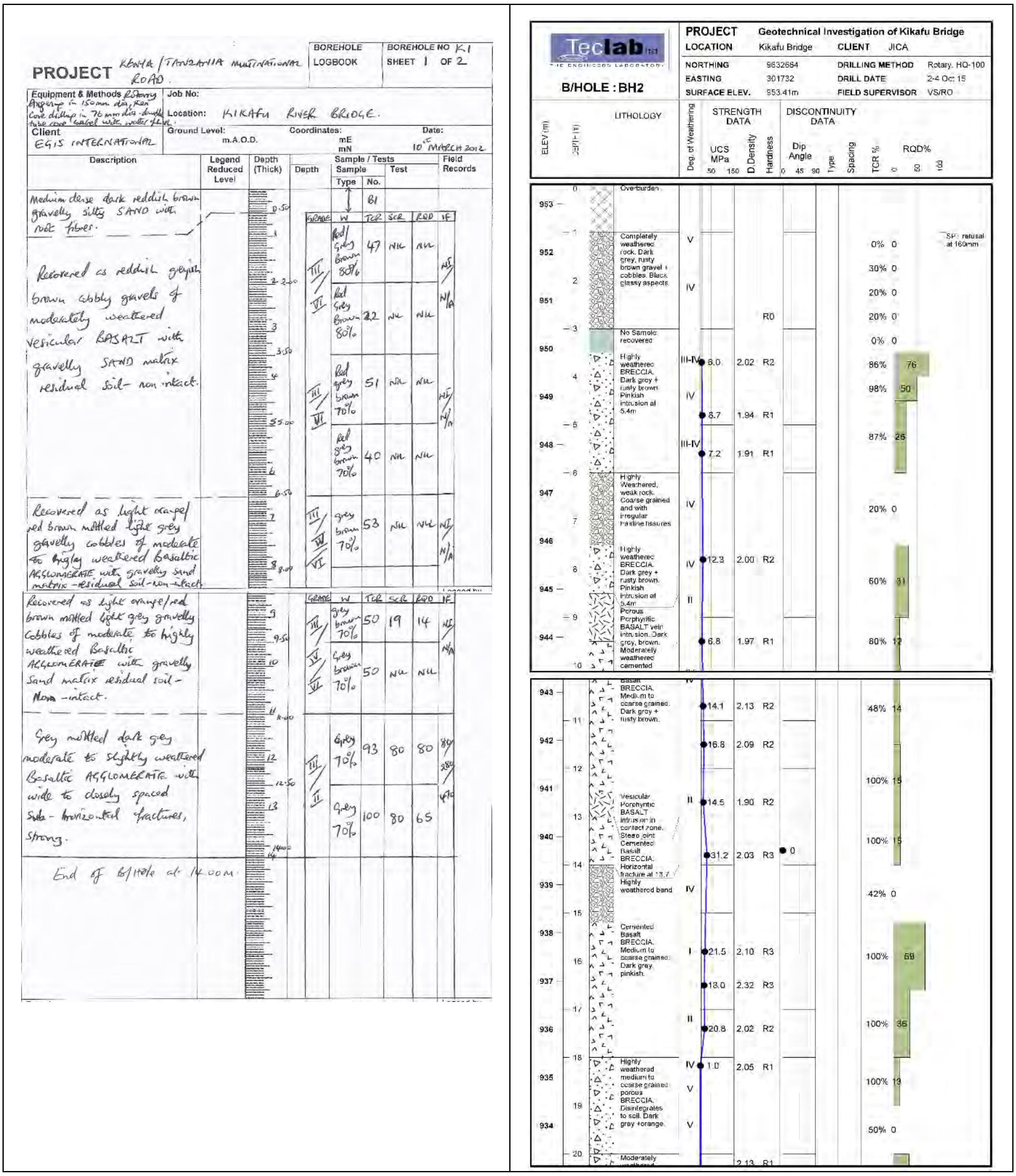


Source: JICA Study Team

Figure 5.1.7 Geographical Profile

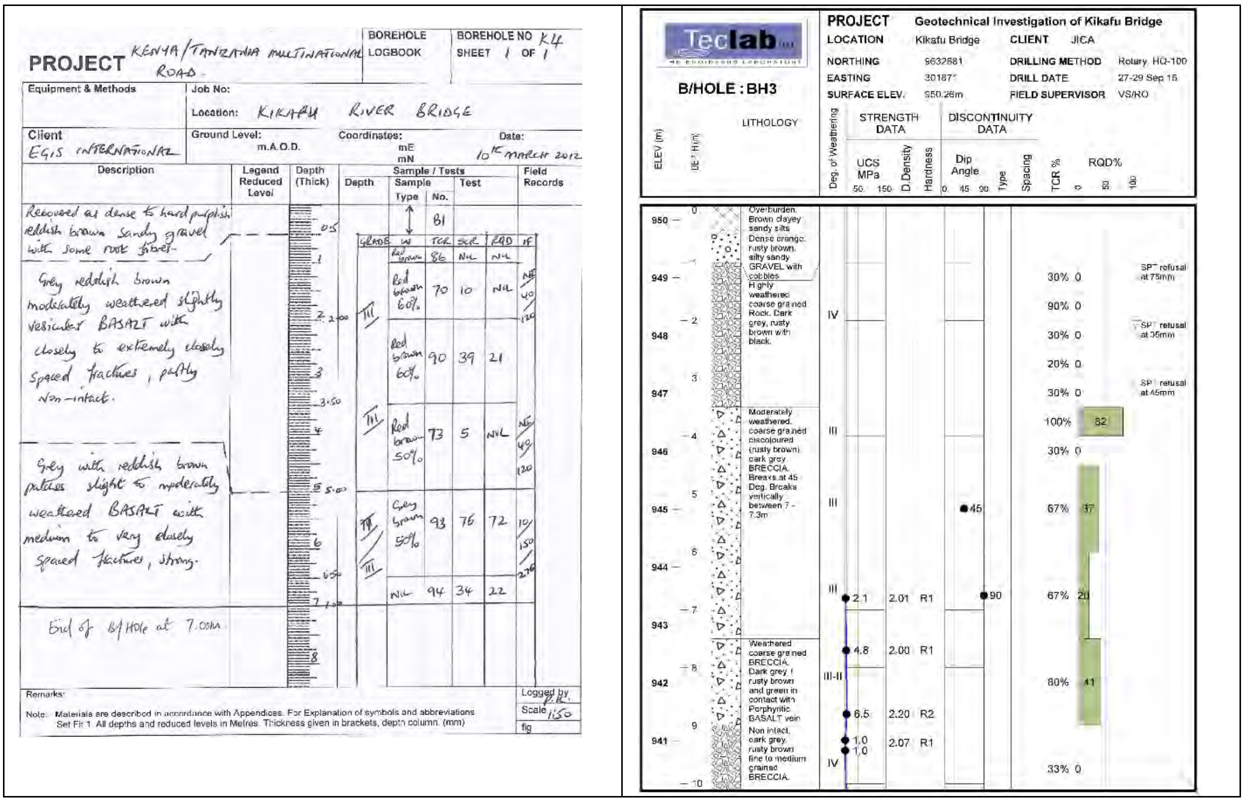
(6) Comparison with the Results of F/S and D/D

As explained earlier, the positions of BH2 and BH3 were decided so as the positions be same as KBH01 and KBH04 of F/S and D/D and the comparisons are expected to help to confirm geological components. The logs are compared between KBH01 and BH2, and KBH 04 and BH3 as shown in Figure 5.1.8 and Figure 5.1.9.



Source: JICA Study Team

Figure 5.1.8 Boring Log for KBH01 and BH2



Source: JICA Study Team

Figure 5.1.9 Boring Log for KBH04 and BH3

It is observed that there is some difference between the results. Interpretations and recommendation are made for these differences as detailed below.

- Since the rocks are heavily weathered, the progresses of the weathering are possibly to differ depending on the points. They could have similar progress of the weathering but could be observed at different depths.
- Further geotechnical investigations are recommended at the structure designed points so as to deliver appropriate quality of design in the detailed design stage.

(7) Lithology

Three rock types are identified on the logs as follows - in the order of their consistency

Volcanic breccia - which appears as a brown rock that is easily broken and has very poor recovery. It has been labelled as a moderately to highly weathered rock because of its intense discoloration. This clay in the rock has a distinctly high rate of water absorption.

Cemented basalt breccia - which is a denser and more compact rock and with mush less water absorption properties. The larger grains varying from medium to large. This rock type has been found to generally have the strongest compressive strength.

Basalt - An alkaline basalt, sometimes green indicating presence of olivine. It has varying degrees of porosity some of which can be seen as large vesicles, in which case, the rock has been identified as vesicular basalt. The pores inadvertently lower the compressive strength of the rock. The largest vesicles are found at the bottom of BH1 and BH2.

The bulk of the hard rock in the three boreholes is the cemented basalt breccia. The alkaline basalt occurs as relatively thin intrusions 0.5m thick. These three geologic rock types subtly merge into each other and it is difficult to make precise distinction of the boundaries. It is however, clear that the rocks have been deposited and subsequently weathered in successive layers at different times.

The layering on the western approach in BH1 and BH2 is more distinct than on the eastern approach in BH3 and BH4. There is a layer of hard competent rock about 5m thick consisting largely of the cemented basalt breccia but with Porphyritic Basalt intrusions below level approximate elevation 940m, which appears to be the clearest marker between successive volcanic deposits. The rock above this layer is generally weak and /or weathered, except for a hard cap in BH1. Above elevation 940m, the rocks are weaker moving eastward. Below elevation 935m, the rocks closest to the river in BH2 and BH3 are more weathered. BH4 was not deep enough to investigate the strength of the formations below elevation 935m.

Finally, BH 3 is layered differently from the other 3 boreholes. Its hard mid layer occurs at a deeper elevation, suggesting the possibility of earlier downward faulting, before later volcanic deposits / lava flows.

(8) Engineering Properties of the Rock

In addition to the recovery percentages determined visually on site, the rocks properties for UCS, dry density and absorption have been determined, and these have been used to arrive to a geological profile for the site. The properties of the rock have been summarized in the borehole logs and summary laboratory test results.

It is seen that the rock strength is varies with depth. It does not necessarily increase uniformly over the investigated depth.

The rock from BH 1 has the highest average compressive strength (about 22MPa). Both BH 1 and 2 on the western approach to the river have large distinct weak formations between competent rocks. And at the bottom of both of these boreholes, there is considerable loss of drilling fluid during drilling, indicating the possibility of large continuous joints in the pores. The rock at the bottom of these boreholes is intact but with large vesicles.

The rock has been analysed further using the Rock Mass Rating (RMR) classification developed by South African Council for Scientific and Industrial Research (CSIR - 1989) based on 5 key parameters:

Rock strength determined from the UCS
RQD
Joint spacing
Ground Water
Condition of the joints

Table 5.1.4 Rock Mass Rating (RMR) System

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 – 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		

5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125
		(Joint water press)/	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favorable	Favorable	Fair	Unfavorable	Very
Ratings	Tunnels &	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES					
Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m	1 year for 10 m	1 week for 5 m span	10 hrs for 2.5 m	30 min for 1 m
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions					
Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weather ing Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45- 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favorable	Favorable	Very unfavorable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavorable	Fair	

Source: South African Council for Scientific and Industrial Research

RMR results for each borehole are shown in the following tables.

Table 5.1.5 RMR of BH1

Rock Mass Rating	UCS Rating	RQD Rating	Joint space Rating	G.Water Rating	J. surface Rating	RMR	Rock Class	Safe Bear. Pressure
4.0 - 5.6m	0	3	10	15	20	**48**	D - Poor rock	0.5 Mpa
5.6 - 7.1m	4	3	10	15	25	**57**	D - Poor rock	0.5 Mpa
7.1 - 8.1m	4	8	10	15	25	**62**	C - Fair rock	1-2 Mpa
8.1 - 9.2m	2	13	10	15	25	**65**	C - Fair rock	1-2 Mpa
9.2 - 10.1m	2	13	10	15	25	**65**	C - Fair rock	1-2 Mpa
10.1 - 11.4m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
11.4 - 13.0m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
13.0 - 15.4m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
15.4 - 18.2m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
18.2 - 20.0m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
20.0 - 21.6m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
21.6 - 23.2m	2	8	10	15	20	**55**	D - Poor rock	0.5 Mpa
23.2 - 24.1m	2	13	10	15	25	**65**	C - Fair rock	1-2 Mpa
24.1 - 25.0m	2	13	10	15	25	**65**	C - Fair rock	1-2 Mpa
25.0 - 26.2m	1	17	10	15	25	**68**	C - Fair rock	1-2 Mpa
26.2 - 27.1m	4	17	10	15	10	**56**	D - Poor rock	0.5 Mpa
27.1 - 28.0m	2	13	10	15	10	**50**	D - Poor rock	0.5 Mpa
28.0 - 29.0m	7	17	10	15	10	**59**	D - Poor rock	0.5 Mpa
29.0 - 30.0m	7	17	10	15	10	**59**	D - Poor rock	0.5 Mpa

Source: JICA Study Team

Table 5.1.6 RMR of BH2

Rock Mass Rating	UCS Rating	RQD Rating	Joint space Rating	G.Water Rating	J. surface Rating	RMR	Rock Class	Safe Bear. Pressure
4.0 - 5.6m	2	8	10	15	20	**55**	D - Poor rock	0.5 Mpa
5.6 - 7.1m	2	8	10	15	20	**55**	D - Poor rock	0.5 Mpa
7.1 - 8.1m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
8.1 - 9.2m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
9.2 - 10.1m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
10.1 - 11.4m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
11.4 - 13.0m	2	3	10	15	25	**55**	D - Poor rock	0.5 Mpa
13.0 - 15.4m	2	3	10	15	25	**55**	D - Poor rock	0.5 Mpa
15.4 - 18.2m	4	3	10	15	25	**57**	D - Poor rock	0.5 Mpa
18.2 - 20.0m	2	8	10	15	25	**60**	C - Fair rock	1-2 Mpa
20.0 - 21.6m	2	13	10	15	25	**65**	C - Fair rock	1-2 Mpa
21.6 - 23.2m	2	8	10	15	25	**60**	C - Fair rock	1-2 Mpa
23.2 - 24.1m	0	3	10	15	20	**48**	D - Poor rock	0.5 Mpa
24.1 - 25.0m	0	8	10	15	20	**53**	D - Poor rock	0.5 Mpa
25.0 - 26.2m	0	8	10	15	20	**53**	D - Poor rock	0.5 Mpa
26.2 - 27.1m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
27.1 - 28.0m	2	8	10	15	10	**45**	D - Poor rock	0.5 Mpa
28.0 - 29.0m	2	8	10	15	10	**45**	D - Poor rock	0.5 Mpa
29.0 - 30.0m	2	8	10	15	10	**45**	D - Poor rock	0.5 Mpa

Source: JICA Study Team

Table 5.1.7 RMR of BH3

Rock Mass Rating	UCS Rating	RQD Rating	Joint space Rating	G.Water Rating	J. surface Rating	RMR	Rock Class	Safe Bear. Pressure
6.0 - 7.3m	1	3	10	15	20	**49**	D - Poor rock	0.5 Mpa
7.3 - 8.3m	1	8	10	15	20	**54**	D - Poor rock	0.5 Mpa
8.3 - 9.1m	2	8	10	15	20	**55**	D - Poor rock	0.5 Mpa
9.1 - 12.2m	0	3	10	15	20	**48**	D - Poor rock	0.5 Mpa
12.2 - 16.8m	4	3	10	15	20	**52**	D - Poor rock	0.5 Mpa
16.8 - 19.1m	7	8	10	15	25	**65**	C - Fair rock	1-2 Mpa
19.1 - 20.2m	2	8	10	15	25	**60**	C - Fair rock	1-2 Mpa
20.2 - 21.5m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
21.5 - 23.0m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
23.0 - 25.2m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
25.2 - 28.0m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa

Source: JICA Study Team

Table 5.1.8 RMR of BH4

Rock Mass Rating	UCS Rating	RQD Rating	Joint space Rating	G.Water Rating	J. surface Rating	RMR	Rock Class	Safe Bear. Pressure
14.7 - 16.1m	4	3	10	15	20	**52**	D - Poor rock	0.5 Mpa
16.1 - 18.7m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
18.7 - 21.2m	2	3	10	15	20	**50**	D - Poor rock	0.5 Mpa
21.2 - 23.1m	4	3	10	15	25	**57**	D - Poor rock	0.5 Mpa
23.1 - 24.1m	4	3	10	15	25	**57**	D - Poor rock	0.5 Mpa
24.1 - 25.1m	4	8	10	15	25	**62**	C - Fair rock	1-2 Mpa
25.1 - 26.1m	2	8	10	15	25	**60**	C - Fair rock	1-2 Mpa
26.1 - 27.5m	2	8	10	15	25	**60**	C - Fair rock	1-2 Mpa

Source: JICA Study Team

It can be seen that this rock classifies as either a poor or fair rock with a conservative safe bearing pressure varying from 0.5MPa to 2MPa. While shallow foundations with a minimum depth of 3m below GL seem feasible, the foundation designer should consider whether the traffic loading is able to activate any quiet slip planes, if any. The steep valley as shown in the geological profile, Figure 5.1.7 presents below unverified indications of preferential slip planes from earlier seismic activity. After verification of the above, the shallow foundations should be on the underlying bedrock, beyond the completely weathered rock zone.

5.2 Study on Alternative Road Design

5.2.1 Design Approach Principal

The project road shall be improved in order to perform sufficient mobility as expected for international freight corridor and shall also contribute to regional economic development including a tourism promotion. Social impacts including land acquisition and kinds of compensation are always unavoidable challenges in developing road network which sometime eliminates benefits from the development due to overburden of budget and long process required.

Quick realization of the Project is also a key because road sections including Kenyan side and dualling of the road section in Arusha have already commenced in construction, the project road section may result in a bottleneck if delayed.

The Study Team therefore understands that appropriate design approach shall mainly be improvement of the existing alignment so as the improvement to be completed within the allocated road reserve as much as possible. Strengthen road safety is another challenge as traffic attempts to drive faster thereby high standard infrastructure required.

As discussed earlier, the road section around the existing Kikafu Bridge has been identified as one of the most critical black spots in the country where many lives have been lost due to traffic accidents. In this particular section, road safety shall therefore be most prioritized in designing.

Taking into account the above, the F/S and D/D introduced a bypass route instead of improvement of existing alignment as shown above which the Study Team concurs.

Design departures shall be limited because the departures may eliminate benefit of high standard design in this particular road section.

5.2.2 Design Standard

The Study Team studies applicable design standards and compares them. As discussed earlier in the previous chapter, old standards and manuals were prepared quite a long time ago and shall not be subject to the Study. The Study Team considers the following standards and manuals and shall be subjected for the comparison as appropriate to the Project.

- Road Geometric Design Manual 2011 edition, Ministry of Works, Transport and Communications; the United Republic of Tanzania. This manual was prepared in May, 2012 as is thus most recent design guideline in the Tanzania.
- Draft Code of Practice for the Trunk Roads 2001 edition, Division of Roads and Transport Technology, CSIR. This manual is commonly employed in the SADC counties (Angola, Botswana, Democratic Republic of Congo, Lesotho, Madagascar, Malawi, Mauritius, Mozambique, Namibia, Seychelles, South Africa, Swaziland, Tanzania, Zambia and Zimbabwe) in designing international trunk road.
- Final Report for Preparation of Transport Facilitation Strategy for the East African Community March 2014, East African Community. This report is most recent transport facility design guideline applicable to the EAC countries (Kenya, Tanzania, Uganda, Rwanda and Burundi) to harmonize the standards prepared by the member countries. It complies with the policy of uniting the countries in economic activities.

Upon the comparison, the road class is considered as an international trunk road and similar. The below table summarizes the comparison of the geometric parameters per elements by the standards.

Table 5.2.1 Comparison of Parameters in Geometric Design by Manuals/Standards

	TZ (DC1)	STACC	EAC
Design Speed (kph)	120 (Flat to Rolling) 90(Rolling to Hilly) 70 (Mountainous)	120	120 (Flat) 100(Rolling) 60 (Mountainous) 60(Steep) Mobility Road Class1
Road Reserve(m)	60	N/A	79-100
Lane Width(m)	3.5	3.7 (recommended) 3.1 (minimum)	3.5 to 3.75
Nos. of Lane	≥4	N/A	
Shoulder Width (m)	2.5	1.5 (minimum) 3.0 (desirable)	2.0(minimum) 1.0 to 1.5 (Rolling, Mountainous)
Median Width (m)	9-12	9.2(desirable) 5.0 (recommended) 1.5(minimum)	N/A
Normal Cross-fall (%)	2.5(AS pave)	N/A	2.5 (ASPave)
Shoulder Cross-fall(%)	2.5(As pave)	N/A	N/A
Minimum Radii	780 (desirable) 667(e_{max}=8%) 760 (e_{max}=6%) Design Speed 120kph	530 (e_{max}=10%) 580 (e_{max}=8%) 650(e_{max}=6%) Design Speed 120kph	N/A
Minimum Curve Length (m)	NA	300	N/A
Maximum Curve Length (m)	NA	1000(Absolute) 800 (Desirable)	N/A
Maximum Grade (%)	3 Design Speed 120kph	3(Flat) 4 (Rolling) 5 (Mountainous~ Design Speed 120kph	3(Flat) 4 (Rolling) 5 (Mountainous~ Design Speed 120kph
Minimum Length of Vertical Curves (m)		180 (100kph) 220 (100kph)	N/A
K value Stopping Sight Distance (Crest)	201	110	N/A
K value Stopping Sight Distance (sag)	74	52	N/A
K value Passing Sight Distance (sag)	664	NA	N/A

Note: Design Class (DC1), Tanzania: AADT more than 8,000
Mobility Road Class 1 EAC: International Trunk Roads/National Trunk Roads
Source: JICA Study Team

The Study Team considers that reviewing the design shall be carried in accordance to the Tanzanian Standard, and the STACC shall also be supplementary referred.

5.2.3 Design Speed

As shown in the above table, all standards/manuals recommend to apply design speed of 120kph for international trunk road. Considering the nature of improvement, 100kph is considered to be reasonably applicable since the geometrical requirements of design speed of 120kph calls large land acquisitions and compensations which is not in line with the improvement strategy.

5.2.4 Road Classification

There are two kinds of classifications of road network in Tanzanian.

(1) Functional Classification

In Tanzania, the existing classification is party based on administrative aspects of the facility and party on functional aspects. The existing network is classified in accordance with the Road Act of 2007.

Table 5.2.2 Functional Road System in Tanzania

Network	Class	Description
National Road	A: Trunk Roads	A national route that two or more regional headquarters; or an international through route that links regional headquarters and another major or important city or town or major port outside Tanzania.
	B:Reginal Roads	Constitute the secondary national routes connecting a truck road and district or regional headquarters in region; or connecting reginal and district headquarters.
District Roads	C: Collector Roads	i) A road linking a district headquarters and a division centers; ii) A road linking a division center with any other division center iii) A road linking a division center with a ward center iv) A road within urban area carrying through traffic which predominantly originates from and destined out of the town and link with either regional or a trunk road
	D: Feeder Roads	i) A road within urban area that links a collector road and other minor road within the vicinity and collects or distributes traffic between residential, industrial and principal business center of town; ii) A village access road linking wards to other wards center
	E:Community Roads	A road within a village or a road links a village to a village

Source: Road Geometric Design Manual edited by JICA Study Team

(2) Design Classification

The main requirement is that a road shall function satisfactory during its service life without major improvements. In terms of servicing the users, this requirement implies absence of major delays or breakdowns in the traffic flow on the regular basis during the design life. To ensure satisfactory functioning of the road, a range of the geometric design standards may be applicable to one functional class as shown in the Table 5.2.3.

Table 5.2.3 Linkage between Road Design Class and Functional Class

Road Design Class	AADT*[veh/day]in the design year	Functional Class				
		A	B	C	D	E
DC1	>8,000					
DC2	4,000-8,000					
DC3	1,000-4,000					
DC4	400-1,000	M				
DC5	200-400		M			
DC6	50-200					
DC7	20-50					
DC8	<20					

Applies to roads in flat to rolling terrain

M: Minimum standard for the appropriate functional class.

Source: Road Geometric Design Manual edited by JICA Study Team

The project road is evidently categorized into A of functional class and DC1of design class, considering traffic characteristics of the project road.

(3) Design Speed

As shown in Table 5.2.1, all the standards and manuals recommend to apply design speed of 120kph for an international trunk road. Considering nature of improvement as discussed earlier, the design speed of 100kph in normal section and 50kph in town section are reasonable choices (applied to the previous F/S and D/D). Reasons are that the design speed of 100kph still brings sufficient infrastructure complied with high mobility as required for the international truck road and geometrical requirements of the design speed of 120kph calls large scale land acquisitions and compensations which is not in line with the improvement strategy for the Project.

Table 5.2.4 shows required geometrical parameters under design speed of 100kph and 50kph according to the Tanzanian Manual.

Table 5.2.4 Required Geometrical Parameters under Design Speed 100kph and 50kph according to the Tanzanian Manual

Elements		Parameters		Remarks
		100kph	50kph	
Road Reserve(m)		60		Decided by Design Class
Lane Width(m)		3.5		Decided by Design Class
Nos. of Lane		≥4		Decided by Design Class
Shoulder Width (m)		2.5		Decided by Design Class
Median Width (m)		9-12		Decided by Design Class
Normal Cross-fall (%)		2.5 (AS pave)		General Requirement
Shoulder Cross-fall (%)		2.5 (As pave)		General Requirement
Head Room	General(m)	4.6		General Requirement
	Under Structure(m)	5.5 (recommended) 5.2 (Absolute)		General Requirement
	Under Power Cable(m)	7.0 (high power) 6.0(high power)		General Requirement
Lateral Clearance	H<0.2m (m)	0.25	0.00	General Requirement
	0.2m≤H (m)	2.00	0.60	General Requirement
	Guardrail (m)	0.75	0.60	General Requirement
Minimum Radii (m)		450 (desirable) 394(e_{max}=8%) 440 (e_{max}=6%)	100 (desirable) 82(e_{max}=8%) 90 (e_{max}=6%)	
Maximum Grade (%)		3.0	7.0 (rolling) 9.0(hilly/mountainous)	
K value Stopping Sight Distance (Crest)		105	10	
K value Stopping Sight Distance (sag)		51	12	
K value Passing Sight Distance (sag)		475	126	

Source: Road Geometric Design Manual compiled by JICA Study Team

5.2.5 Service Level

Tanzanian manual provides the guideline of maximum AADT for two lane roads with the following assumption,

Traffic mix: 25% of large traffic,

Directional split: 60/40,

No passing zone

- Flat terrain: 20%
- Rolling to Hilly terrain: 40%
- Mountainous terrain:60%
- Ratio of 30th HHV to AADT:0.15

Table 5.2.5 Maximum AADT for Two Lane Road

Level of Service		Maximum AADT		
	Description	Flat	Rolling	Hilly/Mountain
A	Free Flow	1,600	700	300
B	Reasonable Free Flow	3,200	1,650	700
C	Stable Flow	5,200	3,000	1,250
D	Approaching Unstable Flow	8,700	4,500	1,900

Source: Road Geometric Design Manual edited by JICA Study Team

The project road shall cater traffic with stable flow which requires to achieve the level of service of C or above.

5.2.6 Geometrical Design

(1) Horizontal Alignment

1) Section for Exiting Alignment Strengthening

As discussed, the horizontal alignment design shall follow the existing parameters as much as possible. Minor adjustment in the geometrical design at the dual carriageway section is needed as explained.

2) Section for Kikafu Bridge

In the F/S and D/D, the horizontal alignment including 3 tangents and 2 circular curves without transition curves, was designed in the section and the gazetting of road reserve has also been completed accordingly.

The Study Team verifies a design compliance of design speed 100kph on the alignment according to the Tanzanian manual. As a result, the Study Team confirms the satisfaction of the requirements.

The Study Team finds no reasonable amendment is needed on the alignment design at this stage.

(2) Vertical Alignment

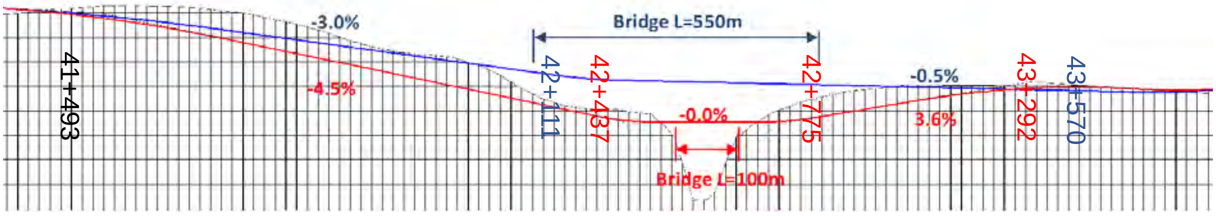
1) Section for Exiting Alignment Strengthening

As discussed, the geometrical design shall follow the existing geometrical parameters as much as possible. Minor adjustment will be needed so as to satisfy required sight distance (i.e. K value) as required in the applied design speed.

2) Kikafu Bridge Section

Alternative study is given on the vertical alignments of 3%-A and 4.5%-B in order to confirm viability.

As shown in Table 5.2.4, the design speed 100kph requires maximum 3% of gradient in vertical alignment designing which may call the cost implication as it requires longer span bridge. On the other hand, the steeper gradient (i.e. 4.5% in this alternative study) requires relatively shorter span bridge which gives smaller impact on the project budget.

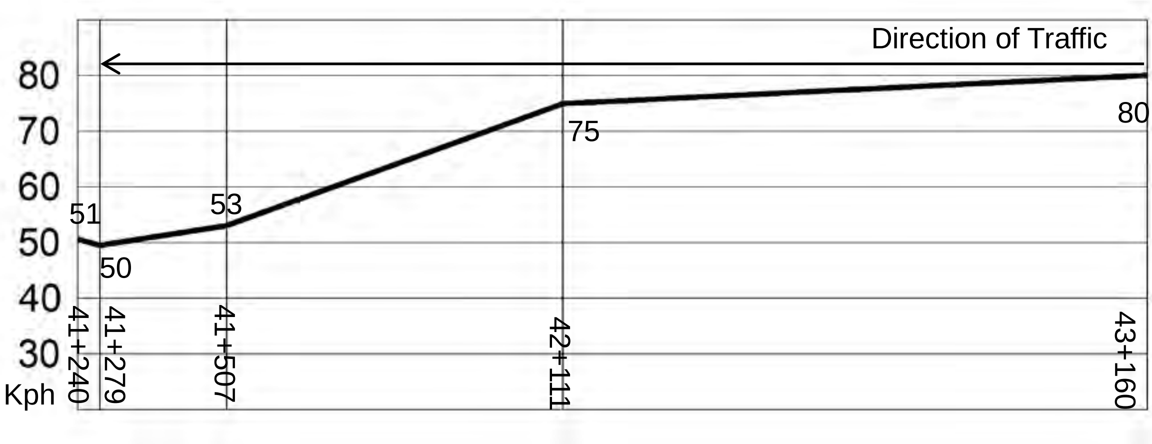


Source: JICA Study Team

Figure 5.2.1 Alternative Study on Vertical Alignments

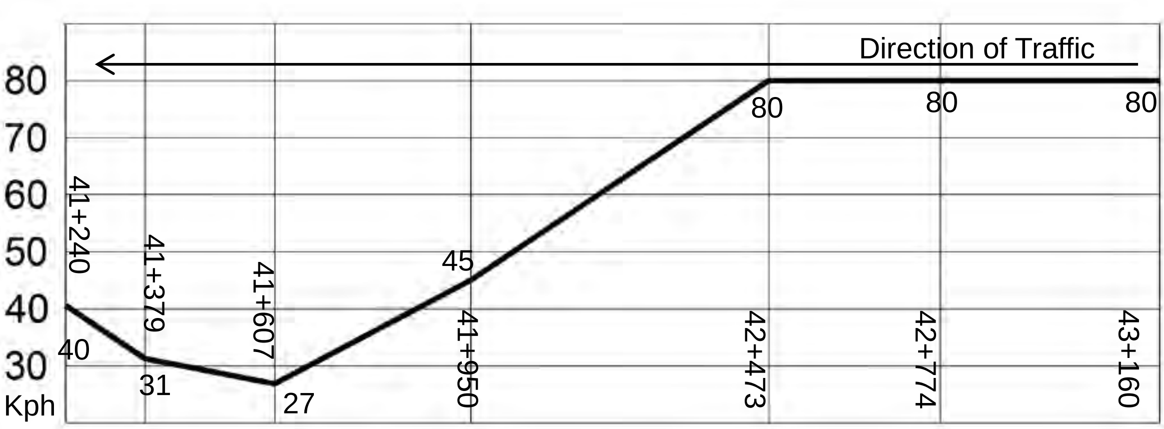
3) Impact on Traffic

Former Japan Highway Public Corporation provided a formula to understand variation of deceleration of loaded heavy traffic. According to the formula, impacts on traffic by the gradients are studied.



Source: JICA Study Team

Figure 5.2.2 Speed Variation 3.0%-Alternative A



Source: JICA Study Team

Figure 5.2.3 Speed Variation 4.5%-Alternative B

As seen at above figures, Speed of Alternative-B (4.5% vertical gradient) falls less than 30kph for longer than 0.21km.

4) Impact on Road Safety

In past many researchers and engineers have tried to capture the relationship between geometrical design parameters (vertical grade/slope) and accidents. The Study Team has found a document summarizing the relationship on the Road Grade and Safety, E. Hauer (2001) as summarized in the table below.

Table 5.2.6 Research Summary for Grade and Accident

Year/ Ref.	Method	Size	Accident modification functions	Acc. type	Conditions	Comments
Raff 1953	Crosstown Associates 1968	15 states	On tangent highway sections there does not appear to be any relation between grade and accident rates.	all	Sample from 15 states	Pooled data does not allow any inferences
Bitzel 1956	Crosstown Associates 1968	German Freeways	0%-2% 2%-4% 4%-6% 6%-8% 1.00 1.45 4.08 4.52	all		Out of line with other results. Most likely due to confounding with other variables
Mullins & Keese, 1961	Crosstown Associates 1968	54 miles of freeway , 10,000 Accidents	Accidents/MVM seem to vary as speed varies: diminish from sag towards crest and then increase towards sag.	All	Texas cities	
Hillier & Wardrop, 1966	C/S along one road	55 miles of motor-way	If curve B then 0% 0.5% 1% 1.5% 2.0% 1.00 1.02 1.09 1.19 1.39 AM	Injury	London to Birmingham straight sections	Differentiates between upgrade and downgrade lanes
Crosstown Associates 1968	C/S along freeway	Chicago Expressway	Accidents/MVM Upgrade 1.87 +/- 0.5% 1.10 Downgrade 2.49	All		
Vostrez & Lundy, 1964	C/S accident rate		Accidents/MVM 4-5% trucks 11% trucks Level 0.84 1.12 Up grade 0.71 1.51 Down grade 1.07 1.29	All	Straight freeway sections in California	When proportion of truck is large, upgrade accident rate grows
Cirillo 1969	C/S Multivariate regression	Interstate system	Grade was significant in one of two models. Annual accidents increase by 0.01 for each 1000 ADT for each 1% of grade	all		
Dunlap et al., 1978	Ohio and Pennsylvania turnpikes		Accident rate increases with downgrade and remains constant with upgrade. AMF is about 1.10/%grade	all		
Hedman, 1990	C/S	Swedish roads.	Study by Brude at al. AMF = 1.044/%grade	NA		
Li et al. 1994	C/S Multi-variate	163 sections 560 km	AMF($\Delta_{\%grade}$), 1+0.136 $\Delta_{\%grade}$/√ Accid/km Ex.: Increase of grade from2% to 3% When0.6accid/km, AMF=1+0.136/.0=1.23	Fat+Inj S EVD*		Model equation includes ADT in additive form. This is illogical.
Maaou 1995	C/S Multi-variate	11539 road sections 1985-92 6680 SV	0.019±0.009 for 1% All roads 0.960±0.015 for 1% Speed limit=55mph 0.892±0.013 for 1% Speed limit,55mph	Single veh, off-the road	Mainly rural two-lane but including HPMS 2.6,7,89 Non intersection accident	Lane width is not included in variables. Section length is variable with negative coefficient

Source:Road Grade and Safety ,E. Hauer. Draft1, April 17, 2001

Amongst these studies, several studies, including Bitzel (1956), found there is a relationship that the accident ratio increases as the vertical grade becomes steeper.

Table 5.2.7 Research Summary for Grade and Accident

Gradient (%)	Accident Rate (accidents/million vehicle km)
0.0-1.9	0.75
2.0-3.9	1.09
4.0-5.9	3.06
6.0-8.0	3.39

Source: Bitzel, I.F., (1956), Effect of motorway design on accidents in Germany. Highways and Bridges Engineering Works

There is an existing statistics and study document for road traffic accidents in Tanzania, prepared by SUMATRA and the statistics and study compiled number of accidents and its causes.

Table 5.2.8 Number of Road Traffic Accidents (2008 to 2012)

Causes of RoadTraffic Accidents	2008	2009	2010	2011	2012
Number of Accidents	20,615	22,739	24,665	23,986	23,578
Deaths	2,905	3,223	3,582	3,981	3,969
Injuries	16,308	17,861	19,263	20,802	20,111

Source: SUMATRA (2012) Annual Report YR 2011/12

Table 5.2.9 Major Causes of Road Traffic Accidents (2000 to 2005)

Causes of RoadTraffic Accidents	2000	2001	2002	2003	2004	2005	subtotal	(%)
Reckless / Dangerous Driving (Inappropriate speed)	7,041	6,743	8,179	10,916	9,366	4,318	46,563	54.5
Defective Motor Vehicles	2,797	2,440	2,641	2,503	2,403	1,164	13,948	16.3
Careless Pedestrians	850	1,056	1,096	1,463	1,337	566	6,368	7.5
Excessive Speed	426	350	340	376	1409	263	3,164	3.7
Careless Motor Cyclists	924	827	827	483	757	286	4,104	4.8
Careless Pedal Cyclists	1,276	891	891	367	607	359	4,391	5.1
Intoxication	170	98	99	68	171	92	698	0.8
Others i.e. Bad road, Slippery	1,064	1,472	1,417	488	989	768	6,198	7.3
Total	14,548	13,877	15,490	16,664	17,039	7,816	85,434	100

Source: Study on Road Accidents in Mainland Tanzania

The Study Team collected the traffic accident reports from the Traffic Police which occurred around existing Kikafu Bridge between January and December 2015.

Table 5.2.10 Traffic Accident data in/near Kikafu Bridge from Jan. to Dec. 2015

No.	DATE	TYPE OF ACCIDENT	TYPE OF VEHICLES INVOLVED	LOCATION	No of Injuries	Fatalities
1	11/01/2015	Salon Collided with motor-cycle.	T 130 BZR T/VEROSA T 650 BRK T. BETTER	Kikafu	2	-
2	09/02/2015	Head on Collision involving two trucks.	T 473 BZT/HILUX and T 321 CBR Scania, trela T 920 AYD	Mid span Kikafu bridge	-	1
3	20/03/2015	Salon Collided with a motor-cycle.	T 570 AJL Toyota Corolla and Motor-cycle T 811 BDT KINFAN	Mid span Kikafu bridge	-	5
4	26/04/2015	Speeding S/Wagon overturned.	DFP 3962 T/LAND CRUISER	Kikafu approach	5	-
5	18/05/2015	Overturned bus.	T 592 BGZ Hino Bus PSV	Kikafu	6	4
6	24/08/2015	Head on Collision involving a truck and P/up.	Toyota Hilux No. T 341 ANR and Truck T 541 ALN	Darajani	4	-
7	17/09/2015	Head on Collision involving two vehicles.	T 430 AQQ M/Fuso T 196 AAN T/Hiace	Kikafu	6	5
8	26/09/2015	Collision involving a Scania truck and three vans.	1. T 632 AJS Scania 2. T 192 BUD T/Hiace 3. T 440 BVA NOAH 4. T 162 BLH T/Hiace	Kikafu	3	-
9	08/10/2015	Tire burst causing vehicle to go astray	T.595 DCY, RAUMU	Kikafu	6	0
10	10/10/2015	Head on collision	T.990 ABH, FUSO T. 156 DKS T, CRESTA	Kikafu	0	1
11	08/11/2015	Head on collision	T.475 BMB, FUSO T.279 APA T, HILUX	Kikafu	4	0
12	11/11/2015	Bus Collided with motorcycle	T. 282 DFH, SCANIA	Kikafu	1	0

			BUS T. 440 BHR, TOYO			
13	25/11/2015	Head on collision	T. 286 BPF M, FUSO T. 280 AWC T, CHASER	Kikafu	0	1

Source: Traffic Police through Kilimanjaro Regional Office, TANROADS


September 26


October 2


October 3

Source: JICA Study Team

Figure 5.2.4 Photographs of Accident around Kikafu Bridges

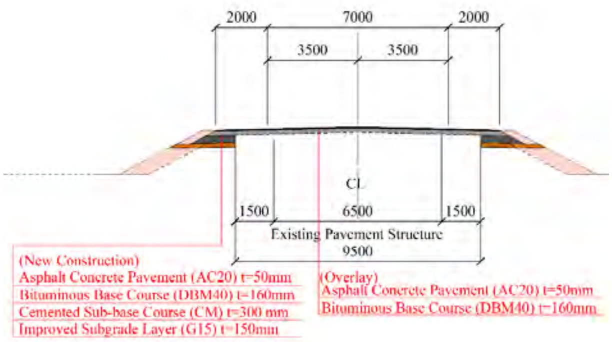
In addition to road geometry improvement, safety gears and facilities need to be considered in the improvement design.

5.2.7 Cross Section Design

(1) Section for Exiting Alignment Strengthening

1) Single Carriageway

F/S and D/D has proposed the geometrical designing to follow the existing parameters as much as possible in order to minimize construction cost. In other word, the existing pavement structure was designed to be used as roadbed. Overlays with 5cm of Asphalt Concrete (AC) and 16cm of Dense Bituminous Macadam (DBM) have therefore been designed as main component of the pavement works.

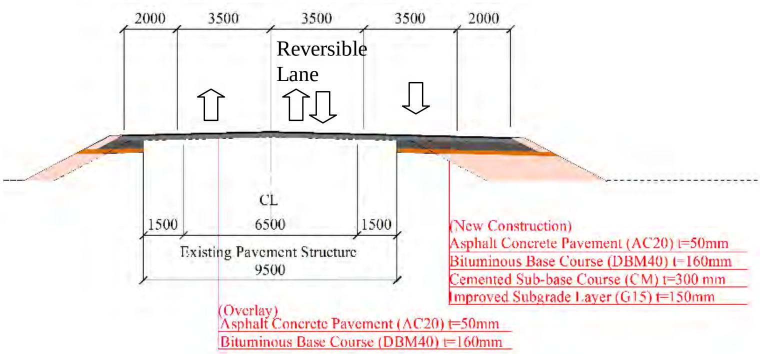


Source:F/S and D/D, drawn by JICA Study Team

Figure 5.2.5 Cross Section Design in F/S and D/D

Cross section design in the F/S and D/D is shown in Figure 5.2.5.

For the critical road segment where the traffic instantaneously exceeds the capacity, the arrangement of reversible lane may be considered.



Source: JICA Study Team

Figure 5.2.6 Reversible Lane Arrangement

The quantity and distribution of traffic volume are the primary parameters in determining the applicability and effectiveness potential of the reversible lane. Reversible operation is applicable to multi-lane roadways in which a directionally unbalanced traffic flow leaves one or more of the minor flow direction lanes underutilized and, in particular, segments with minimal turning and stopping manoeuvres.

Prior experience also shows that the reversible lane may be applicable for corridors that suffer from congestion but where, after study, it has been concluded that no other acceptable alternative improvement scenarios is available, including right-of-way limitations that preclude widening an existing facility.

Apart from the traffic aspect and the right of way constrain, the traffic control and road safety are another critical issues in determination. The application of the reversible lane is questionable if no organized traffic control system exists and no road users have awareness of road safety and keep discipline of driving rule

Close relationship between the traffic regulator, Police and road O&M agency, Road Administrator is also necessary in the operation as essential institutional framework. Taking into account the above, the applicability of the reversible lane is preliminary studied and its result is shown in Table 5.2.11.

Table 5.2.11 Reversible Lane Study Result

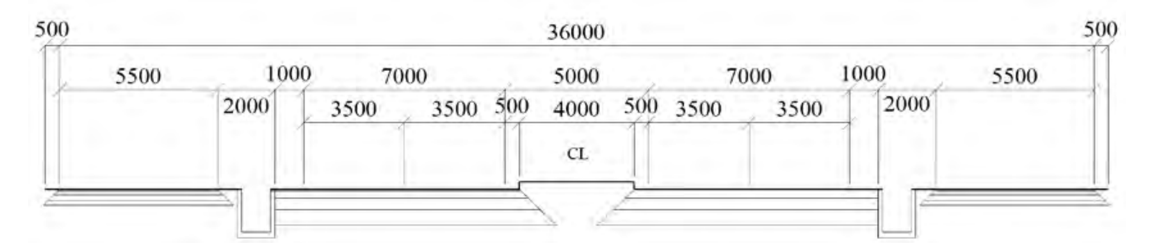
	Criteria	Applicability	Remarks
1	Traffic	Yes	Unbalanced traffic flow can be observed in peak hours around township.
2	Right of Way	No	Land is available for dualling
3	Traffic Control/ Safety	No	Consciousness for road safety by road users is not sufficient enough to follow reversible lane system.
4	Institutional Framework	No	Although discussions are regularly made between Traffic Police and TANROADS, no framework in managing traffic has been established.
5	Cost	No	Since the road is designed as single carriageway, provision of additional lane for merely increasing capacity at peak hours would not bring economical viability.
6	Overall Evaluation	Not Applicable	

Source: JICA Study Team

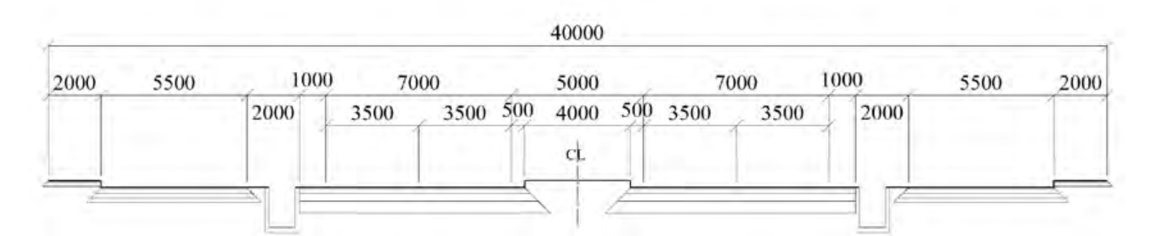
2) Dual Carriageway

As discussed earlier, the sections of Tengeru –Usa, Kikafu and Moshi shall be improved as dual carriageway road. Design principal of dualling is to accommodate within decided road reserve.

The Study recommends to apply the following cross section taking into account road safety and future urban developments.



Tengeru-Usa,
F/S and D/D design provided wide median as 9m -12m. In urban area in Tanzania, space for future BRT is considered in new road planning whose idea is adopted in the F/S and D/D design. The section between Tengeru and Usa is far away from Arusha Town Area which makes the BRT unreality even in future. The service road which would be combined usage with pedestrian walkway is reasonable considering road safety and future development. TANROADS recommends the drainage type to be u-shape which is appropriate to urban area and to avoid vehicle fall accident into the drainage.

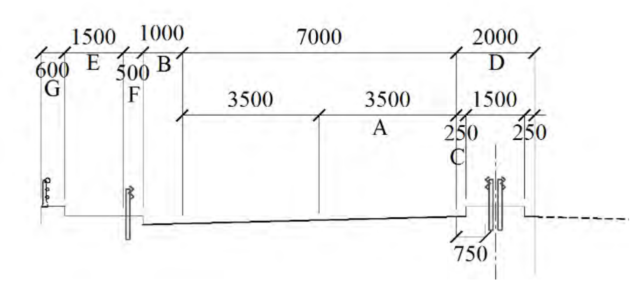


Moshi
This cross section design respects the F/S and D/D design. The design made dualling of carriageway and the rest of cross sectional elements remain the same in the F/S and D/D. The provisions of both service road and walkway may be reviewed at section where the space is critical.

3) Cross Section for New Kikafu Bridge

As the Kikafu River section consists of the approach and the bridge section, the bridge section is studied as shown in the following table.

Table 5.2.12 Cross Section Designs for New Kikafu River Section

Cross Section	Elements		Dim (m)	References
Bridge Section 	A	Traffic Lane	3.5	Tanzanian
	B	Outer Shoulder	1.0	STACC*2
	C	Inner edge strip	0.25	Economic Consideration (Door mirror width)*3
	D	Median	2.0	Accommodation of required facility width
	E	Walkway	1.5	Footway width at bridge from Tanzanian Manual
	F	Safety Buffer	0.5	Accommodation of required facility width
	G	Parapet	0.6	Required facility width

SATCC*1:9-12m of median according to the TZ Manual is considered to be too wide judged from economic and functional aspects for this project, the STACC recommends reasonable width of 5.0m taking into account accommodation of right turn lane at junction.
SATCC*2: STACC advices to consider provision of emergency vehicle passing space in addition to the allocated the traffic lanes in determination in shoulder width for critical road section and this shoulder 1.0m results in 8.25m for available vehicle passage space which can accommodate for 3 vehicles to pass in parallel in emergency case.
Door mirror width*3: Referred from Japanese Standards
Source: JICA Study Team

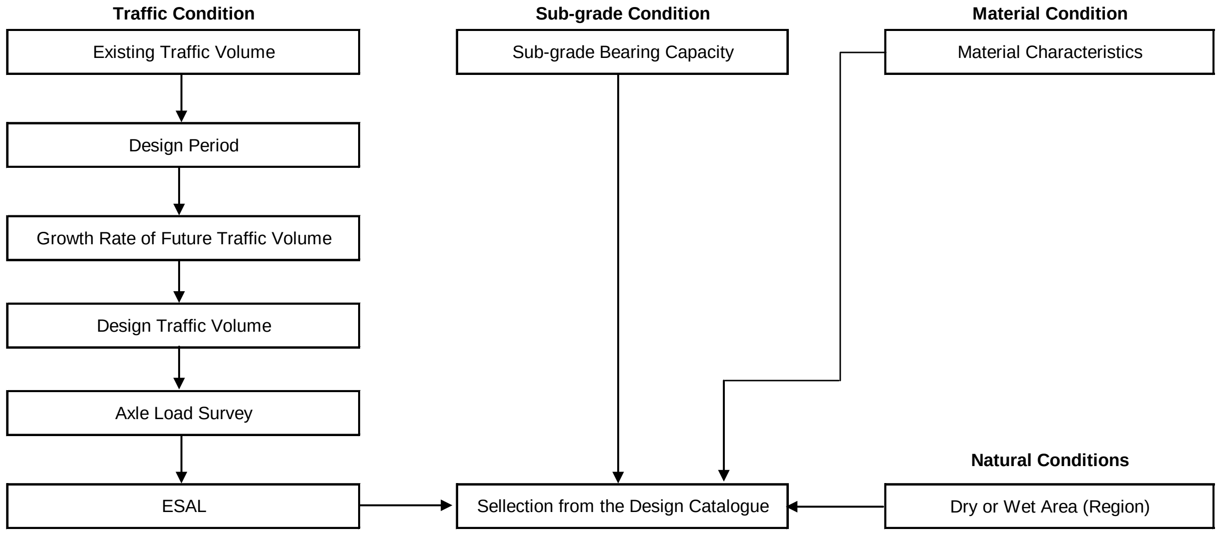
5.2.8 Pavement Design

(1) Design Standard

Ministry of Works, Transport and Communications, Tanzania has prepared "Pavement and Material Design Manual 1999" that has been applied for the road projects in the country. The manual covers pavement designs for both new construction and rehabilitation. The manual prepared the design catalogue in designing for new pavement construction that is the matrix of CBR and traffic. The catalogue is however applicable below TLC 50 which is 20 to 50 E80 x 10^6. In normal practice, AASHTO or other international standard are applied for the traffic that is greater than TLC50 in the country.

(2) Design Procedure

The SATCC gives the design procedure as follows.



Source: SATCC

Figure 5.2.7 Pavement Design Procedure

(3) Pavement Design Life

The design life of a road can have a large impact on the design specifications of its pavement structure. Hence, it is important to decide on an appropriate period. Usually, a 10-, 15-, or 20-year period is adopted, with the selection of an appropriate design life being dependent on the unique circumstances of the individual project. The following table from the SATCC standard provides some guidance on the selection.

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

In the case of a selection of a “long” design life, the initial capital cost may be large in order for the road to be able to sustain the forecasted cumulative traffic loading over the 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 sought. 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.

TANROADS is of opinion that design life shall be selected as longer as possible because the traffic increases drastically in recent years which is sometime larger than a result of traffic forecast.

F/S and D/D described that the design life was decided as 20 years thus shall be maintained in this study.

(4) Traffic Factor

1) Traffic Load Classes (TLC) in Tanzania

As earlier manual prepares Traffic Load Classes (TLC) according to design traffic loading ranges.

Table 5.2.13 Traffic Load Classes in Tanzania

Design Traffic Loading [E80x10^6]	Traffic Load Class (TLC)	*Traffic Load Class (TLC-H)
<0.2	TLC 02	-
0.2 to 0.5	TLC 05	TLC05-H
0.5 to 1	TLC 1	TLC1-H
1 to 3	TLC 3	TLC3-H
3 to 10	TLC 10	TLC10-H
10 to 20	TLC 20	TLC20-H
20 to 50	TLC 50	TLC50-H

Source: Pavement and Material Design Manual 1999

*Where the heavy (>13t) axles' proportion of E80 is 50% or higher the Traffic Load Class shall be given an index,

2) Equivalent Factors (EF)

The Manual prepares heavy vehicle category as shown in the following table and traffic loading data shall be processed by the classes.

Table 5.2.14 Heavy Vehicle Category in Tanzania

Heavy Vehicle Category	Definition
Medium Goods Vehicle MGV	-2 axles, incl steering axle, and -3 tonnes empty weight, or more
Heavy Goods Vehicle HGV	-3 axles, incl steering axle, and -3 tonnes empty weight, or more
Very Heavy Goods Vehicle VHGV	-4 or more axles, incl steering axle, and -3 tonnes empty weight, or more
Buses	Seating capacity of 40, or more

Source: Pavement and Material Design Manual 1999

"Traffic Survey and Analysis Report" of the F/S and D/D describes the Adopted EF values which were obtained at the road side traffic loading surveys with portable weigh bridge.

Table 5.2.15 Adopted EF Values in F/S and D/D

STATION: NGARAMTONI	DIRECTION	BUS	LGV	MGV	HGV	Art. Trucks
Average	Nairobi bound	0.688	0.032	1.656	2.517	2.913
	Arusha bound	0.302	0.006	4.764	1.509	2.589
Maximum	Nairobi bound	4.394	0.219	11.296	6.741	7.079
	Arusha bound	1.113	0.012	72.820	5.055	11.489
STATION: MWIKA	**DIRECTION**	**BUS**	**LGV**	**MGV**	**HGV**	**Art. Trucks**
Average	Voi bound	0.344	0.032	0.909	1.581	0.369
	Arusha bound	0.227	0.023	0.668	0.933	2.675
Maximum	Voi bound	0.892	0.305	2.178	3.048	0.464
	Arusha bound	1.038	0.109	1.545	3.656	5.161
STATION: MWATATE	**DIRECTION**	**BUS**	**LGV**	**MGV**	**HGV**	**Art. Trucks**
Average	Mwatate bound	0.027	0.121	0.663	0.386	0.760
	Wundanyi bound	0.227	0.689	1.071	1.534	7.907
Maximum	Mwatate bound	0.053	1.820	1.766	0.452	3.921
	Wundanyi bound	0.554	0.843	4.471	3.820	44.606
STATION: VOI	**DIRECTION**	**BUS**	**LGV**	**MGV**	**HGV**	**Art. Trucks**
Average	Arusha bound	0.227	0.006	0.593	1.534	7.907
	Voi bound	0.031	0.007	1.265	0.386	0.760
Maximum	Arusha bound	0.554	0.014	2.962	3.820	44.606
	Voi bound	0.124	0.016	8.133	0.452	3.921

Source: F/S and D/D

According to the above table, average value of articulated truck, for example, is very small as around 3.0 and it calls for suspicion in either surveying method or data processing.

TANROADS Kilimanjaro provided the traffic loading data from January to March and some other months in 2014. Numbers of samples are 67,957 for MGV, 12,829 for HGV and 26,207 for VHGV which are processed for the estimation of EF using the following formula,

$$\text{Equivalent Factor} = [\text{Axle Load (kg)} / 8160]^{4.5}$$

As results of the data processing, the EF by the heavy vehicle categories are obtained as follows,

- MGV=3.88
- HGV=3.26
- VHGV=8.83

3) Estimation of Design Traffic Loading (ESAL)

Average Daily Traffic and Growth Rate

Average daily traffic and growth rates are calculated based on a result of traffic forecast and the scenario that has been applied was with the project case.

The project is expected to be commenced by 2019 and the base traffics as at 2019 are estimated according to traffic growths as follows.[1]

[1] Grouped traffics were estimated by average of individual link traffic data in the group.

Table 5.2.16 Estimation of Traffic Growth by Heavy Vehicle Category

1.Traffic Growth		2015			2035			2035/2015		
Link	Section	MGV	HGV	VHG	MGV	HGV	VHGV	MGV	HGV	VHGV
515	TENGERU-USA WEST	1,025	401	829	3,119	1,297	2,665	3.04	3.24	3.21
512	USA WEST-USA EAST	957	471	726	2,885	1,522	2,327	3.02	3.23	3.21
510	USA EAST-KIA JCT (KILIMANJARO / ARUSHA BDR)	642	385	694	1,935	1,244	2,224	3.02	3.23	3.21
505	KIA JCT (KILIMANJARO / ARUSHA BDR)-BOMANGOMBE	554	222	622	1,672	718	1,995	3.02	3.23	3.21
500	BOMANGOMBE-KWASADALA	688	365	684	2,075	1,181	2,197	3.02	3.23	3.21
495	KWASADALA-KMT (KILIMANJARO MACH. TOOLS)	688	365	684	2,075	1,181	2,197	3.02	3.23	3.21
490	KMT (KILIMANJARO MACH. TOOLS)-KIBOSHO RD JCT	822	508	747	2,479	1,643	2,398	3.02	3.23	3.21
485	KIBOSHO RD JCT-MOSHI	495	285	475	1,494	921	1,523	3.02	3.23	3.21
480	MOSHI-KIBORILONI	1,008	362	453	2,828	1,188	1,513	2.80	3.28	3.34
475	KIBORILONI-SANGO	1,439	232	471	4,034	761	1,571	2.80	3.28	3.34
470	SANGO-KAWAWA	1,439	232	471	4,034	761	1,571	2.80	3.28	3.34
465	KAWAWA-UCHIRA	1,439	232	471	4,034	761	1,571	2.80	3.28	3.34
460	UCHIRA-HIMO JCT	1,439	232	471	4,034	761	1,571	2.80	3.28	3.34
2220	HIMO JCT-HIMO	820	115	138	2,139	309	372	2.61	2.69	2.69
2225	HIMO-KILACHA	46	11	55	132	35	162	2.90	3.11	2.95
2230	KILACHA - TARAKEA JCT	46	11	55	132	35	162	2.90	3.11	2.95
2233	TARAKEA JCT - HOLILI CUSTOMS (TANZANIA/KENYA BDR)	33	6	24	96	18	71	2.90	3.11	2.95
520		543	228	317	1,653	737	1,019	3.04	3.24	3.21
							Average	2.92	3.20	3.17
							Growth Rate	1.058	1.063	1.063
							VEF	3.885	3.262	8.831

Source: JICA Study Team

Table 5.2.17 Estimation of Average Daily Traffic in 2019

Design Section Traffic

1.Traffic Growth		2015			2019		
Link	Section	MGV	HGV	VHGV	MGV	HGV	VHGV
515	TENGERU-USA WEST	991	436	777	1,241	557	991
512	USA WEST-USA EAST						
510	USA EAST-KIA JCT (KILIMANJARO / ARUSHA BDR)	648	355	651	812	454	830
505	KIA JCT (KILIMANJARO / ARUSHA BDR)-BOMANGOMBE						
500	BOMANGOMBE-KWASADALA						
495	KWASADALA-KMT (KILIMANJARO MACH. TOOLS)						
490	KMT (KILIMANJARO MACH. TOOLS)-KIBOSHO RD JCT						
485	KIBOSHO RD JCT-MOSHI						
480	MOSHI-KIBORILONI	1,264	234	412	1,584	299	526
475	KIBORILONI-SANGO						
470	SANGO-KAWAWA						
465	KAWAWA-UCHIRA						
460	UCHIRA-HIMO JCT						
2220	HIMO JCT-HIMO						
2225	HIMO-KILACHA	41	10	45	52	12	57
2230	KILACHA - TARAKEA JCT						
2233	TARAKEA JCT - HOLILI CUSTOMS (TANZANIA/KENYA BDR)						
520							

Source: JICA Study Team

The required structural number (SN) in pavement design for KIA road shall exceptionally be estimated as a result of back calculation from the pavement composition design in the F/S and D/D since the traffic demand forecast for KIA access road in the Study is not comprehensive, without detailed analysis on future passenger and cargo demand generated to/from KIA.

The Cumulative Design Traffic is estimated by the following formula,

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

where:

DT is the cumulative design traffic in a vehicle category, for one direction, and
T = average daily traffic in a vehicle category in the first year (one direction)
r = average assumed growth rate, per cent per annum
p = design period in years (20 years)

The Manual specifies lane distribution factors as shown in the following table.

Table 5.2.18 Lane Distribution Factor

Road Type	Design Traffic Loading	Comments
Single Carriageway		
Paved road width 4.5m or less	Up to twice the sum of the ESALs in each direction* (2.0)	As a minimum, total traffic must be designed for since there will be significant overlap in each direction For width of 3.5m or less, double the total should be used due to channelization
Paved road width 4.5m to 6.0m	80% of the sum of the ESALs in each direction (1.8)	To allow for considerable overlap in the central section of the road
Paved road with more than 6.0m	Total ESALs in the most heavily trafficked direction	No overlap effectively, vehicles remaining in lanes
Dual Carriageway		
Less than 2,000 commercial vehicles per day in one direction	90% of the ESALs in direction (0.9)	The majority of heavy vehicles will travel in one lane effectively
More than 2,000 commercial vehicles per day in one direction	80% of the ESALs in the direction (0.8)	The majority of heavy vehicles will still travel in one lane effectively, but greater congestion leads to more switching
*Judicious to use double the total ESALs expected, as normally these are low trafficked roads and this may lead to difference in pavement structure		

Source: Pavement and Material Design Manual 1999

<u>Estimation of Design Traffic Load</u>

Applying the above traffic factor, data and formulae, the design traffic loadings are obtained as shown in the following tables.

Table 5.2.19 Estimations of Design Traffic Loadings

Tengeru | Usa East | Dual Carriageway

A	B	C	D	E	F	G	H	I	J
Type of Vehicle	Daily Traffic Volume in a First Year	Directional Distribution Factor	Daily Traffic Volume for One Direction	Growth Rate	Design Year	Cumulative Design Traffic	VEF	Lane Distribution Factor	ESAL
	(Nos.)	(%)	(Nos.)	(%)	(Year)	(Nos.)			
MGV	1,241	50	621	5.8	20	8,160,942	3.88	0.8	25,362,303
HGV	557	50	278	6.3	20	3,855,274	3.26	0.8	12,576,615
VHGV	991	50	496	6.3	20	6,878,475	8.83	0.8	60,740,764
Total	2,789		1,395			18,894,691		Total	98.7E+6

Usa East | Moshi | Single Carriageway

A	B	C	D	E	F	G	H	I	J
Type of Vehicle	Daily Traffic Volume in a First Year	Directional Distribution Factor	Daily Traffic Volume for One Direction	Growth Rate	Design Year	Cumulative Design Traffic	VEF	Lane Distribution Factor	ESAL
	(Nos.)	(%)	(Nos.)	(%)	(Year)	(Nos.)			
MGV	812	50	406	5.8	20	5,335,495	3.88	1	20,726,842
HGV	454	50	227	6.3	20	3,148,012	3.26	1	10,269,394
VHGV	830	50	415	6.3	20	5,755,175	8.83	1	50,821,405
Total	2,096		1,048			14,238,682		Total	81.8E+6

Usa East | Moshi | Dual Carriageway

A	B	C	D	E	F	G	H	I	J
Type of Vehicle	Daily Traffic Volume in a First Year	Directional Distribution Factor	Daily Traffic Volume for One Direction	Growth Rate	Design Year	Cumulative Design Traffic	VEF	Lane Distribution Factor	ESAL
	(Nos.)	(%)	(Nos.)	(%)	(Year)	(Nos.)			
MGV	812	50	406	5.8	20	5,335,495	3.88	0.8	16,581,474
HGV	454	50	227	6.3	20	3,148,012	3.26	0.8	10,269,394
VHGV	830	50	415	6.3	20	5,755,175	8.83	0.8	50,821,405
Total	2,096		1,048			14,238,682		Total	77.7E+6

Moshi Himo Single Carriageway

A	B	C	D	E	F	G	H	I	J
Type of Vehicle	Daily Traffic Volume in a First Year	Directional Distribution Factor	Daily Traffic Volume for One Direction	Growth Rate	Design Year	Cumulative Design Traffic	VEF	Lane Distribution Factor	ESAL
	(Nos.)	(%)	(Nos.)	(%)	(Year)	(Nos.)			
MGV	1,584	50	792	5.8	20	10,408,158	3.88	1	40,432,657
HGV	299	50	150	6.3	20	2,080,184	3.26	1	6,785,943
VHGV	526	50	263	6.3	20	3,647,256	8.83	1	32,207,300
Total	2,408		1,205			16,135,598		Total	79.4E+6

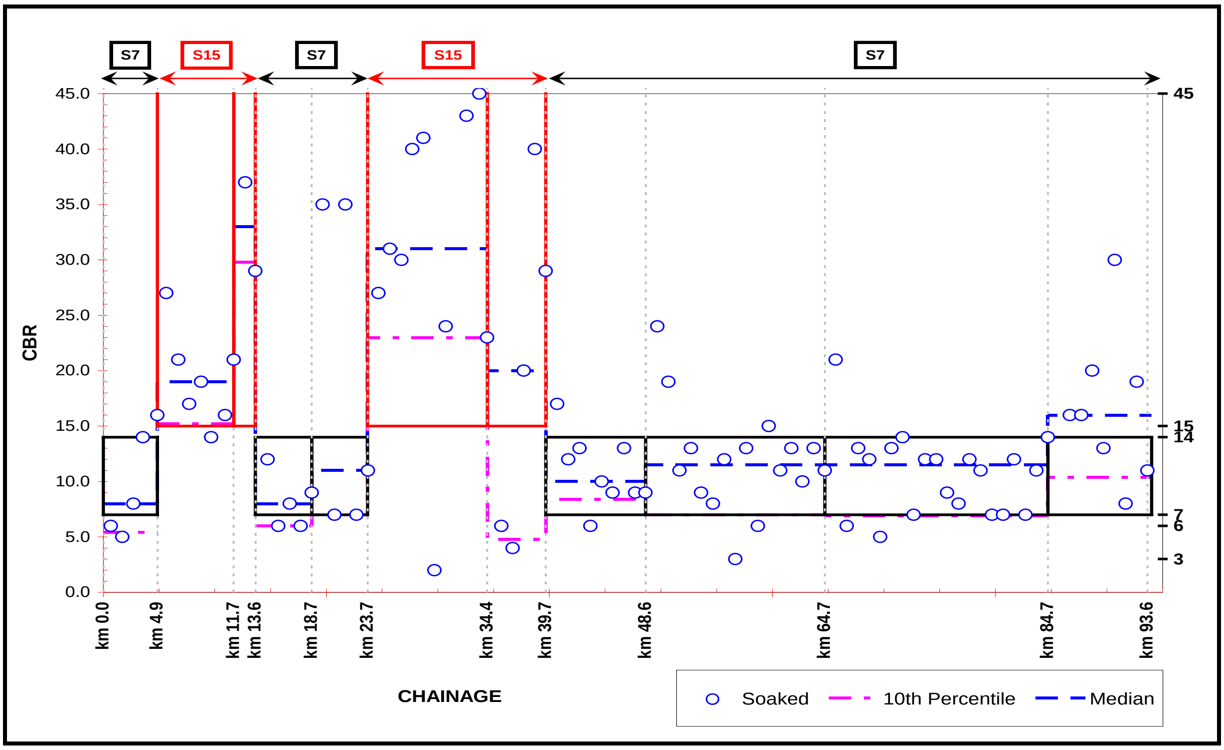
Himo Holili Single Carriageway

A	B	C	D	E	F	G	H	I	J
Type of Vehicle	Daily Traffic Volume in a First Year	Directional Distribution Factor	Daily Traffic Volume for One Direction	Growth Rate	Design Year	Cumulative Design Traffic	VEF	Lane Distribution Factor	ESAL
	(Nos.)	(%)	(Nos.)	(%)	(Year)	(Nos.)			
MGV	52	50	26	5.8	20	341,682	3.88	1	1,327,335
HGV	12	50	6	6.3	20	83,207	3.26	1	271,438
VHGV	57	50	28	6.3	20	388,301	8.83	1	3,428,914
Total	121		60			813,190		Total	5.0E+6

Source: JICA Study Team

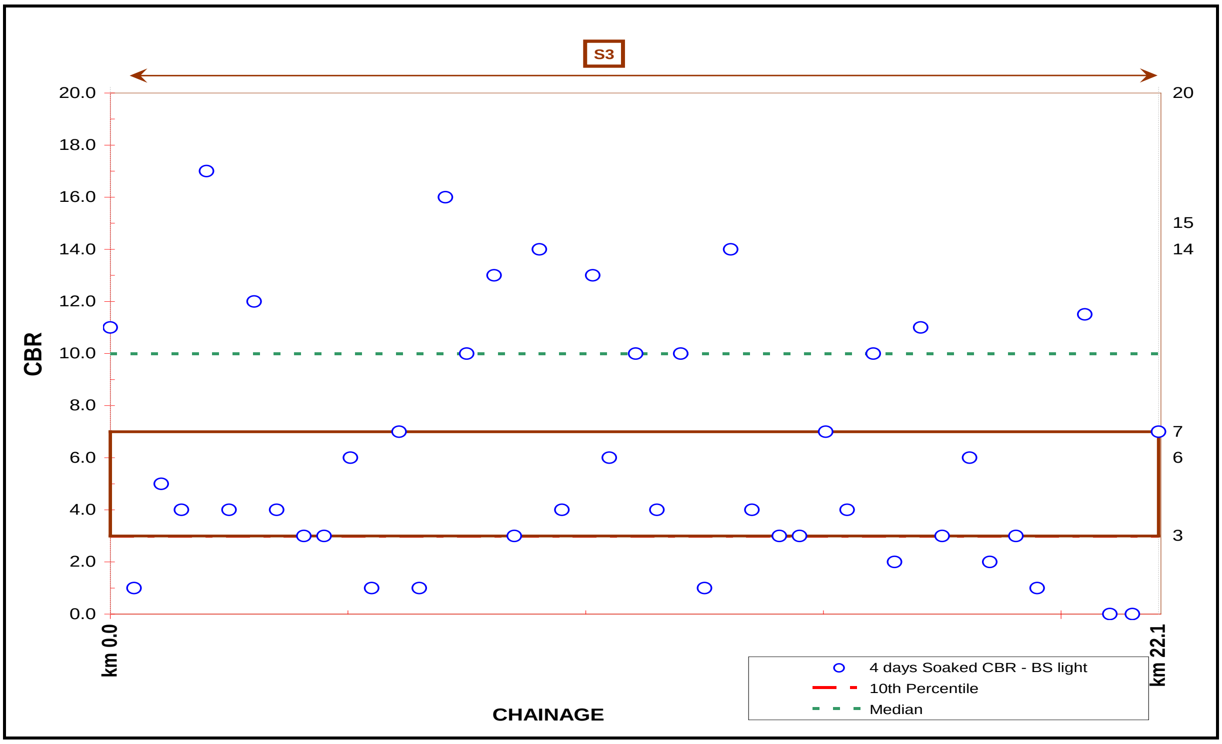
(5) Subgrade Condition

"Pavement and Material Report" of the F/S and D/D prepares design CBR by sections based on the soaked CBR Test results.



Source: F/S and D/D

Figure 5.2.8 Design CBR (Subgrade Condition) for Usa East to Holili



Source: F/S and D/D

Figure 5.2.9 Design CBR (Subgrade Condition) for Arusha-Usa East

(6) Pavement Composition

The manual specifies a pavement design matrix combining the Traffic Class and the Subgrade Class with design pavement configuration. However, it covers up to TLC50 (20-50x10^6 ESALs) of the design traffic while the design traffic for the project road was estimated to be 5 to 98.7x10^6 in ESALs.

The SATCC suggests that AASHTO or UK ‘s pavement design method shall be employed in case the design traffic exceeds the range of TLC50. The AASHTO method (AASHTO Guide for Design Pavement Structures 1993) was thus employed for this pavement configuration design.

The AASHTO Guide gives the basic design equation for flexible pavement (asphalt pavement) as follows,

$$\log_{10}(W18) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

where

W_{18}: predicted number of 18-kip equivalent single axle load (ESAL/ESAs) applications,
Z_R: standard normal deviate corresponding to level of reliability,
S_0: combined standard error of traffic prediction and performance prediction
△PSI: difference between the initial design serviceability index, p_0, and the design terminal serviceability index p_1, and,
M_R: resilient modulus of roadbed soil (psi)

SN is equal to the structural number indicative of the total pavement thickness required:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

where

a_i: i th layer coefficient,
D_i: i^{th} layer thickness (inches), and
m_i: i^{th} layer drainage coefficient.

1) Reliability

Reliability is a means of incorporating some degree of certainty into the design process to ensure that the various design alternatives will last for the analysis period. The reliability design factor accounts for chance variations in both traffic prediction (w18) and the performance prediction (W18), and therefore provides a predetermined level of assurance (R) that pavement sections will last for the period for which they were designed.

The 1993 AASHTO Guide provided some recommended level as shown in Table 5.2.20.

Table 5.2.20 Recommended Level of Reliability

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Source: 1993 AASHTO Guide

The AASHTO Guide gives the standard normal deviate (ZR) value corresponding to selected level of reliability as shown in the following table and suggests the combined standard error of the traffic prediction and performance prediction (S0) of 0.45 for flexible pavement based on the AASHTO Road test.

Table 5.2.21 Standard Normal Deviate (Z_R) Value Corresponding to Selected Levels of Reliability

Reliability R(%)	Standard Normal Deviate Z_R	Reliability R(%)	Standard Normal Deviate Z_R	Reliability R(%)	Standard Normal Deviate Z_R
50	0	90	-1.282	96	-1.751
60	-0.253	91	-1.340	97	-1.881
70	-0.524	92	-1.405	98	-2.054
75	-0.674	93	-1.476	99	-2.337
80	-0.841	94	-1.555	99.9	-3.090
85	-1.037	95	-1.645	99.99	-3.750

Source: AASHTO Guide

2) Performance Criteria

Pavement performance is represented by the serviceability history of a pavement. The serviceability of pavement is defined as its ability to serve the type of traffic (automobiles and trucks) which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 (impossible road) to 5 (perfect road).

Selection of lowest allowable PSI i.e. terminal serviceable index (p_t) is based on the lowest index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. An index of 2.5 or higher is suggested by the AASHTO Guide for design of major highways and 2.0 for highways with lesser traffic volumes.

The initial serviceability index (p_0) is the value immediately after construction. p_0 observed at the AASHTO Road Test was 4.2 for flexible pavement.

The total change in serviceability index is expressed by $\triangle PSI = p_0 - p_t$

3) Roadbed Soil

The resilient modulus (MR) is used to characterize soil property. The AASHTO Guide introduces the equation estimating MR from CBR or R –value, as follows:

MR(psi)=1,500 x CBR

4) Structural Layer Coefficient

The pavement strength is expressed by the structural number (SN) which is indicative of the total pavement thickness required. In order to convert actual layer thickness into SN, the structural layer coefficients (ai) are required.

Referring to the Manual, the following coefficients were applied as follows,

i. Asphalt concrete surface course:a_1=0.4
ii. Bituminous Treated Base Course(eg.DBM) a_2=0.3
iii. Cemented Subbase Course a_3=0.11
iv. Improved Subgrade a_4=0.09

5) Drainage Coefficient

The structural layer coefficients were modified considering the efforts of a certain level of drainage on predicted pavement performance. The factor for modifying the structural layer coefficient is referred to as mi value. The recommended mi values are given in the AASHTO Guideline as shown in the following table.

Table 5.2.22 Recommended mi values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements

Quality of Drainage	Percent of Time Pavement Structure is exposed to Moisture Levels approaching Saturation			
	Less than 1%	1-5%	5-25%	Greater than 25%
Excellent (water removed within 2hrs)	1.25-1.20	1.35-1.30	1.30-1.20	1.20
Good (water removed within 1day)	1.20-1.15	1.25-1.15	1.15-1.00	1.00
Fair (water removed within 1 week)	1.15-1.10	1.15-1.05	1.00-0.80	0.80
Poor (water removed within 1 month)	1.10-1.00	1.05-0.80	0.80-0.60	0.60
Very Poor (water will not drain)	1.00-0.90	0.95-0.75	0.75-0.40	0.40

Source AASHTO Guide

6) Design Output

The pavement structural pavement designs are carried out employing AASHTO method and the design CBRs have been decided according to Figure 5.2.8 and Figure 5.2.9.

Table 5.2.23 Structural Pavement Design Outputs

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT (Design CBR 3%) (AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	98,680,000

CBR (%)	3.0	kg/cm^3
Resilient Modulus (MR)	4,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (∆PSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	7.394	

CHECK EQUATION : 7.994 = 7.994

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	0.00	0.00
AC Base (ATB)	0.3	9.84	2.95
Cemented CM	0.11	15.75	1.73
Aggregate class G15	0.09	21.65	1.95
			7.421

7.394

Tengeru –Usa Dual Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT (Design CBR 7%) (AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	81,817,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (∆PSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.595	

CHECK EQUATION : 7.913 = 7.913

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	0.00	0.00
AC Base (ATB)	0.3	6.30	1.89
Cemented CM	0.11	13.78	1.52
Aggregate class G15	0.09	15.75	1.42
			5.610

5.595

Usa East-Moshi (Except Kifafu) Single Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	77,672,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.555	

CHECK EQUATION : 7.890 = 7.890

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	0.00	0.00
AC Base (ATB)	0.3	6.30	1.89
Cemented CM	0.11	11.81	1.30
Aggregate class G15	0.09	17.72	1.59
			5.571

5.555

Usa East-Moshi (Except Kifafu) Dual Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR15%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	77,672,000

CBR (%)	15.0	kg/cm^3
Resilient Modulus (MR)	22,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	4.304	

CHECK EQUATION : 7.890 = 7.890

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	0.00	0.00
AC Base (ATB)	0.3	6.30	1.89
Cemented CM	0.11	7.87	0.87
Aggregate class G15	0.09	9.84	0.89
			4.429

4.304

Kikafu Dual Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	79,425,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.572	

CHECK EQUATION : 7.900 = 7.900

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	0.00	0.00	
AC Base (ATB)	0.3	6.30	1.89	
Cemented CM	0.11	11.81	1.30	
Aggregate class G15	0.09	19.69	1.77	
			5.748	5.572

Moshi- Himo Single Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	5,028,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	3.694	

CHECK EQUATION : 6.701 = 6.701

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	0.00	0.00	
AC Base (ATB)	0.3	3.94	1.18	
Cemented CM	0.11	7.87	0.87	
Aggregate class G15	0.09	9.84	0.89	
			3.720	3.694

Himo- Holili Single Carriageway

Source: JICA Study Team

According to the material report of the F/S and D/D, material property from borrow pits indicated high plastic index(PI) which results that cement is not applicable as an agent for the stabilization. Lime shall be applicable as an alternative agent.

Instead of the stabilization, the application of base course materials (i.e. CRR or CRS) is the optimum option at the lower part of pavement structure of which expectation is to perform more steadily compared to the stabilized material. It however calls higher construction cost because of small quantities available along the project road.

In order to seek an economical option, recycling of the existing base course material with stabilization shall be further studied in the detailed design stage.

5.2.9 Pavement Design Improvement Study

Stabilizing materials with cement is an attractive option to allow less suitable materials to be used as base course or subbase. This is especially the case when good quality natural materials are not available or scarce. The problem with cement treated materials however is that they are brittle in nature and are sensitive to overloading.

There were some observations that pavement structure failures have occurred on the road sections where the stabilized base/subbase course have been applied

Against the backdrop of the pavement failure cases and its ongoing analysis by JICA, the Study recommends not to apply stabilization in construction of base/subbase course.

This section describes the pavement design improvement referring to the latest edition of pavement design documents, fieldwork trends and the opinion of TANROADS.

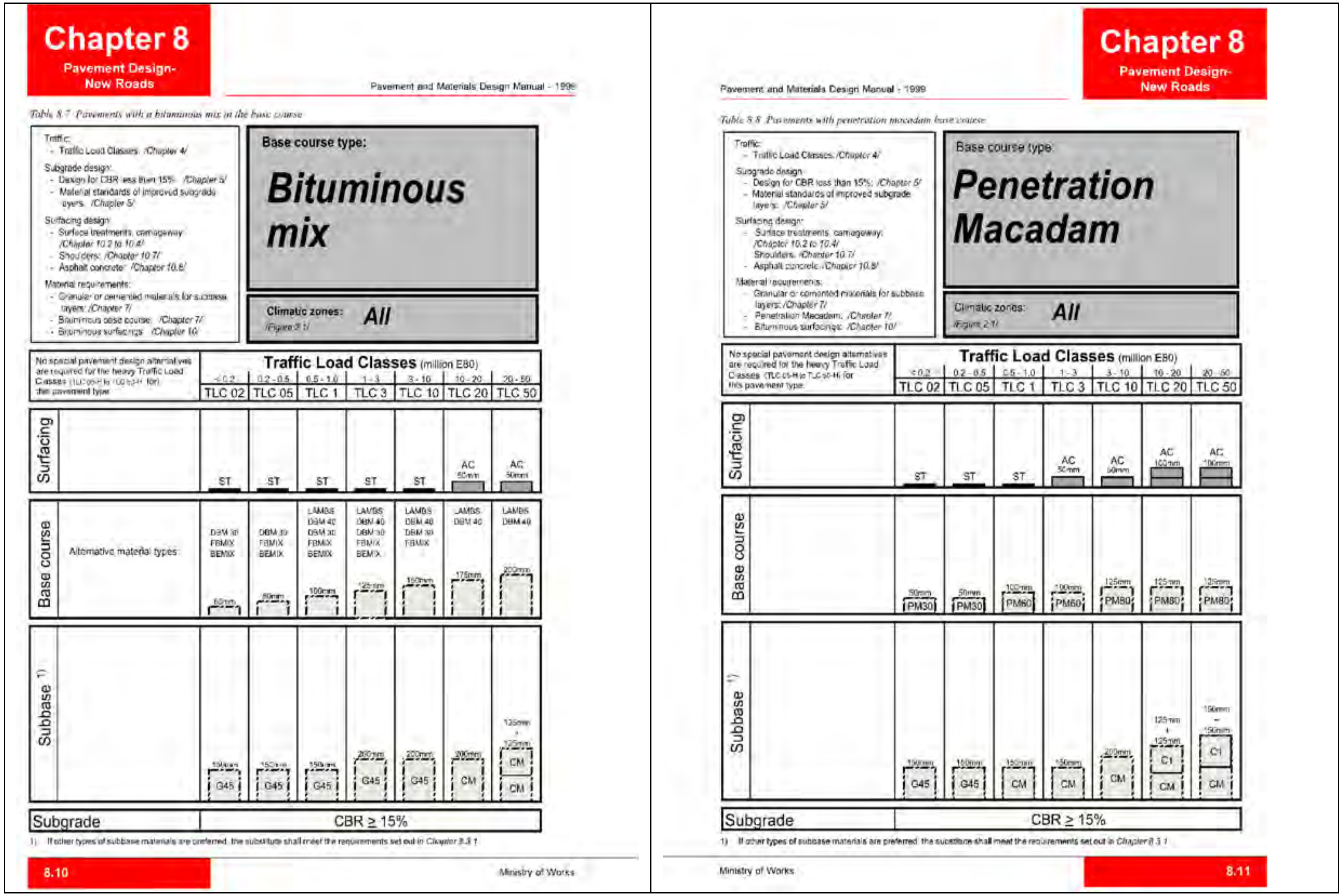
(1) Improvement of Pavement Composition

As explained earlier, the EAC tries to make harmonization in its highway designing. In order to standardize the design parameters, some studies have been carried out. The design guideline has been prepared in the designing of international truck roads as a result of the studies.

As for the pavement materials, the following recommendation can be found in the guideline,

"The study recommends codes for unbound, cemented and bituminous materials for EAC Partner States. It is worthy pointing out that in the UK, for instance, Dense Bitumen Macadam (DBM) can be used for roads carrying up to 80 msa, whereas a DBM thickness design for traffic loads in excess of 80 msa must include a 125 mm thick lower road base layer of Hot Rolled Asphalt (HRA), with DBM or Dense Tar Macadam (DTM) as the reminder of the road base material".

Application of bituminous base course is a common practice in the developing countries as the Tanzania Pavement Manual also states it.



Source: Pavement and Materials Design Manual

Notable point in the guideline is that it suggested the minimum thickness of the bituminous material of 125mm which needs to be considered in the improvement pavement study.

Surface a course and a binder course layers are, in general, applied for the road section where large heavy traffic number is estimated. This is because of a benefit of less maintenance as the surface course solely be refreshed when fatigue and damage occur on pavement. As a result of review of the design traffic loading explained in 5.2.8(4), the ESALs were re-calculated as greater than $50x10^6$ for all of the study sections which have not been covered in the Tanzanian Pavement Manuals. These ESALs were thus as extremely large as the two layers required in designing.

Taking into account the above, the pavement composition design was improved by the design segments.

Table 5.2.24 Structural Pavement Design Improvement Outputs

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 3%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	98,680,000

CBR (%)	3.0	kg/cm^3
Resilient Modulus (MR)	4,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	7.394	

CHECK EQUATION : 7.994 = 7.994

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	1.97	0.79	
AC Base (ATB)	0.3	5.91	1.77	
CRS	0.14	13.78	1.93	
Aggregate class G15	0.09	23.62	2.13	
			7.402	7.394

Tengeru –Usa Dual Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	81,817,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.595	

CHECK EQUATION : 7.913 = 7.913

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	1.97	0.79	
AC Base (ATB)	0.3	5.91	1.77	
CRS	0.14	9.84	1.38	
Aggregate class G15	0.09	11.81	1.06	
			5.787	5.595

Usa -Moshi (Except Kifafu) Single Carriageway S7

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	77,672,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.555	

CHECK EQUATION : 7.890 = 7.890

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	1.97	0.79
AC Base (ATB)	0.3	5.91	1.77
CRS	0.14	9.84	1.38
Aggregate class G15	0.09	11.81	1.06
			5.787

5.555

Usa East-Moshi (Except Kifafu) Dual Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR15%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	0.8

Total ESAL for 20 years	77,672,000

CBR (%)	15.0	kg/cm^3
Resilient Modulus (MR)	22,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	4.304	

CHECK EQUATION : 7.890 = 7.890

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number
AC Wearing Course	0.4	1.97	0.79
AC Binder Course	0.4	1.97	0.79
AC Base (ATB)	0.3	4.92	1.48
CRS	0.14	5.91	0.83
Aggregate class G15	0.09	5.91	0.53
			4.409

4.304

Kikafu Dual Carriageway S15

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	79,425,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	5.572	

CHECK EQUATION : 7.900 = 7.900

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	1.97	0.79	
AC Base (ATB)	0.3	4.92	1.48	
CRS	0.14	9.84	1.38	
Aggregate class G15	0.09	15.75	1.42	
			5.846	5.572

Moshi-Himo Single Carriageway

PAVEMENT THICKNESS DESIGN - FLEXIBLE PAVEMENT
(Design CBR 7%)
(AASHTO)

DATA TRAFFIC :

Design Life (Year)	20
Distribution Factor	0.5
Lane Coefficeint	1

Total ESAL for 20 years	5,028,000

CBR (%)	7.0	kg/cm^3
Resilient Modulus (MR)	10,500	psi
SERVICEABILITY :		
- Terminal Serviceability (Pt)	2.50	
- Initial Serviceability (Po)	4.20	
- Serviceability Loss (ΔPSI)	1.70	
RELIABILITY, R (%)	80.00	
STANDARD NORMAL DEVIATION (Zr)	(0.841)	
STANDARD DEVIATION (So)	0.45	
DRAINAGE COEFFICIENT (m)	1.00	
STRUCTURAL NUMBER (SN)	3.694	

CHECK EQUATION : 6.701 = 6.701

Pavement Thickness:

Pavement Type	Layer coeff.	Layer thickness (inch)	Structural Number	
AC Wearing Course	0.4	1.97	0.79	
AC Binder Course	0.4	1.97	0.79	
AC Base (ATB)	0.3	3.94	1.18	
CRS	0.14	5.91	0.83	
Aggregate class G15	0.09	5.91	0.53	
			4.114	3.694

Himo-Holili Single Carriageway

Source: JICA Study Team

(2) Material Design Improvements

The Superpave mix design method was designed to replace existing Hveem and Marshall methods. The volumetric analysis common to the Hveem and Marshall methods provides the basis for the Superpave mix design method. The Superpave system ties asphalt binder and aggregate selection into the mix design process, and considers traffic and climate as well. The compaction devices from the Hveem and Marshall procedures have been replaced by a gyratory compactor and the compaction effort in mix design is tied to expected traffic.

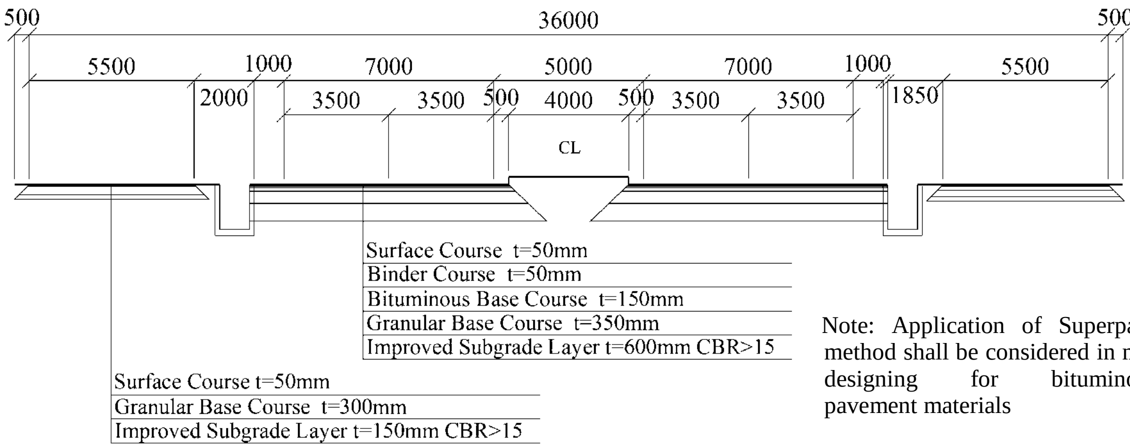
As earlier EAC guideline suggests to apply the Superpave mix design approach as follows,

"The existing pavement design standards that are used in the EAC region do not provide room for consideration of pavement temperature in mix design. Such provisions could have made it possible to make use of Superpave asphalt mix design approach."

Recommendation
Efforts should be made to undertake performance graded asphalt specification which corresponds to the maximum and minimum pavement temperatures of the region for Superpave mix design.

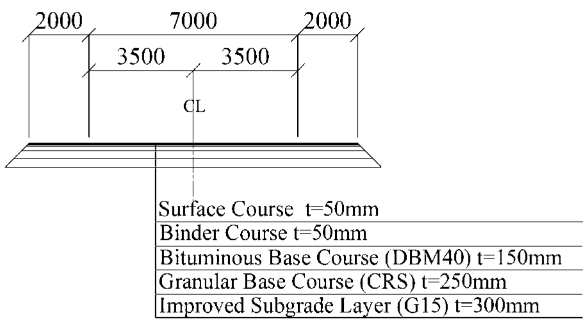
(3) Pavement Improvement Designs

Consequently, the pavement improvement design is prepared as shown in the following figures.

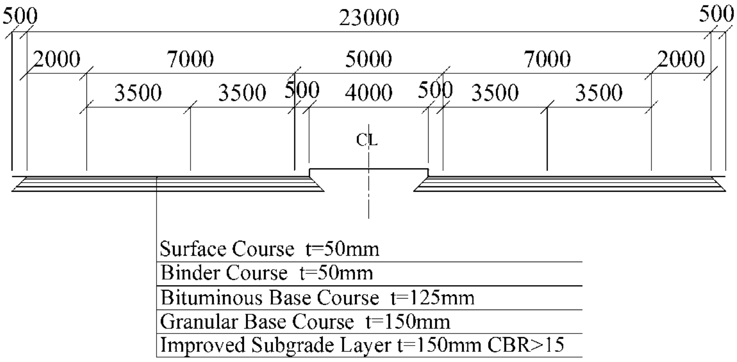


Note: Application of Superpave method shall be considered in mix designing for bituminous pavement materials

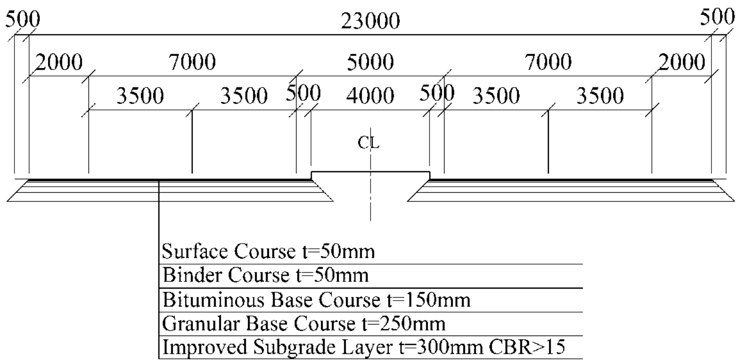
Tengeru-Usa (Dual Carriageway)



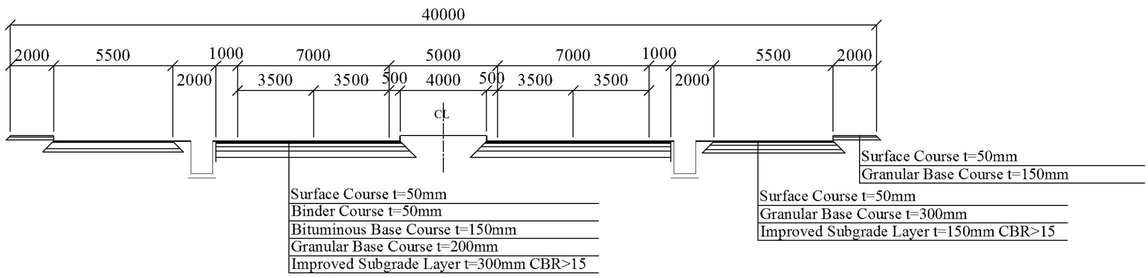
Usa -Moshi (Single Carriageway)



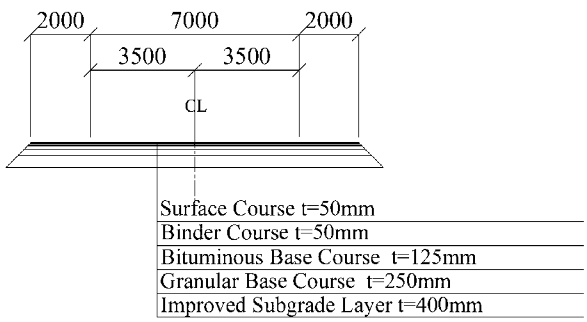
Kikafu (Dual Carriageway)



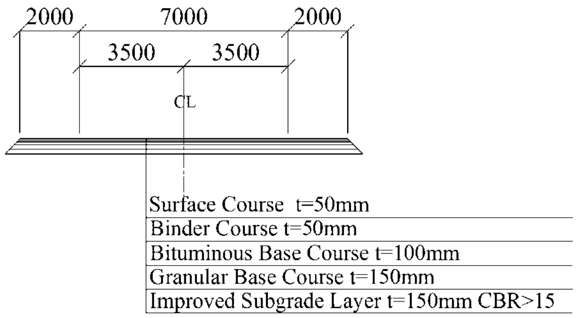
Moshi Town (Dual Carriageway, without Service Road and Footpath)



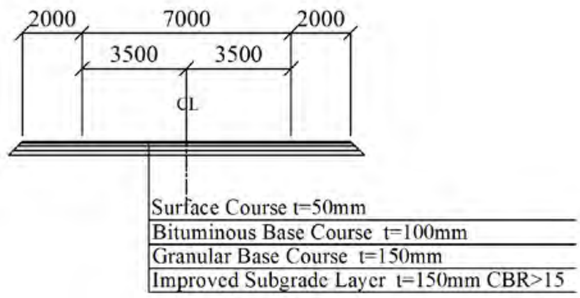
Moshi Town (Dual Carriageway, with Service Road and Footpath)



Moshi-Himo (Single Carriageway)



Himo-Holili (Single Carriageway)



KIA Access Road

Source: JICA Study Team

Figure 5.2.10 Pavement Improvement Designs

5.2.10 Junction Design

Traffic analysis is most critical element in determination of the type of junction. Road safety, O&M, land availability, traffic arrangement and geometrical requirement are also the criteria in the determination.

Generally speaking, signalized junction has advantages for traffic capacity while the roundabout has better road safety. Traffic arrangement with signalized junction is better than that with roundabout unless number of the legs is more than four (4). However, roundabout has been more popular than signalized junction in many developing countries due to operational reason as the signalized junction requires electricity supply which is not stable in the countries.

In Dar es Salaam and Arusha, there are some signals using solar power system and they look performing well. LED makes the solar power applicable for the lights because of low consumption of electricity. However, the solar power unit needs battery of which life span is short as 2 or 3 years that bring high recurrent maintenance cost. Another considerable element is that the solar power is the weather dependant that is not good for critical road safety device as is required to work whenever the traffic flows. Desirably, the solar power shall be applied as sub resources, not as main for the signal unit if the road safety is seriously prioritized. City power therefore remains as major resource for the signal to work.

Electricity supply is thus still a critical element in determination of junction type in developing countries and Tanzania is also no exception. Investigation of the electricity supply condition is therefore important.

There are some existing reports showing the electricity supply in the region, and following tables are referred from the reports.

Table 5.2.25 Electricity Supply Condition in Kilimanjaro Region

Month	Load Shedding	Suspension by Accident	Suspension by Maintenance
Jan. 2010	0.18 HR	113.47 HR	10.36HR
Feb. 2010	121.11 HR	48.06 HR	5.33 HR
Mar. 2010	159.41 HR	272.28 HR	24.53 HR
Apr. 2010	29.50 HR	120.29 HR	74.48 HR
May 2010	25.56 HR	46.55 HR	45.00 HR

Source: Monthly Operation Report, Kiyumgi Sub-Station, TANESCO

Table 5.2.26 Voltage Fluctuations in Kilimanjaro Region (Highest/Lowest)

Month	132 kV Transmission System	66 kV Transmission System	33 kV Transmission System
Jan. 2010	134/112kV	68/58kV	35/31kV
Feb. 2010	135/125kV	67/62kV	35/32kV
Mar. 2010	134/126kV	67/60kV	35/31kV
Apr. 2010	135/124kV	68/60kV	35/30kV
May 2010	137/115kV	67/60kV	34/30kV
Acceptable Range	147.7~118.8kV	72.6~59.4kV	36.3~29.7kV

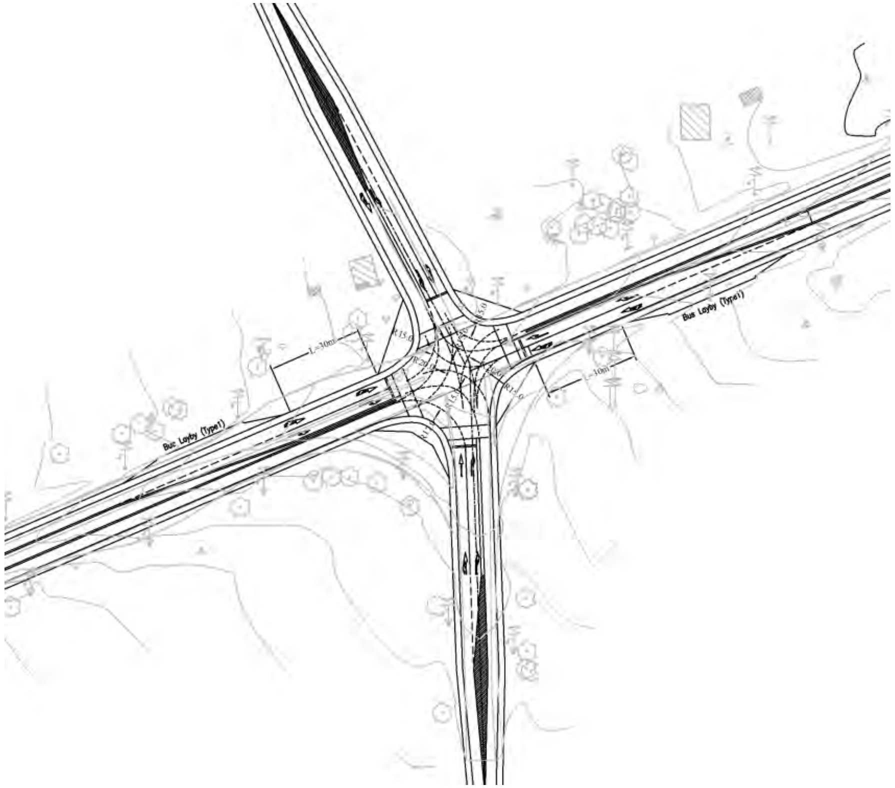
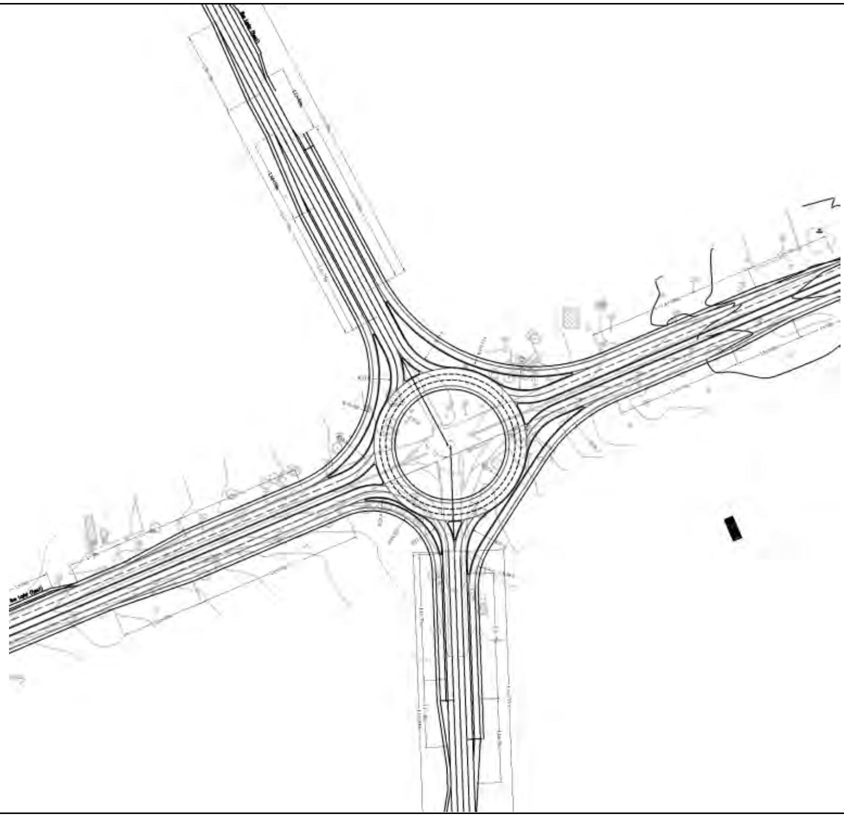
Source: Monthly Operation Report, Kiyumgi Sub-Station, TANESCO

From the above tables give an interpretation that the electricity supply condition in the project area can be identified as poor.

(1) Alternative Designs

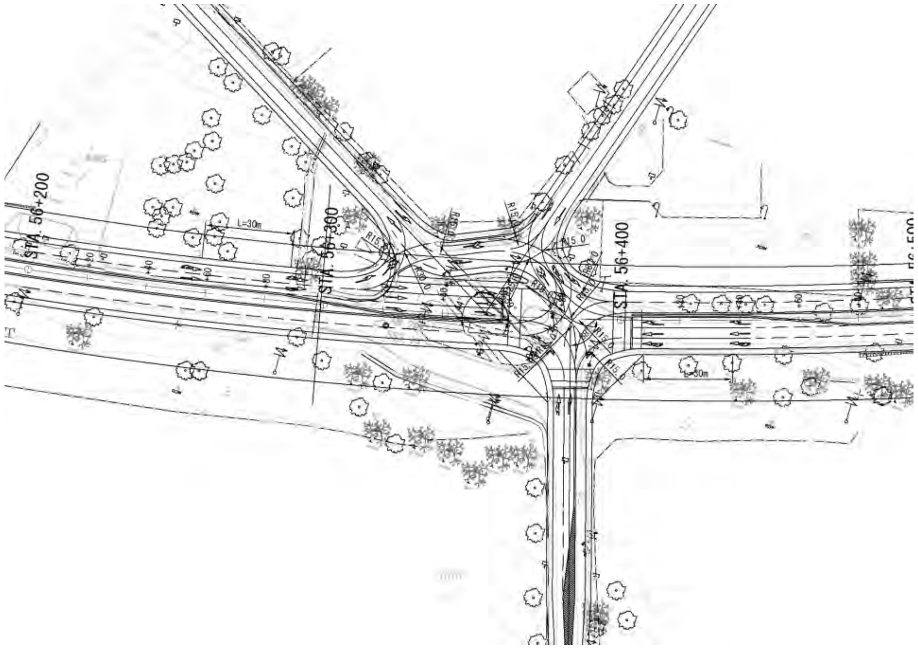
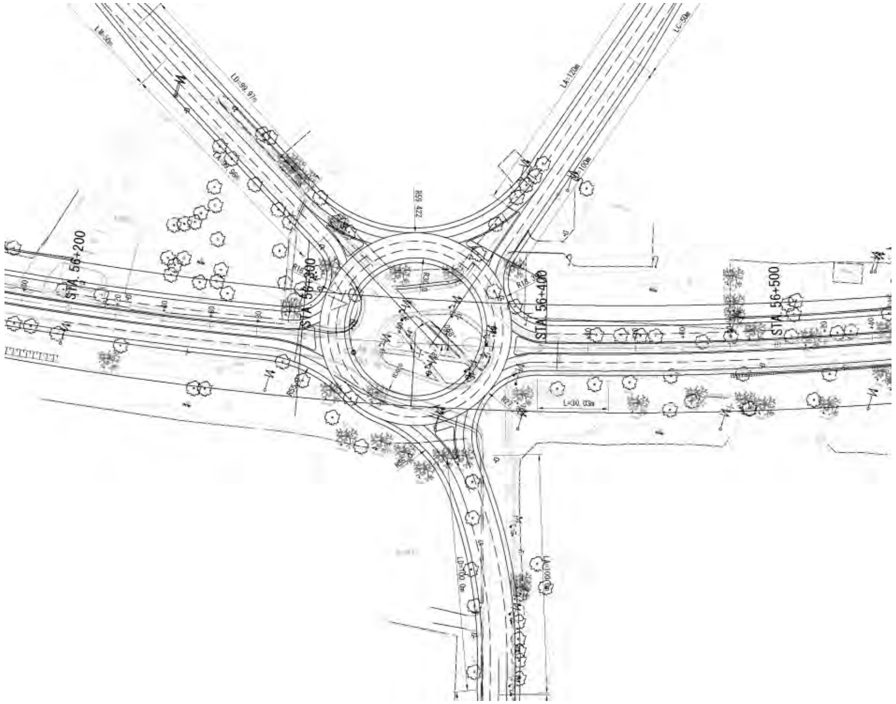
Alternative designs by junction types (Signalized and Roundabout) are prepared for three (3) junctions including KIA, Moshi and Kikafu. The Study recommends roundabout option as an optimum intersection design at these three junctions, considering traffic operation and safety.

1) KIA Junction

Signalized	Roundabout (Recommended Option)
	
Advantage: No land acquisition. More traffic capacity (750 vehicles/hour/lane) **Disadvantage**: Unstable electricity which may cause unreliable operation and increase in traffic accident risk	**Advantage:** Better traffic operation where less traffic (550 vehicles/hour/lane), Better landscape for tourist with monumental structure **Disadvantage:** May (or may not) require small scale land acquisition

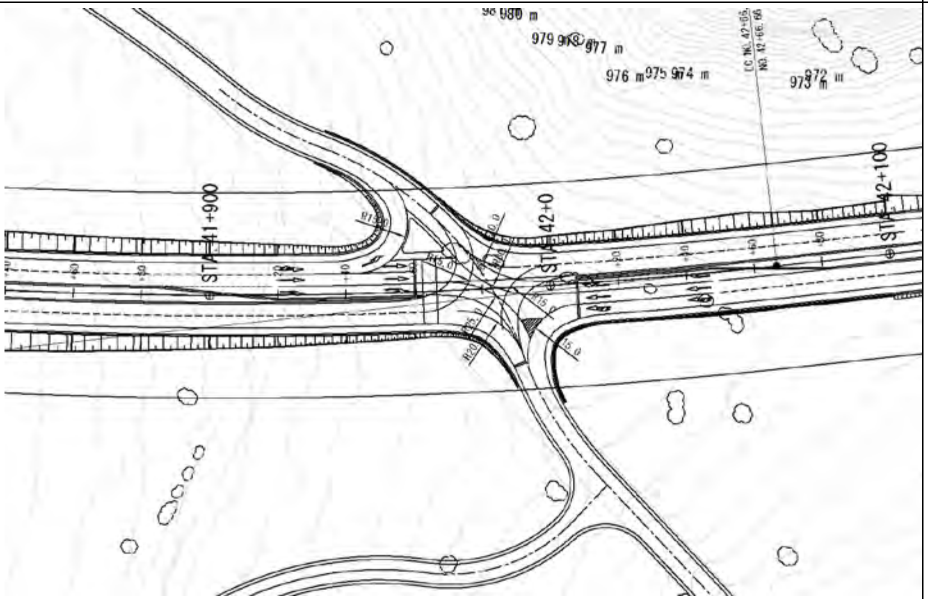
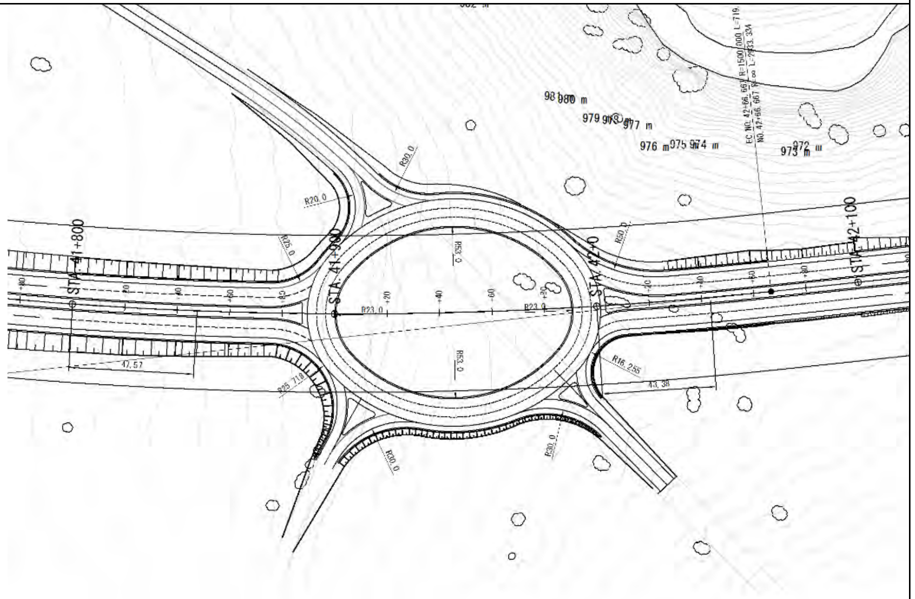
Source: JICA Study Team

2) Moshi Intersection

Signalized	Roundabout (Recommended Option)
	
Disadvantage: Unstable electricity which may cause unreliable operation and increase in traffic accident risk	**Advantage:** Better traffic operation since 5 legs with smaller crossing angle **Disadvantage:** Less traffic capacity where excessive amount of traffic (over 550 vehicles/ hour/lane), May (or may not) require small scale land acquisition

Source: JICA Study Team

3) Kikafu (West of Kikafu Bridge access to proposed Roadside Station)

Signalized	Roundabout (Recommended Option)
	
Advantage: Higher travel speed Disadvantage: Requires realignment of connected local road. Higher travel speed and non signalized operation may cause increase in traffic accident risk	**Advantage:** Better traffic operation since 5 legs with smaller crossing angle, Better landscape for tourist when Roadside Station developed **Disadvantage:** Lower travel speed (but reduce traffic accident risk), Larger size required due to waving length

Source: JICA Study Team

5.2.11 Road Safety Improvement Option

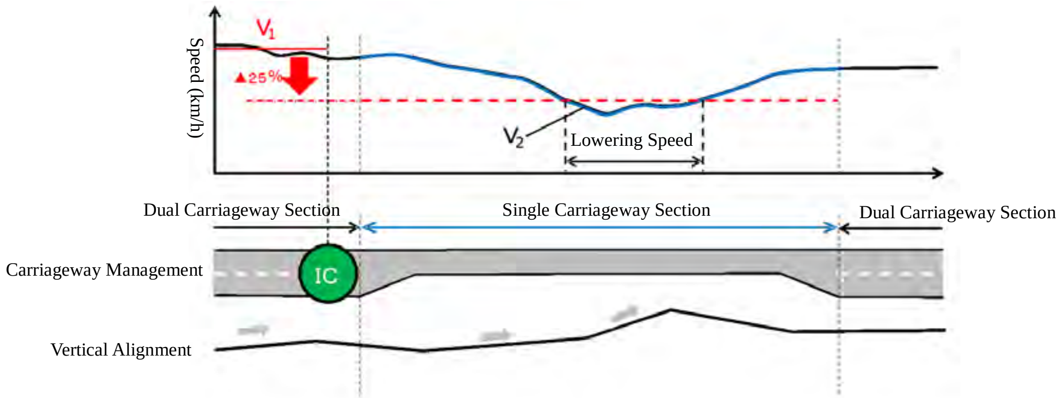
There is an observation that slow traveling traffic causes queuing even in off peak hours that increases road safety risk as fast traveling traffic attempts to make unreasonable overtaking in single carriageway section.

The stage opening to traffic with single carriageway in Japanese highways is common management practice applied in the early stage of the development. The traffic safety and inappropriate level of service however are the issues that have been raised in this single carriageway management.

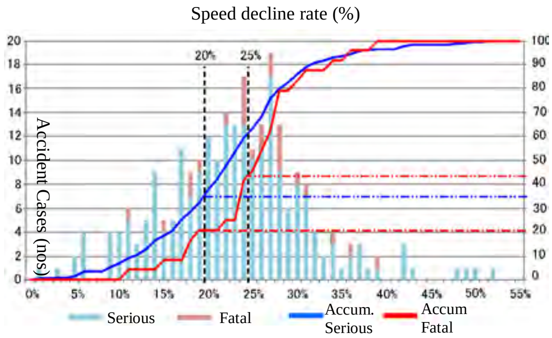
As a countermeasure, the provision of additional lanes that allows fast traffic to enable overtaking has been implemented in Japan.

A consideration for the provision according to Japanese approach is shown below.

Decline speed: Single carriageway section that travel speed (V_2) decline 25% of that in adjacent dual carriageway section shall be identified.



Relationship between traffic accident and speed decline rate



As seen in the relationship, the following finding can be identified,

- Approximately 65% of serious accidents occur in speed decline rate of more than 20%.
- Approximately 80% of fatal accidents occur in speed decline rate of more than 20%.
- Approximately 40% of serious accidents and 55% of fatal accidents occur in speed decline rate of more than 25%.

LOS according to Highway Capacity Manual (HCM) 2010

According to HCM2010, the level of service in single carriageway section have been identified based on average traveling speed.

Level of Service		Average Traveling Speed (Mile/hr) <kph>
A	Most passing maneuvers can be made with little or no delay.Service flow rate a total of 420 pcu/hr for both direction and about 15% of capacity can be achieved.	55-(60) <88.51-(96.56)>
B	Flow rates may reach 27% of capacity with continuous passing sight distance. Flow rate of 750 pcu/hr total for both direction.	50-55 <80.46-88.51>
C	Flow still stable. Flow rates under ideal condition equal to 43% of capacity with continuous passing sight distance or 1,200 pcu/hr total for both direction.	45-50 <72.42-80.46>
D	Approaching unstable flow. Flow rates, two directions, at 64% of capacity with continuous passing opportunity, or a total of 1,800 pcu/hr for both direction.	40-45 <64.37-72.42>
E	Flow rates under ideal conditions, total two way, equal to 2,800 pcu/hr. Level E may never be attained.	-40 <-64.37)

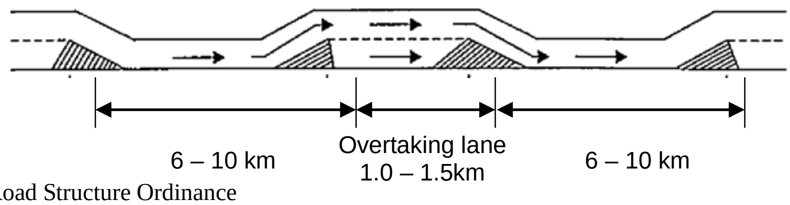
According to「A policy on Geometric Design of Highways and Streets 2011」(i.e. Green Book), target LOS for single carriageway in rural area is "C 45-50 mile/h"

Speed of 40-50 miles/h in LOS C corresponds 20-25% of speed decline rate at LOS A (55-60 miles/h)

Source: Ministry of Land, Infrastructure, Transport and Tourism in Japan[2]

The illustration above is the basis in identification of critical speed decline rate of 25%. It is therefore that section with a speed decline rate more than 25% that needs to apply an additional lane.

Basing on the above considerations, the Japanese Road Structure Ordinance specifies interval and length of the additional lane which are 6-10km and 1.0-1.5km which shall be considerable in the next design stage.



Source: Japanese Road Structure Ordinance

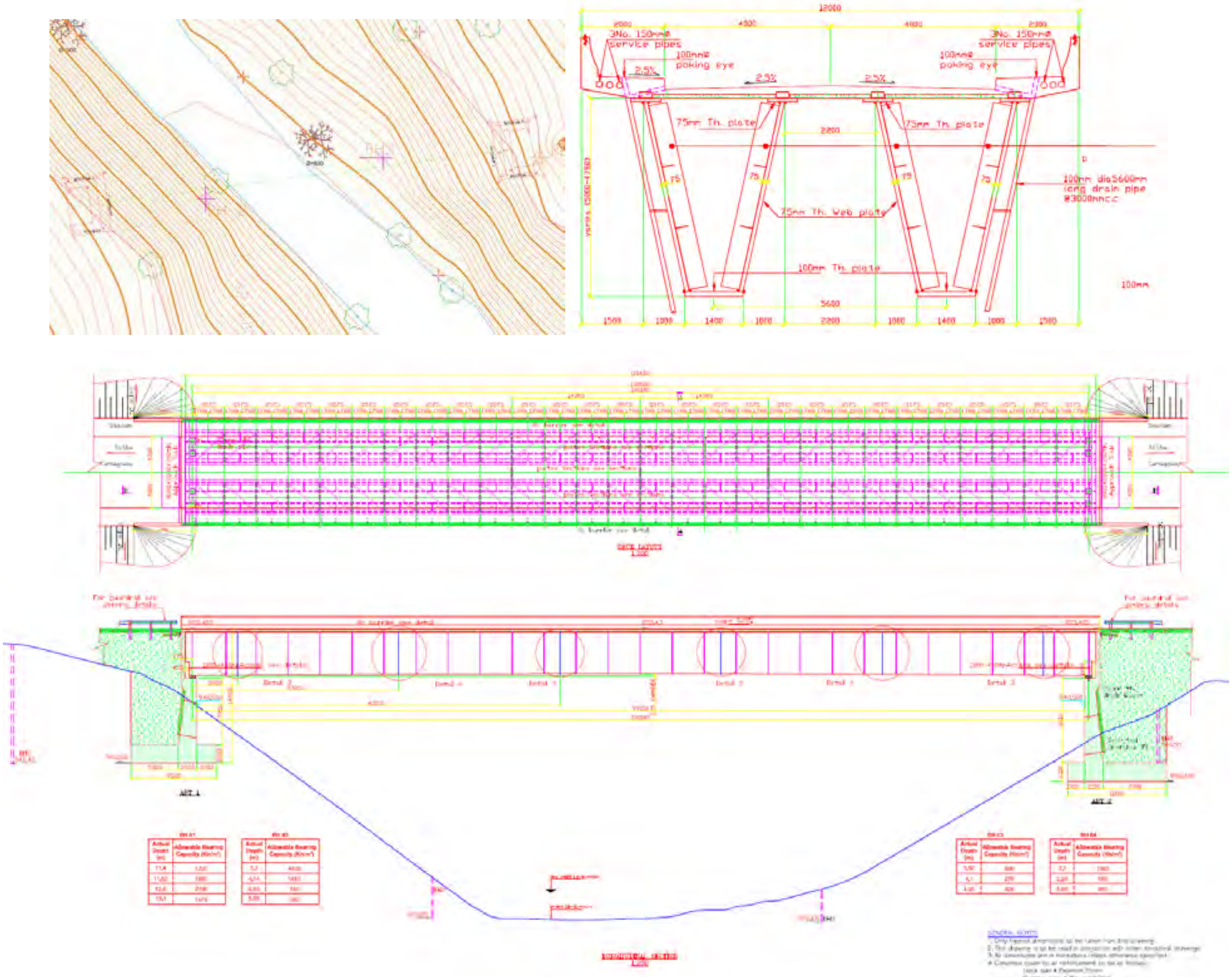
Figure 5.2.11 Interval and Length of Additional Lane

[2] http://www.mlit.go.jp/common/001122664.pdf

5.3 Study on Alternative Bridge Type

5.3.1 Review of Bridge Design by Previous FS and DD

In the previous F/S, the Kikafu Bridge was designed based on BS5400 as the applicable code for design of road structures in Tanzania. The structure was designed for all dead and transient loads. The live loads were applied for full HA loading and checked for 45 units of HB loading. The deck cross-section is based on two lane width (2x3.5m) together with 0.5m hard strips, mount-up walkways of 2.0m on both sides of the road. The overall deck width is thereby 12.0m. The bridge type is applied to simple composite box girder bridge (with open-topped trapezoidal steel sections).



Source: F/S and D/D

Figure 5.3.1 Layout of Kikafu Bridge designed by F/S

The reviews of the design of pervious F/S found the following major concerns;

- Width of bridge planned to accommodate only two lane traffic (2@3.5m) with respective 0.5m narrow shoulders despite of an increasing traffic demand.
- Difficulty in construction on both end of abutments (invert-T type) with the height being 14.5m and 16.7m respectively and located on steep slope (nearly 40 degree).
- No median space and barriers between traffic lanes.
- No lighting facilities and safety devices designed.

- Some details of structure are not clear in the drawings especially specification of materials.
- Normal structure steel be used for frequent maintenance to protect from corrosion.
- No detailed hydrological information provided in the design report.

5.3.2 Study on Alternative Bridge Type

(1) Bridge Site

The proposed new Kikafu Bridge is to be located at the new alignment section. The alignment was designed to shift approximately 600m downstream side of the existing bridge site and provide a shortcut approximately 2km shorter than the length of the existing road section. The straight alignment seems to provide good driver's sight hence be considered to decrease the risk of traffic accident.

However, the new bridge is to be located at the sag point of vertical profile which will be required for nearly 5% gradient of approach roads cannot satisfy the design criteria at the design speed of 100km/h. In order to provide an additional climbing lane along the approach road to keep the lower design speed of 80km/h. When a bridge is to be located across a steep valley section, as a general practice, a longer span bridge (i.e. location of the abutments away from the valley) is preferred that generally reduces the cost of the substructure but increases the cost of superstructure.

The overall cost performance of the construction works should be considered when deciding the positon of the bridge in terms of workability and other aspects.

Existing Kikafu Bridge (view of downstream side)

Existing Kikafu Bridge

Existing approach road to Kikafu Bridge (Moshi side)

New alignment section (geotechnical boring)

View of Kikafu River from left bank

View of Kikafu River from right bank

View of approach road (right bank)

Existing community road (right bank)

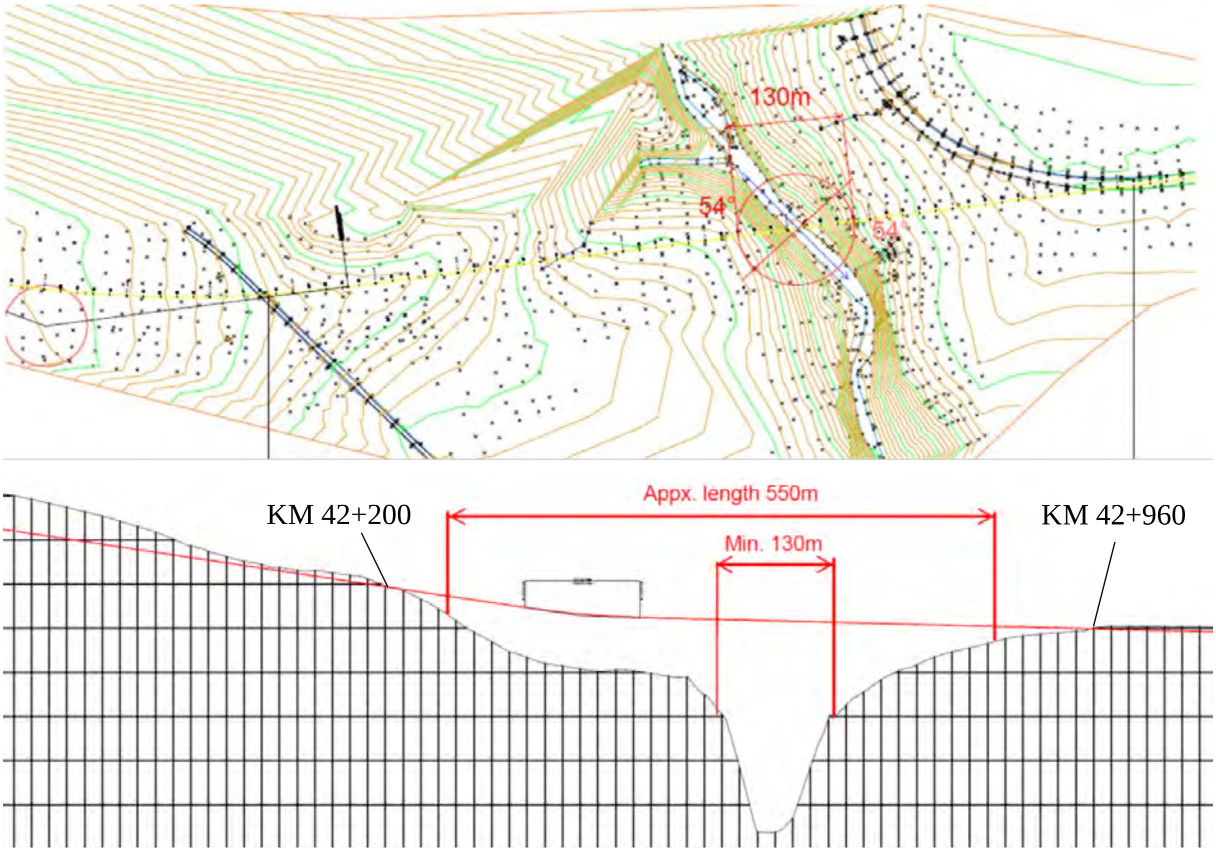
Source: JICA Study Team

Figure 5.3.2 Photographs taken at/around Kikafu Bridge

(2) Determination of the Bridge Length

The new bridge alignment is designed to cross the Kikafu River horizontally at 54 degree skew angle to the river flow. It is designed to locate the both abutments at the steep slope at vertically around 40 degree angled slope. As a result of geotechnical survey, it is identified that the spread foundations would be applicable due to the appearance of bedrock at shallow depth. In order to keep the clearance of front toe of footing from the slope surface, the footing would be accommodated into the rock substratum more than 5m depth cover from the top of the footing.

In this context, the rock excavation on the steep slope concerns potentially harmful to the construction speeds and increase of the risk for cost overrun during the works. Hence, the Study Team recommends to keep a longer span length more than 130 m where it can be observed on the gentler slope at the both locations of abutments. The span length would be longer however it is reasonably to avoid the risk for the rock excavation works so that the abutment can be safely constructed on stable ground. To secure an economical bridge design, the detailed survey works should be conducted to identify the precise locations of abutments during the detailed design.



Source: F/S and D/D

Figure 5.3.3 Bridge Length and Minimum Span Length of New Kikafu Bridge

(3) Comparative study of preferable bridge type

The superstructure of new Kikafu Bridge should be erected around 40m higher than the river bed which would be required for the difficult construction of piers in the river area. Meanwhile, the construction of piers on the steep slope will necessitate highly dangerous excavation work that will be difficult to ensure work safety. For these reasons, the new bridge should be designed to be a single span bridge without piers over 130m at the center span.

[Premise of design configurations for the selection of bridge type]

- Bridge length: Around 550m
- Both bridge ends are determined abutments from a reasonable height of abutments (H<8m).
- Skew angle of bridge: 54 degrees
- Pier arrangements in the river are technically considered subject to disturbance of water current if the skew angle is less than 60 degrees
- Min. span length more than 130m

To minimize the risk of the foundation and substructure work, the excavation at steep slope should be avoided.

Given the above design configuration, the feasible superstructure types were studied referring to the Japanese practices. The different structure types of concrete and steel girders both provide the applicable range of span length for each bridge type respectively. In respect to the initial screening of bridge types, the following bridge types were selected in accordance with the charts given in the following figure.

[Concrete bridge]

- **PC Continuous Box Girder Bridge**
- **PC Rigid Frame (Ramen) Continuous Box Girder Bridge**
- **Cable Stayed (Single pylon) Bridge**
- **Extradosed Bridge**
- **Arch (Upper Deck) Bridge**

[Steel bridge]

- **Steel Continuous Box Girder (Steel Slab) Bridge**
- **Continuous Steel Truss Bridge**
- **Arch Bridge**
- **Steel Cable Stayed Bridge**
- **Suspension Bridge**

Table 5.3.1 Type of Superstructure by Applicable Span Length (Concrete Bridge)

Type of Superstructure by Applicable Span Length (concrete)

Type		Section member	Erection Method	Applicable Span Length (m) 20 40 60 80 100 150 200 250 300	Max. span length (m)	Girder depth/ span ratio	Remarks
Simple RC	Cast insitu	Simple slab	All stage		10	1/10~1/15	
		Continuous slab			20	1/11~1/16	
		Simple hollow slab			15	1/14~1/17	
		Continuous hollow slab			20	1/15~1/18	
Simple PC Girder	Pre-cast beam	Pre-tension slab	Crane		Normal (24)	1/14~1/26	JIS A5313
		Pre-tension T shaped girder	Crane		(24)	1/18~1/19	JIS A5313
		Post-tension slab	Crane tempo. girder		(45)	1/24~1/29	
		Post-tension T shaped girder	Crane tempo. girder		(45)	1/13~1/17	
		Post-tension PC/RC composite girder	Crane tempo. girder		(45)	1/12~1/16	
	Cast insitu	Post-tension hollow slab	All stage		54	1/20~1/24	
		Box girder	All stage		69	1/16~1/20	
Precast PC Continuous Girder	Pre-cast PC beam	Pre-tension slab	Crane		(24)	1/14~1/26	
		Pre-tension T shaped girder	Crane		(24)	1/18~1/19	
		Post-tension slab	Crane tempo. girder		(35)	1/24~1/26	
		Post-tension T shaped girder	Crane tempo. girder		41	1/13~1/17	
		Post-tension PC/RC composite girder	Crane tempo. girder		(45)	1/12~1/16	
PC Continuous girder		Hollow slab	All stage Movable		45	1/20~1/24	
		Box girder	All stage		60	1/16~1/22	
			Movable		45	1/17~1/23	
			Incremental		69	1/15~1/17	
			Cantilever		170	1/18~1/36 On Mid	
		Composite girder	All stage Movable		39	1/17	
Rigid Frame (Rumen)	T shaped	Hollow slab	All stage		30	1/20	
		Box girder	All stage		48	1/18	
			Cantilever		104	1/18~1/35 On Mid	
	Continuous	Hollow slab	All stage		32	1/20~1/22	
		Box girder	All stage		50	1/18	
			Cantilever		153	1/18~1/35 On Mid	
Cable Stayed		Hollow slab/box Edge girder	All stage		96	1/40~1/80	
			Cantilever		260	1/40~1/80	
Extra-dosed		Box girder	**Cantilever**		180	1/30~1/50 On Mid	
Arch		Hollow girder	**All stage Cantilever Roaring Support**		235	−	
Steel/concrete composite		Waved steel plate	All stage		−	1/17~1/21	
			Incremental		46	1/15~1/21	
			Cantilever		97	1/17~1/38 On Mid	

( ) is indicated max. span length at normal design for standard type of bridge

Mostly applicable range / Relatively applicable range

Source: JICA Study Team, referring to Bridge Design Literatures in Japan (Ministry of Land, Infrastructure and Transport, Pre-stress Concrete Construction Association, Japan Bridge Construction Association)

Table 5.3.2 Type of Superstructure by Applicable Span Length (Steel Bridge)

Type of Superstructure by Applicable Span Length (Steel)

Type			Applicable Span Length (m) 20 40 60 80 100 150 200 250 300	Max. span length (m)	Girder depth/ span ratio	Remarks
Plate Girder Bridge	Simple steel H shaped girder			25	h/L= 1/14 ~27	
	Simple steel non-composite I shaped girder			Normal 44	1/15~20	
	Simple non-composite box girder			60	1/16~21	
	Continuous steel non-composite I girder			70	1/18~25	
	Continuous steel non-composite box girder			75	1/19~26	
	Continuous steel I girder (steel slab)			65	1/16~22	
	Continuous steel box girder (steel slab)			Few girder 91	1/15~20	
	Integral bridge (rigid frame)			190	1/20~30	
	Narrow-width box girder			300	1/22~28	
Rigid frame (π shaped ramen)				234	1/32	
Truss	Simple truss			164	1/7~9	
	Continuous truss (with gelber hinge)			548	1/8~10	
Stiffened deck Arch	Upper deck	Langer		150	f/L= 1/6 ~7	
		Lhose		140	1/6.6~6.8	
	Mid	Lhose/ Nielsen		329	1/6.0~7.3	
	Lower deck	Langer		330	1/6.0~7.3	
		Trussed Langer		518	1/6.8~6.9	
		Lhose		175	1/6.8~6.9	
		Nielsen Lhose		305	1/6.5	
Non stiffened arch				518	1/5.3~6.3	
Cable stayed bridge				890	1/4.7	
Suspension bridge				1,991	1/8.4	

Mostly applicable range   Relevantly applicable range

Note (1) Girder depth of arch bridge is indicated a ratio of arch rise

(2) Girder depth of truss bridge is indicated a ratio of length of main member/ span length

(3) Continuous steel I shaped bridge (few main girder) is applied only to a grade road (expressway, etc.)

Source: JICA Study Team, referring to Bridge Design Literatures in Japan (Ministry of Land, Infrastructure and Transport, Pre-stress Concrete Construction Association, Japan Bridge Construction Association)

Among these feasible types of bridge, the comparative study has been conducted as given in the following table. As a result, the following types of bridge were selected as preferable alternatives for the detailed comparative study given in the same table.

[Concrete bridge]

Alt.-1 PC Continuous Box Girder Bridge

Alt.-2 PC Rigid Frame (Ramen) Continuous Box Girder Bridge

Alt.-3 Extradosed Bridge

Alt.-4 PC Cable Stayed Bridge

[Steel bridge]

Alt.-5 Steel Continuous Box Girder (Steel Slab) Bridge

Alt.-6 Continuous Steel Truss Bridge

Alt.-7- Steel Arch Bridge

The detailed comparative study in the following tables resulted in the following types of bridges obtaining the highest scores of AHP among the alternative bridges.

[Concrete bridge]

Alt.-3 Extradosed Bridge

[Steel bridge]

Alt.-7- Steel Arch Bridge

PC Extradosed Bridge has obtained the highest score among the alternatives. The Steel Arch Bridge including through type/ half- through type arch bridges are followed. These bridges have main girders, cable and pylon above the road surface. These bridges may have damage when those are accidentally hit by vehicle. Given the practices that numbers of bridges are damaged, the protective measure should be considered when adopting any of these bridge types from the viewpoint of safe operation after completion.

In addition, steel bridge should be required for more frequent maintenance than that of concrete bridge. The weathering steel (anti-corrosive steel) is an option for preferable measure to reduce the life-cycle cost of steel bridge. However, the application of weathering steel (like rusting color of girder) is subject to the study in an aspect of aesthetic view of bridge.

On December 11, 2015, a tripartite meeting was held involving TANROADS, JICA and Study Team and confirmed PC Extradosed Bridge as an optimum bridge type, considering maintainability, workability during construction and landscape.

Table 5.3.3 Selection of Bridge Type of New Kikafu Bridge

(1) Candidates Bridge Type (Concrete Bridge)

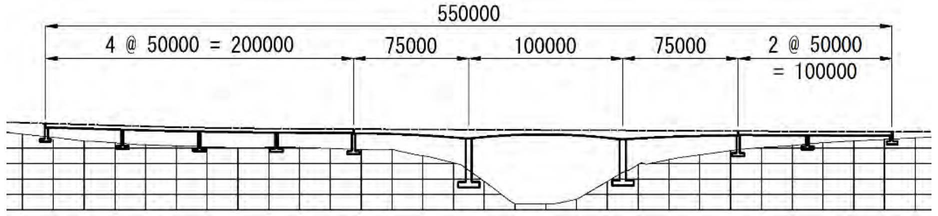
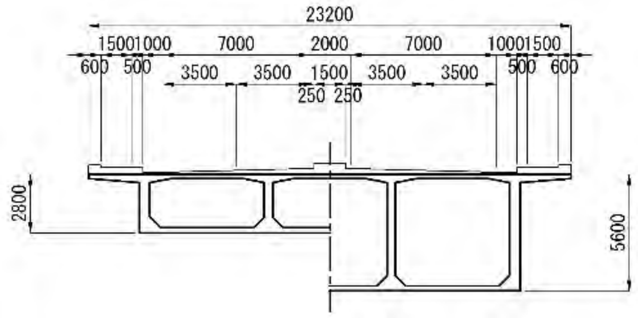
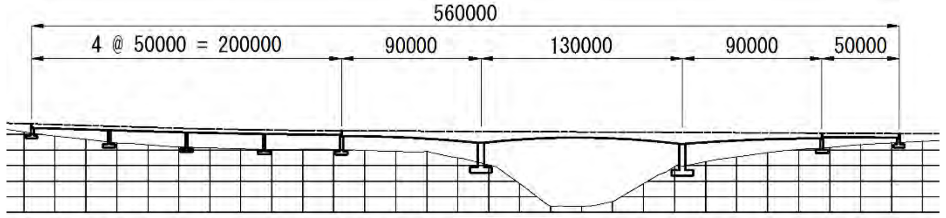
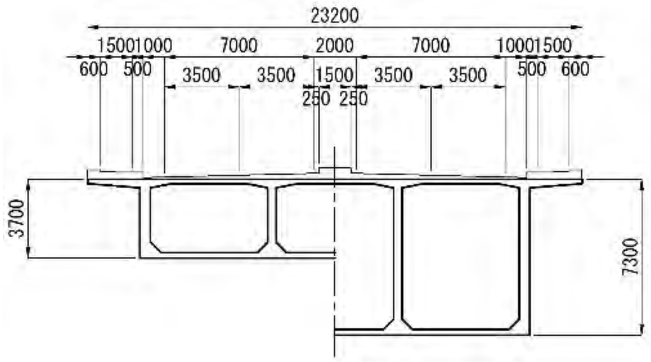
Main Span	Type	PC Continuous Box Girder	PC Rigid Frame (Ramen) Continuous Box	Cable stayed (Single pylon)	Extradosed	Arch
	Erection	Cantilever	Cantilever	Cantilever	Cantilever	Cantilever/ Roaring/ Cable erection
	Economic Span length (m)	50-110m	40 - 130m	100 - 250m	100 - 180m	50 – 230m
	Photos					
Practice in EAC countries		Yes	Yes	Under construction in Uganda/ Tanzania	Yes (e.g., Abai Bridge in Ethiopia)	Yes
Maintainability		Superior (concrete structure)	Superior (concrete structure)	Fair (necessary cable maintenance)	Superior (using maintenance free cable)	Superior (concrete structure)
Landscape		Fair	Fair (unbalanced with low height piers)	Fair (Single pylon)	Superior (balance with twin towers)	Superior with beauty of arch
Initial Assessment		Assumed a limit of practical span length be adopted that might have risk to secure a robust structure.	Assumed the lower pier height with spread foundation that would be technically inadequate to form a rigid frame (ramen).	Assumed that the construction of tall pylon at the edge of river would be difficult within single dry season and required higher technique for maintenance of the stayed cables	Recommended in applying suitable span length that would be technically efficient as well as economically optimized.	Not recommended in the complex work sequence that would be practically difficult to control the quality and safety taking into account the local condition.

(2) Candidates Bridge Type (Steel Bridge)

Main Span	Type	Steel Continuous Box Girder (Steel Slab)	Continuous Steel Truss	Arch	Steel Cable Stayed Bridge	Suspension Bridge
	Erection	Steel (Weathering steel)	Steel (Weathering steel) Composite Steel/ concrete	Steel (Weathering steel)	Steel (Weathering steel)	Steel
	Economic Span length (m)	70 - 160m	75 – 110m	70 – 180m	130 – 300m	Over 130m
	Photos					
Practice in EAC countries		Yes	Yes	Yes	No (except minor bridge)	No (except minor bridge)
Maintainability		Good (weathering steel) (Concerns with quality assurance of maintenance of pavement on the steel slab).	Good (weathering steel)	Good (weathering steel)	Good (weathering steel)	Good (weathering steel) (Need maintenance of main cables that requires certain ability and capacity).
Landscape		Fair	Good	Superior with beauty of arch	Fair (due to single pylon)	Fair (due to small scale)
Initial Assessment		Needs certain maintenance ability for paving on the steel slab.	The depth of fresh rock is uncertain from the observation of subsoil survey (boring) given weathering and fluctuating of rock substratum that would be subject to the design variation and a risk for cost increase.	For deck arch, soundness of rock substratum is uncertain that would be subject to the design variation of foundation and risk for cost increase. Risk for foundation works at steel slope shall also be minimized. For tied arch, structurally applicable and appearance of bridge is exquisite.	Not recommended due to unreasonable bridge structure required the span length for Kikafu River.	Not recommended due to unreasonable bridge structure not to meet the requirement of the span length of Kikafu River.

Source: JICA Study Team

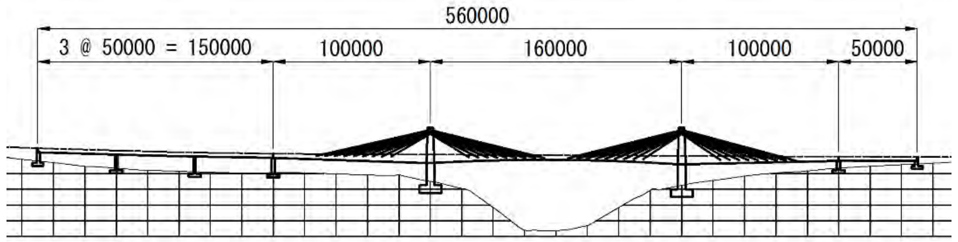
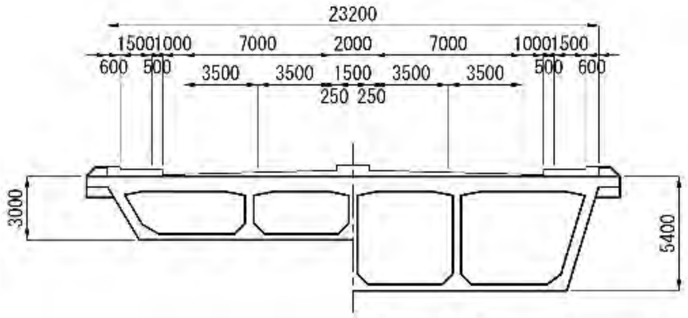
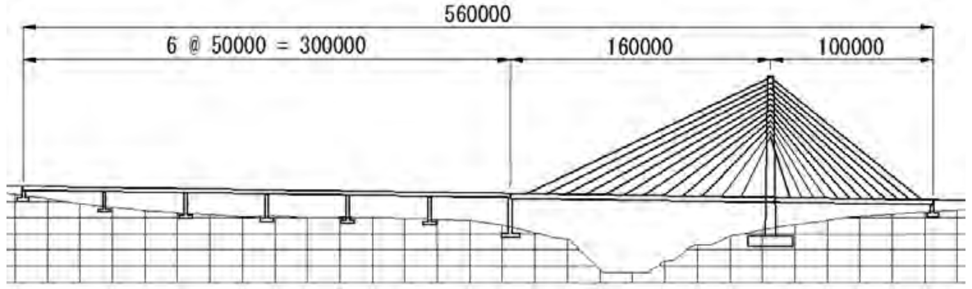
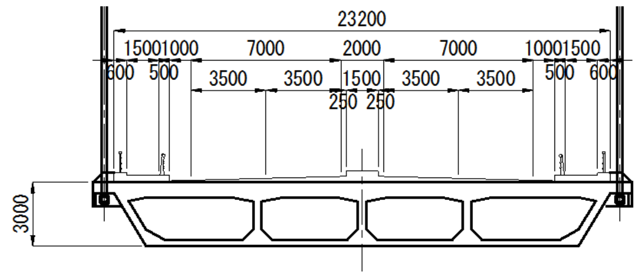
Table 5.3.4 Comparative Study for Possible Candidates Bridges (Concrete Bridge)

Side View	Cross Section	Criteria		Assessment		Score[*2,3]	
Alt-1 5 span continuous PC box girder bridge+ PC box girder bridge 		Cost (Mil. USD) *1	Super.	17.3 (12.1)	Most reasonable cost due to general type of bridge and abundant practices in EAC member countries however implicated potential cost increase for foundation and substructure works.	A	B (0.65)
			Sub.	8.7 (6.1)			
			Temp.	7.8 (5.5)			
			Total	33.8 (23.6)			
		Structure		- Economic span length ranging between 60m and 110 m - Efficient girder erection by cantilever method using form traveler crossing over deep valley safely - High durability and maintainability for pre-stressed concrete girder		B	
This bridge structurally performs a high maintainability in connecting PC box girders by minimizing expansion joints of the superstructure. Launching girders from the piers located in the river slope is presumed a difficult construction concerning the excavation of hard rock (assumed 50-100MN/m2) on the steep slope (40-45 degree).	Construction Period: 3.0 years	Workability		- Applying cantilever erection over the river assumed while applying all stage method at on land viaduct - Potential risk for the pier construction to be located on the steep slope of the river that be difficult to haul materials and equipment for the excavation.		C	
		Maintainability		Free maintenance by using PC		A	
		Landscape		Fair aesthetic view due to general type of bridge		B	
Alt-2 3 span PC continuous rigid frame PC box girder bridge +PC box girder bridge 		Cost (Mil. USD) *1	Super.	19.3 (13.5)	Most reasonable cost due to general type of bridge and abundant practices in EAC member countries.	A	C (0.50)
			Sub.	7.7 (5.4)			
			Temp.	8.1 (5.7)			
			Total	35.2 (24.6)			
		Structure		- Economic range of span length between 40m and 130 m. - Efficient girder erection by cantilever method using form traveler crossing over deep valley safely - A span 130m with low pier (assumed less than 20m height) standing on spread foundation be structurally difficult to form a rigid frame system (ramen)		C	
This bridge performs a high durability and maintainability eliminating bearing shoes at launching piers and saving maintenance cost in connecting girders. Unbalanced structure in combination between long span and low piers might be impossible to form rigid frame system.	2.9 years	Workability		- A similar work sequence with Alt.1 assumed using cantilever erection over the river and all staging method on land section. - A potential risk for requiring shortening of middle span and rearrangement of configuration if rigid frame system cannot be workable		C	
		Maintainability		- Free maintenance by using PC. - Rigid frame between girder and piers without the bearing shoe for saving maintenance cost		A	
		Landscape		Fair aesthetic view due to low piers		C	

Note1: Cost indicates construction costs of 4 lane bridge and cost in (parenthesis) indicates that of 2 lane bridge. Note2: Above score indicates A (Reasonable or superior), B (Fair or tolerable), C (Unreasonable or inferior). Note3: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A=1.0, B=0.5, C=0.0). The higher the score, the better the performance of the bridge type.

Source: JICA Study Team

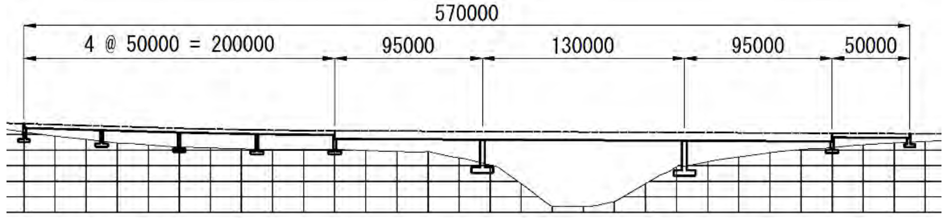
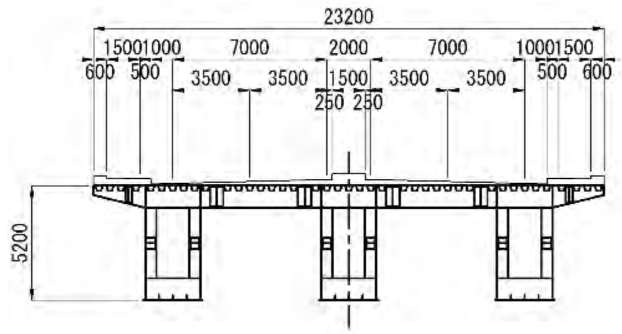
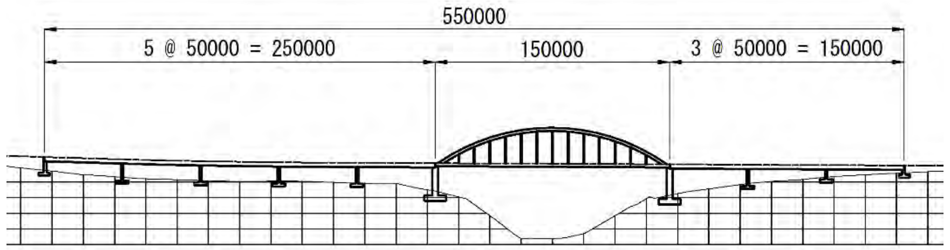
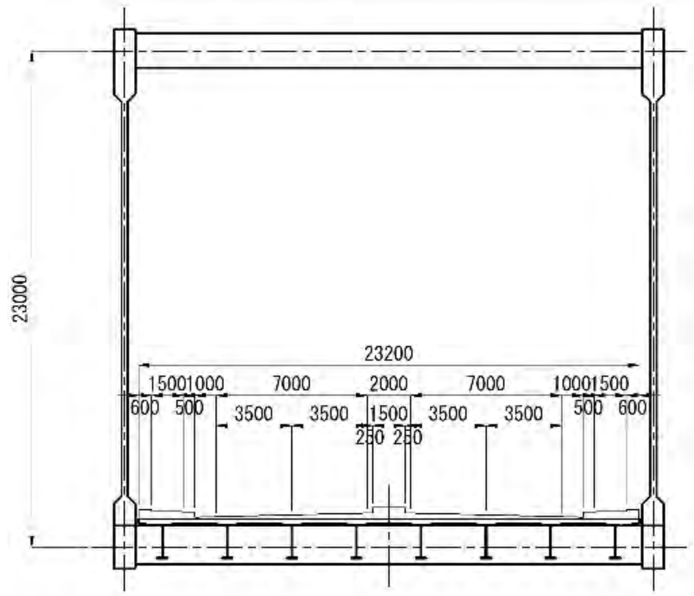
Table 5.3.5 Comparative Study for Possible Candidates Bridges (Concrete Bridge)

Side View	Cross Section	Criteria			Assessment	Score[*2,3]	
Alt-3 3 span continuous PC extradosed bridge+ PC box girder bridge 		Cost (Mil. USD) *1	Super.	26.9 (18.8)	Tolerable cost to realize a suitable longer span PC bridge however implicated comparatively longer construction period	B	A (0.85)
			Sub.	10.8 (7.5)			
			Temp.	11.3 (7.9)			
			Total	49.0 (34.2)			
		Structure			- Economic span length around 150m. - Efficiently arranging outer cables from tower to enable longer span length with using box girder. - High durability and maintainability for pre-stressed concrete girder	A	
		Workability			- Similar cantilever erection like Alt 1 and 2 assumed in addition to tower construction	A	
This bridge provides a longer span bridge which can reduce the number of piers in anticipated the rock excavation at foundation works and provides high maintainability minimizing expansion joint of superstructure. The aesthetic view of this bridge is one of the best candidates harmonizes with the scenery of the site.	3.1 years	Maintainability			- Free maintenance by using PC (if using maintenance free outer cable).	A	
		Landscape			- Superior aesthetic view in combination with twin towers of bridge	A	
Alt-4 2 span continuous PC cable stated bridge+ PC box girder bridge 		Cost (Mil. USD) *1	Super.	27.6 (19.3)	Slightly higher cost to meet the site condition of Kikafu over 160m span length by single pylon cable stayed bridge.	B	B (0.57)
			Sub.	11.0 (7.7)			
			Temp.	11.6 (8.1)			
			Total	50.2 (35.1)			
		Structure			- Economic span length around 200m longer than that of Alt.3. - Depth of box girder can be reduced efficiently but critical structural behaviour by wind force	B	
	3.5 years	Workability			- More difficult cantilever erection than Alt 1, 2 and 3 assumed in addition to high pylon construction - Potential risk with the construction of high pylon, cantilever work and large spread footing - Longer construction period assumed for high pylon construction than other PC bridges	B	
This bridge has two lateral stay-cables plane that are anchored at the edge of the transverse ribs of box girder, fixed at a main pylon. Unsymmetrical stay-cables give an unique impact to the aesthetic view in the background of Mt. Kilimanjaro.		Maintainability			- Need maintenance of stayed cable be required higher technique - Need operational control for the traffic during strong wind	B	
		Landscape			- Good aesthetic view in combination with high pylon of bridge might harm the view of Mt Kilimanjaro	A	

Note1: Cost indicates construction costs of 4 lane bridge and cost in (parenthesis) indicates that of 2 lane bridge. Note2: Above score indicates A (Reasonable or superior), B (Fair or tolerable), C (Unreasonable or inferior). Note3: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A=1.0, B=0.5, C=0.0). The higher the score, the better the performance of the bridge type.

Source: JICA Study Team

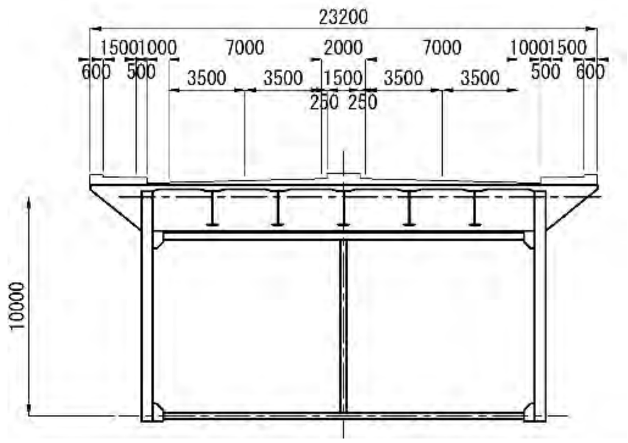
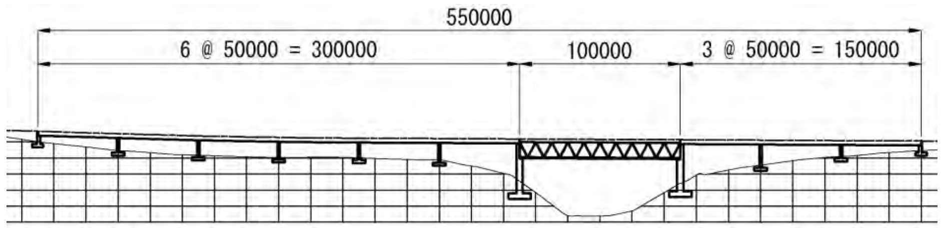
Table 5.3.6 Comparative Study for Possible Candidates Bridges (Steel Bridge)

Side View	Cross Section	Criteria			Assessment		Score[*2,3]
Alt-5: 3 span continuous steel box girder bridge (steel slab) + Steel I girder bridge  This bridge shortens the construction time by prefabricating the girders at factory in advance. Using weathering steel also saves maintenance cost however a rusty color of girder may harm the aesthetic view of bridge.	 2.8 years	Cost (Mil. USD) *1	Super.	31.0 (21.7)	Comparatively higher cost among the alternatives in spite of the shortest construction time.	C	C (0.28)
			Sub.	12.4 (8.7)			
			Temp.	13.0 (9.1)			
			Total	56.5 (39.5)			
		Structure			- Appx. 20% of self-weight lighter than concrete girder to efficiently save cost by downsizing of substructure and foundation. - Possible saving time for slab works eliminating the steps for form works and concrete casting by using steel slab	A	
		Workability			- Possibly applying launching erection by using temporary nose girder crossing over the river while applying TC bent supporting the girder on land section. - Difficult to locate the temporary bent on steep slope during the erection of girder over the river	B	
		Maintainability			- Recommend to apply weathering steel for saving maintenance routine - Require higher technical capacity for the maintenance of pavement on the steel slab	C	
		Landscape			Non symbolic view due to general type of bridge	C	
Alt-6: Simple steel tied arch bridge+ Steel I girder bridge  This bridge shortens the construction time similarly with Alt.1. The aesthetic view is one of the best candidates however using weathering steel may harm the aesthetic view of bridge with the rusty color although saving maintenance cost.	 2.7 years	Cost (Mil. USD) *1	Super.	27.1 (18.9)	Tolerable cost to realize a suitable longer span bridge with a good aesthetic view	B	A (0.75)
			Sub.	10.8 (7.6)			
			Temp.	11.4 (8.0)			
			Total	49.3 (34.5)			
		Structure			- Tide arch bridge incorporates a tie between two opposite ends of the arch. The tie is capable of withstanding the horizontal thrust forces which is normally exerted by the abutments of an arch bridge. - Vertical members and hanger cable tiding between Arch rib and bottom chord separates the type of arch called as "Lohse" using vertical member and "Nielsen" using oblique cables.	A	
		Workability			- Assumed that cable erection method would be applied to erect the arch rib crossing over the Kikafu River. - Need skilled workers to control the erection of members from cables	B	
		Maintainability			Recommend to apply weathering steel for saving maintenance routine	A	
		Landscape			Superior aesthetic view with combination of arch however harming with color of bridge if weathering steel is applied.	A	

Note1: Cost indicates construction costs of 4 lane bridge and cost in (parenthesis) indicates that of 2 lane bridge. Note2: Above score indicates A (Reasonable or superior), B (Fair or tolerable), C (Unreasonable or inferior). Note3: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A=1.0, B=0.5, C=0.0). The higher the score, the better the performance of the bridge type.

Source: JICA Study Team

Table 5.3.7 Comparative Study for Possible Candidates Bridges (Steel Bridge)

Side View	Cross Section	Criteria			Assessment	Score[*2,3]	
Alt-7 Simple steel warren truss bridge + Steel I girder bridge		Cost (Mil. USD) [*1]	Super.	22.9 (16.0)	Tolerable cost however implicates potential cost increase for foundation and substructure works.	B	C (0.52)
			Sub.	9.2 (6.4)			
			Temp.	9.6 (6.7)			
			Total	41.7 (29.2)			
		Structure			- A truss girder associates with an isosceles triangle steel frame provides light self-weight to enable downsizing of the foundation and substructure. - Also, rationalized truss girder associates with ratis frame using shaped steel provide more self- weight down.	A	
This bridge shortens the construction time similarly with other steel bridges however requires construction of piers on the river slope that is presumed a difficult construction concerning the excavation of hard rock (50-100MN/m2) on the steep slope (40 -45degree).		Workability			- Assumed that launching erection method would be applied for crossing the Kikafu River while stage erection using TC bent support can be generally applied. - Potential risk for the pier construction to be located on the steep slope of the river that be difficult to haul materials and equipment for the excavation.	C	
	2.8 years	Maintainability			Recommend to apply weathering steel for saving maintenance routine and use PC slab to increase durability	A	
		Landscape			No good with unbalanced structure due to high truss and low piers	C	

Note1: Cost indicates construction costs of 4 lane bridge and cost in (parenthesis) indicates that of 2 lane bridge. Note2: Above score indicates A (Reasonable or superior), B (Fair or tolerable), C (Unreasonable or inferior). Note3: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A=1.0, B=0.5, C=0.0). The higher the score, the better the performance of the bridge type.

Source: JICA Study Team

5.3.3 Bridge Design

(1) Design Standards

The design standards used for bridge design are as follows:

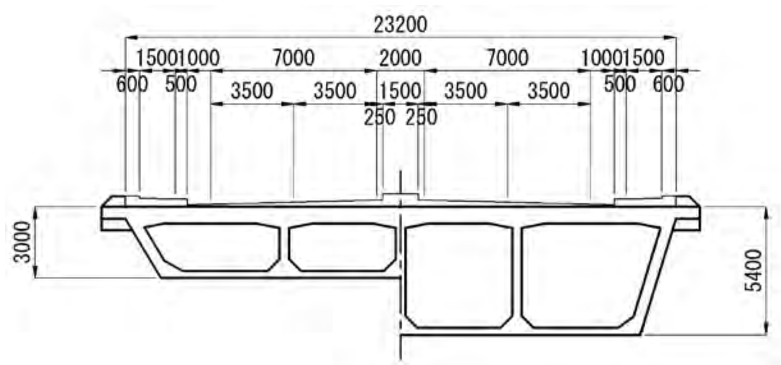
- Latest British Standard BS 5400
- Specifications for Highway Bridges (Part I-V), Japan Road Association
- Specifications for design and construction of PC cable stayed & Extradosed bridge, Japan Prestressed Concrete Institute
- AASHTO LRFD 2007, Section 3
- Draft Code of Practice for the Design of Road Bridges and Culverts Reprinted July 2001, SATCC

(2) Geometric Design Criteria

Table 5.3.8 Geometric Design Criteria

Criteria	Value
1. Bridge Length	560 m (Main span L=160m)
2. Free board from HWL	1.5m
3. Maximum Grade	3.0%
4. Minimum Grade	0.3%
5. Design Speed	100km/h
6. Total Width of the Bridge	23.2m (0.6+1.5+1.0+7.0+2.0+7.0+1.0+1.5+0.6)
7. Cross Fall	2.5% (for road way), -1.0% (for sidewalk)

Source: JICA Study Team



Source: JICA Study Team

Figure 5.3.4 Typical Cross-section for New Kikafu Bridge

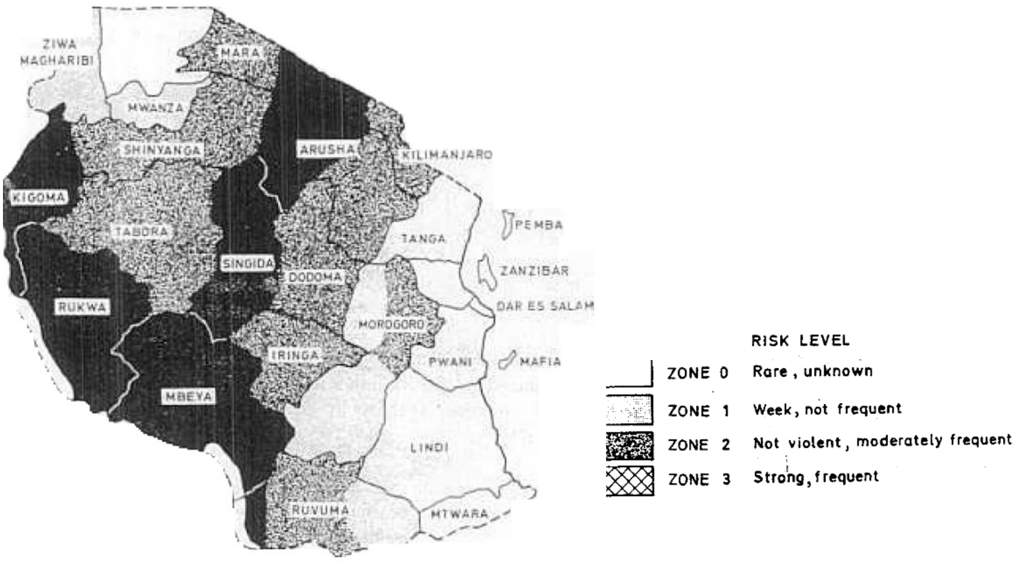
(3) Design Loads

Live Load

NA and NB 45 units are applied. However, the design of the new Kikafu Bridge is considered to apply B-live loading in compliance with Extradosed Design Specification published in 2000.

Seismic Load

According to the following figure, seismic risk map in Tanzania indicates that Kikafu is in the area of secondly low risk for earthquake. Accordingly, the level for common seismic force (Kh=0.1: Level-1) is considered as precondition for bridge design.



Source: BRU Technical Guideline no.2, Loads for Structural Design Building Research Unit, Ministry of Lands, Housing and Urban Development

Figure 5.3.5 Seismic Risk Map of Tanzania

Other Loads

The following types of loads are considered as required for bridge design:

- Dead Load
- Impact Load
- Wind Load
- Influence of creep for concrete
- Influence of dry shrinkage for concrete
- Earth pressure
- Static water pressure

Load combinations

The new Kikafu Bridge is designed against the severest unfavourable combination of loads and forces among those given in the following table.

Table 5.3.9 Load Combinations

Load combination	
Superstructure	1. Principal load (P) + particular load to be regards as principal load (PP) 2. (P) + (PP) + Thermal force (T) 3. (P) + (PP) + Wind load (W) 4. (P) + (PP) + (T) + (W) 5. Principal load other than live load and impact + Earthquake force (EQ) 6. Wind load (W) 7. Temporary load during erection (ER)
Substructure	1. (P) + (PP) 2. (P) + (PP) + (T) 3. (P) + (PP) + (W) 4. (P) + (PP) + (T) + (W) 5. Principal load other than live load and impact + (EQ) 6. Temporary load during erection (ER)

Source: Specifications for Highway Bridges, Japan

(4) Material

Unit Weight for Materials

Table 5.3.10 Unit Weight for Materials

Designation	Self-weight (kN/m^3)	Designation	Self-weight (kN/m^3)
Steel	77.0	Cement, Mortar	21.0
Reinforced Concrete	24.5	Asphalt Concrete	22.5
Pre-stressed Concrete	24.5	Concrete Pavement	23.0
Non-reinforced Concrete	23.0	Timber	8.0

Strength of Materials

Specifications in terms of strength for concrete, reinforcement and other materials applied in accordance with Japanese Standards and Specifications or equivalent.

Table 5.3.11 Concrete Strength

Designation	Minimum Strength (N/mm^2)
PC Box Girder	36.0
Abutment & Pier	21.0
Concrete Pile	30.0
Lean Concrete	18.0

Source: JICA Study Team

Table 5.3.12 Strength of Reinforcement

Designation	Yield Strength (N/mm^2)
Round Bar	$\sigma py > 235$
Deformed Bar (SD295)	$295 < \sigma py < 390$
Deformed Bar (SD345)	$345 < \sigma py < 440$

Source: JICA Study Team

Table 5.3.13 Properties of Pre-Stress Seven-Wire Strand

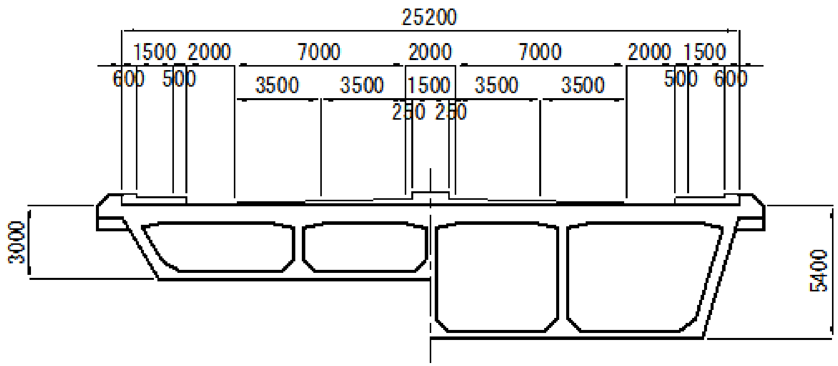
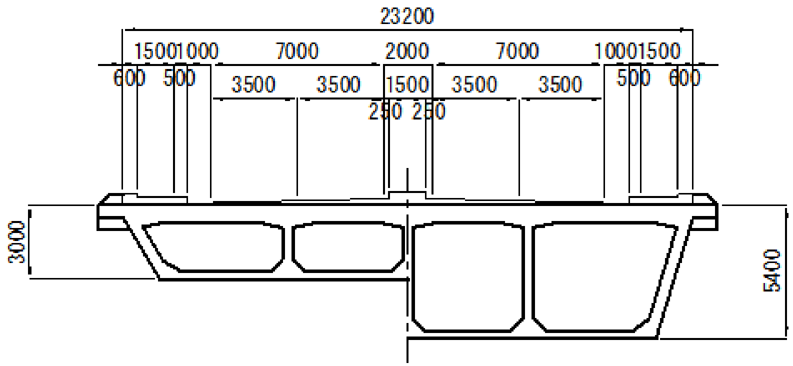
Designation	Nominal Diameter (mm)	Load at 0.2% Elongation (kN)	Specified Characteristic Breaking Load (kN)	Max. Elongation at Max. Load (%)	Nominal Steel Area (m m^2)	Mass (kg/m)
SWPR7AN SWPR7AL	9.3	75.5	88.8	3.5	51.61	0.405
	10.8	102	120	3.5	69.68	0.546
	12.4	136	160	3.5	92.90	0.729
	15.2	204	240	3.5	138.7	1.101
SWPR7BN SWPR7BL	9.5	86.8	102	3.5	54.84	0.432
	11.1	118	138	3.5	74.19	0.580
	12.7	156	183	3.5	98.71	0.774
	15.2	222	261	3.5	138.7	1.101

Note: N & L of designation means that N is normal product, L is low relaxation product.
Source: Specifications for Highway Bridges, Japan

(5) Cross Section of Bridge

The total width of 23.2m of cross section of bridge is selected with minimum 1.0m width of shoulder.

Table 5.3.14 Cross Section Option for New Kikafu Bridge

Option	Sketch	Remarks
Cross section with full width of Shoulder (2.0m)		Bridge Area: 14,112m2 Cost : 1.08 Keep a full shoulder width (2.0m) from the standard cross section that provides good driver's view and comfort.
Cross section with half width of Shoulder (1.0m)		Bridge Area: 12,992m2 Cost: 1.00 Half shoulder width (1.0m) hinders slightly driver's visibility and comfort but provides reasonable cost reduction.

5.3.4 Optional Superstructure by Dualling Configuration

There are several types of composite PC girders that have been invented in Japan and other countries. The following discussion introduces the optional advanced superstructure for PC Extradosed Bridge. Unique composite structures, such as using corrugated steel panel and steel pipe truss, etc. the web of PC girder, could be applicable for the new Kikafu Bridge however those are eligible when the cost of foundation and substructure can be effectively deducted by reducing the self-weight of superstructure. Area around the new Kikafu Bridge requires only seismic design for common earthquake force (Level-1) and the bridge has normal spread foundation. As a practical aspect, the conventional PC box girder would be eligible for the new Kikafu Bridge taking into account the site condition and other factors given in the comparative studies.

For the dualling configuration, the integrated four lanes section (Alt-A) would be suitable for dualling configuration at the economic factor (minimum initial cost) however the dual separate two lane section (Alt-B) would be desirable considering the traffic safety factor.

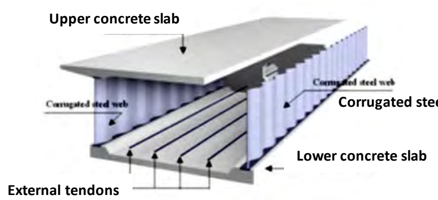
Table 5.3.15 Summary of Optional Superstructure with Dualling Configuration

Opt.	Struc -ture	Section xLane	Cost (Mil.USD)	Econo -mic	Struc -ture	Work -ability	Mainte -nability	Traffic safety	Land -scape	Assese -ment
Alt-1A	PC Box	1x4	49.0	A+ (0.20)	A (0.11)	B (0.08)	A+ (0.14)	B (0.19)	A (0.08)	A- (0.80)
Alt-1B		2x2	58.9	B- (0.10)	A+ (0.12)	A (0.12)	A (0.12)	A (0.29)	B (0.05)	A- (0.80)
Alt-2A	Corrugated. Steel	1x4	51.0	A (0.18)	B+ (0.08)	B+ (0.09)	C+ (0.05)	B (0.19)	A (0.08)	B (0.68)
Alt-2B		2x2	60.9	B- (0.10)	A (0.11)	A+ (0.13)	C (0.04)	A (0.29)	B (0.05)	B+ (0.72)
Alt-3A	Composite Truss	1x4	53.0	A- (0.16)	B+ (0.08)	B+ (0.09)	C+ (0.05)	B (0.19)	A (0.08)	B (0.66)
Alt-3B		2x2	63.3	C+ (0.08)	A (0.11)	A+ (0.13)	C (0.04)	A (0.29)	B (0.05)	B+ (0.70)

Source: JICA Study Team

On 23rd May, 2016, a tripartite meeting was held involving TANROADS, JICA and Study Team and confirmed PC box and integrated four lanes (Alt-1A) as an optimum superstructure type and agreed, considering maintainability and initial investment cost.

1) PC bridge with Corrugated Steel Web

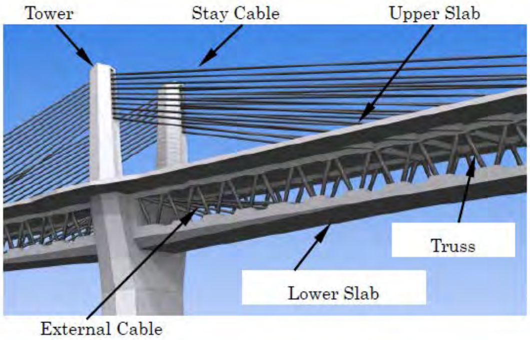


Typical cross section of PC bridge with corrugated steel web

PC box girder which is formed with light-weight corrugated steel plates instead of concrete webs. The corrugated steel plate webs are capable of withstanding shear forces without absorbing unwanted axial stresses due to prestressing, thus enabling efficient prestressing of top and bottom concrete deck slabs, thus resulting in an "accordion effect" (See Image below). Moreover, the corrugated webs also provide high shear buckling resistance. Use of light-weight corrugated steel plates for webs causes a reduction of self-weight around 10 to 15% comparing to conventional PC box girder. Therefore, this type of bridge enables the use of longer spans and reduction of construction cost. The weight of a segment to be cantilevered during erection can also be reduced, thus longer erection segments can be adopted and construction period can be shortened. This also eliminates assembly of reinforcement, cable arrangement and concrete placement for concrete webs. Thus, saving of construction manpower, quality enhancement and improvement of durability are attained. In addition, replacing the damaged deck slabs is easier than that of ordinary PC bridges.

2) PC Bridge with Composite Trussed Web

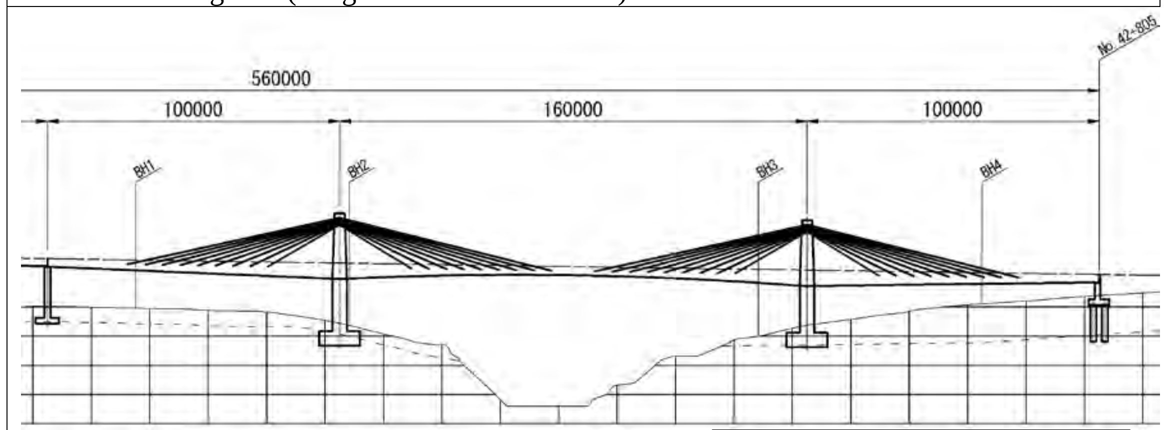
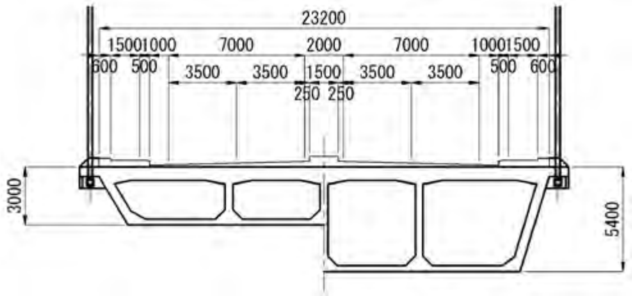
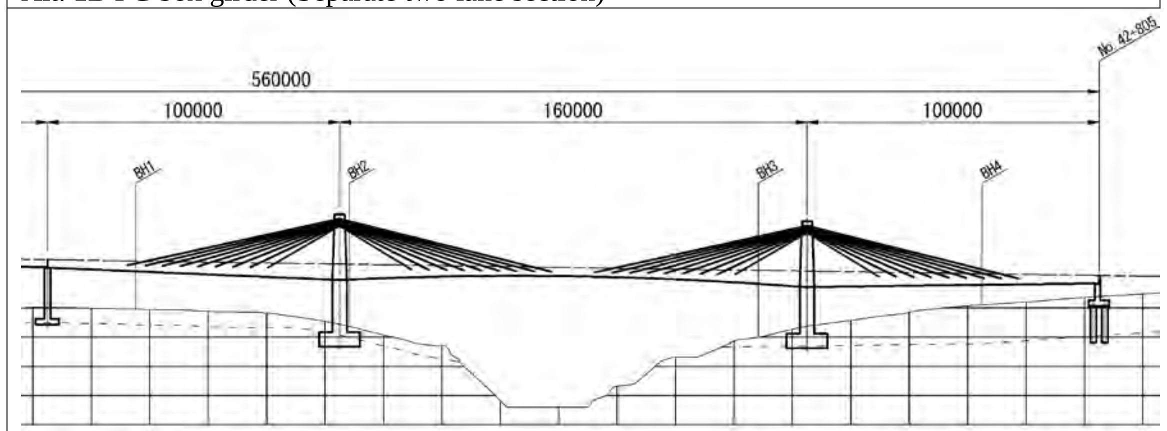
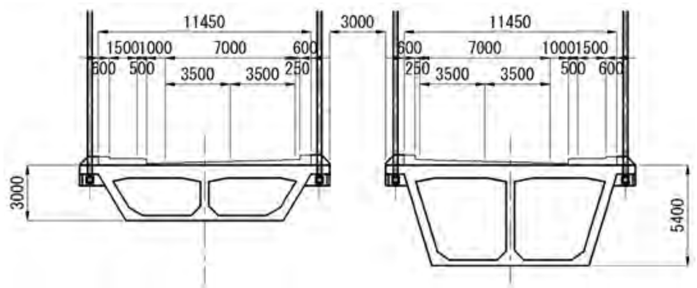
A composite truss bridge, which is called as Hybridge structure” has been developed that combines concrete upper and lower slabs together with steel trussed web. This structure rationalizes structural performances with reducing self-weight and shortening work sequences (not requiring rebar installation and form work, etc.). A structure is proposed to simplify the cantilever construction, and to utilize both steel and concrete effectively to realize a new composite structure. The combination of the advanced techniques has enhanced seismic resistance and saved the construction cost. There is a PC Extradosed bridge utilizing composite truss techniques replacing the web of box girder with a steel pipe truss as shown in the photo (See below).



General View of Composite Trussed Web

Fudo Bridge, Gunma Pref. Japan

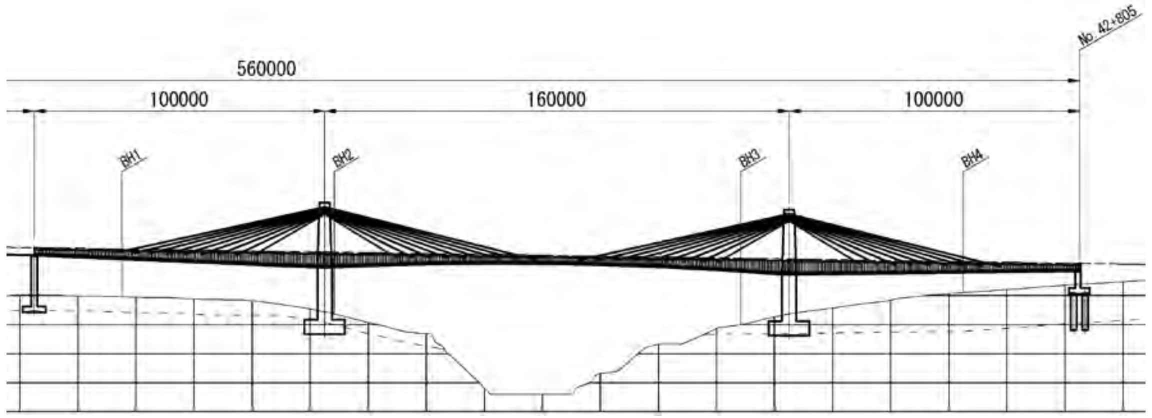
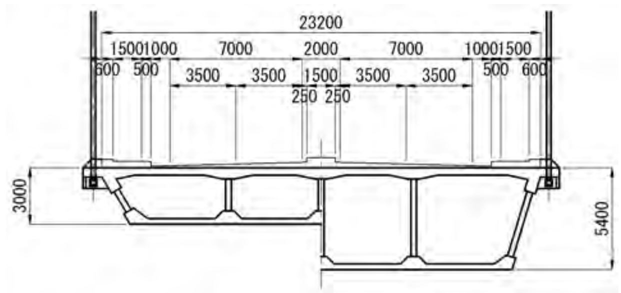
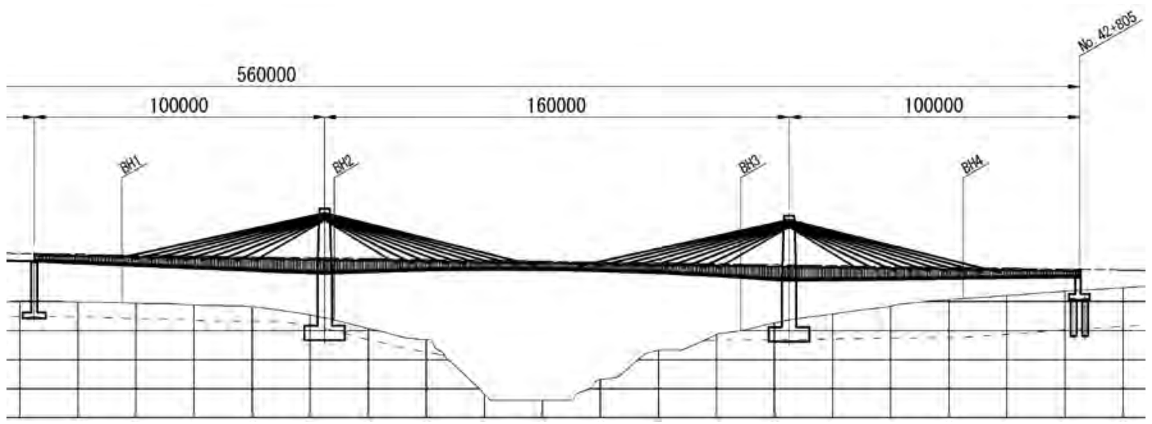
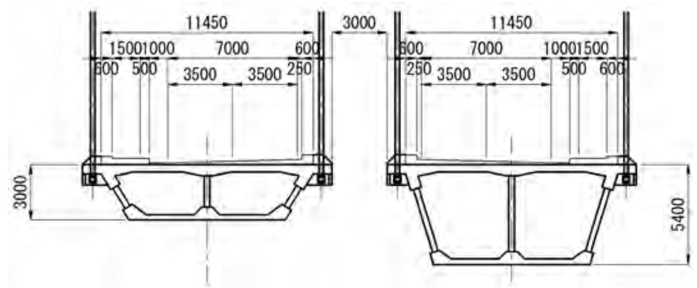
Table 5.3.16 Comparative Study for Optional Superstructure and Cross Section (1/3)

	Side view	Cross Section	Criteria		Assessments	Result[*1]
PC Extradose d Bridge (PC Box Girder)	Alt. 1A PC box girder (Integrated four lane section)  Construction time: 3.1 y		Cost (Mil. USD)	Super.	26.9	A+
				Sub.	10.8	
				Temp.	11.3	
				Total	49.0	
			Structure		Economic span length around 150m. Efficiently using deviated outer cables by lower height pylon enable the bridge realize longer span more than normal PC girder bridge. High durable and practical bridge structure in the world	A
			Workability		Cantilever erection with form traveler is required for quality control of high strength concrete which cast-in-situ concrete implicates longer cycle time for superstructure.	B
			Maintainability		Free maintenance for PC structure except that of the outer cable (stay cables).	A+
			Traffic safety		No alternative diversion can be provided if a traffic accident is happened on the bridge that cause damages of traffic flow.	B
			Landscape		Symbolic aesthetic view can be provided	A
	Alt. 1B PC box girder (Separate two lane section)  Construction time[*2]: 2.9y, (1.0)y+2.9 y= 3.9y		Cost (Mil. USD)	Super.	30.1	B-
				Sub.	14.6	
				Temp.	14.2	
				Total	58.9	
			Structure		Easier construction manner with simple shape of pylon more than that of four lanes however increase of cost appx. 20% against that of four lanes.	A+
			Workability		Cantilever erection with form traveler needs quality control of high strength concrete which cast-in-situ concrete implicates longer cycle time for superstructure. Latter bridge needs to consider necessary measure for adjacent work to the forth construed bridge.	A
			Maintainability		Free maintenance for PC structure except that of the outer cable (stay cables). Need more maintenance for separated sections	A
			Traffic safety		Another bridge can be alternative for traffic diversion at closure of either bridge (however requiring control of traffic speed on narrow bridge)	A
			Landscape		Symbolic aesthetic view can be provided however duplicated stay cables by extended from four towers annoying view of background (Kilimanjaro)	B

Note1: Above score indicates A+ (Most Superior) to C- (Most Inferior). Note2: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A+=1.0 to C-=0.2). The higher the score, the better the performance of the bridge type. Note2: The left figure indicates as construction time of first two lane bridge and right figure as that of total construction time including second two lane bridge.

Source: JICA Study Team

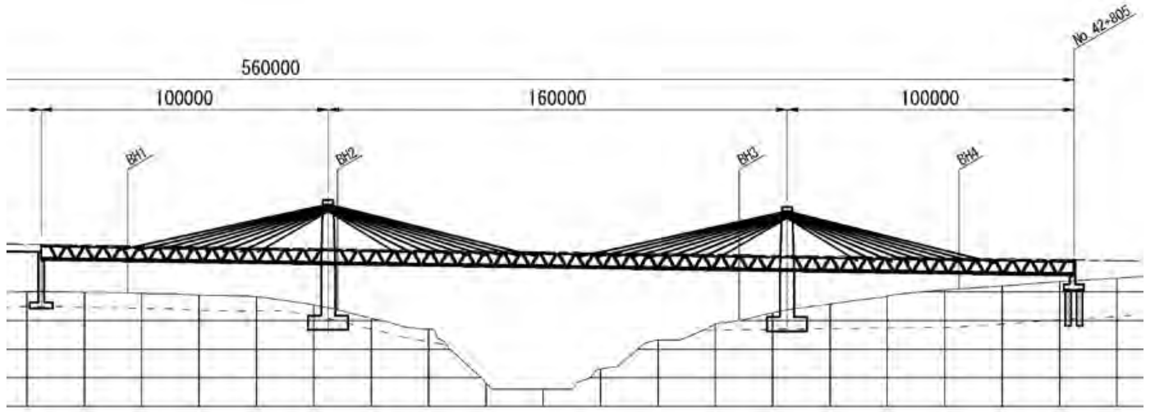
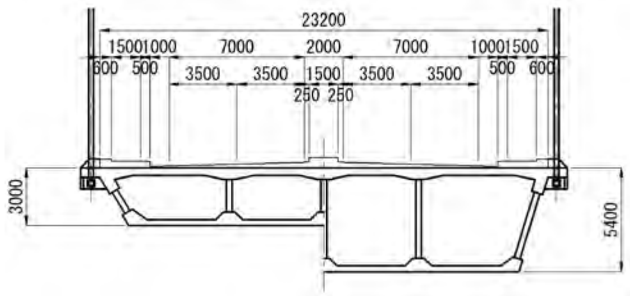
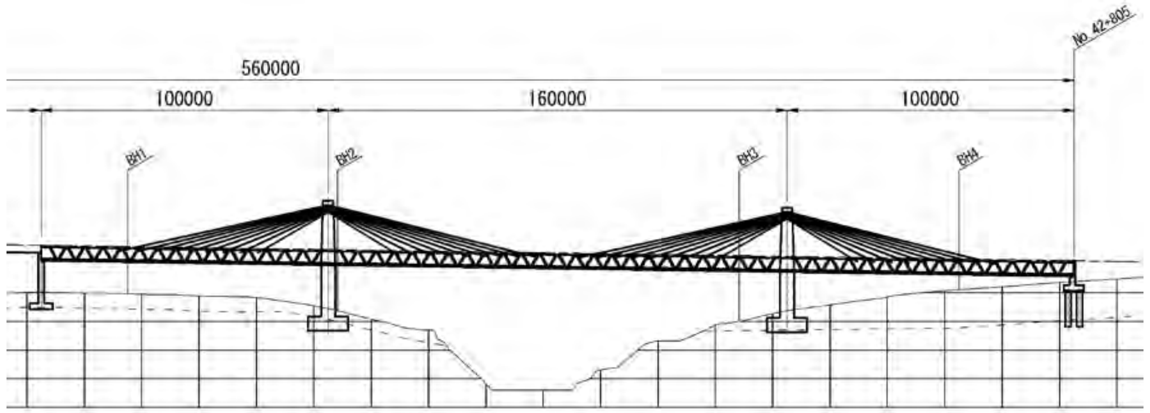
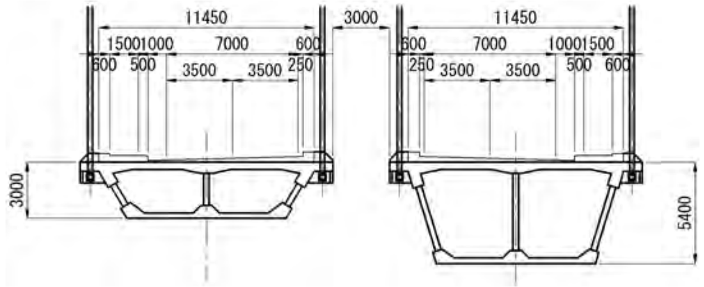
Table 5.3.17 Comparative Study for Optional Superstructure and Cross Section (2/3)

	Side view	Cross Section	Criteria		Assessments	Result[*1]
PC Extradosed(Corrugated steel Web) Bridge	Alt. 2A Corrugated steel web girder (Integrated four lane section)  Construction time: 3.0 y		Cost (Mil. USD)	Super.	30.6	B
				Sub.	8.9	
				Temp.	11.5	
				Total	51.0	
			Structure		Nearly 10 to 15% decrease of self-weight could be achieved by using corrugated steel web for superstructure. Efficient pre-stressing enable the top/ bottom slab to ensure higher sharing capacity. Using steel diaphragms correspond to wider box section to relieve the bending stress of bottom slab.	B+
			Workability		Cantilever erection minimizing the cast-in situ concrete work by using corrugated steel web eliminates the installation for rebar, pc cables and form work however subject to connection of corrugated steel panel.	B+
			Maintainability		Higher durability by using weathering steel however requiring stay cables and measure to keep watertight between web and bottom slab	C+
			Traffic safety		No alternative diversion can be provided if a traffic accident is happened on the bridge that cause damages of traffic flow.	B
			Landscape		Symbolic aesthetic view can be provided	A
	Alt. 2B Corrugated steel web (Separate two lane section)  Construction time[*2]: 2.8y, (1.0)y+2.8 y= 3.8y		Cost (Mil. USD)	Super.	34.2	B-
				Sub.	12.0	
				Temp.	14.6	
				Total	60.9	
			Structure		Nearly 10 to 15% of self-weight could be decreased by using corrugated steel web for superstructure. Efficient pre-stressing enable the top/ bottom slab to ensure higher sharing capacity. however increase approx..15% of cost for provision of dualling separate sections.	A
			Workability		Cantilever erection minimizing the concrete work by using corrugated steel for web of girder eliminates installation of rebar, PC cables and form but subject to connection of corrugated steel. Latter bridge requires necessary measure for adjacent work to the forth existing bridge.	A+
			Maintainability		High durability by using weathering steel except requiring maintenance for stay cables and measure to keep watertight between web and bottom slab. Separate section requires additional maintenance.	C
			Traffic safety		Another bridge can be alternative for traffic diversion at closure of either bridge (however requiring control of traffic speed on narrow bridge)	A
			Landscape		Symbolic aesthetic view can be provided however duplicated stay cables extended from four towers annoying view of background (Kilimanjaro)	B

Note1: Above score indicates A+ (Most Superior) to C- (Most Inferior). Note2: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A+=1.0 to C-=0.2). The higher the score, the better the performance of the bridge type. Note2: The left figure indicates as construction time of first two lane bridge and right figure as that of total construction time including second two lane bridge.

Source: JICA Study Team

Table 5.3.18 Comparative Study for Optional Superstructure and Cross Section (3/3)

	Side view	Cross Section	Criteria		Assessments	Result*1
PC Extradosed (Composite trussed Web) Bridge	Alt. 3A Composite steel trussed web (Integrated four lane section)  Construction time: 3.0 y		Cost (Mil. USD)	Super.	31.6	A-
				Sub.	9.3	
				Temp.	12.1	
				Total	53.0	
			Structure		Nearly 15 to 20% of self-weight can be decreased by using steel pipe trussed web for superstructure. Efficiently reducing the self-weight that keeps minimizing increase of weight to variable depth of girder by trussed web structure. Only one practice in Japan for this type of ED bridge.	B+
			Workability		Cantilever erection minimizing the cast-in situ concrete work by using corrugated steel for web of girder eliminates the installation for rebar, PC cables and form work however subject to installation of steel pipe trussed web requiring certain higher techniques.	B+
			Maintainability		High durability by using weathering steel except requiring maintenance of stay cables and care for connection between web (steel pipe) and bottom slab. Separate section requires additional maintenance.	C+
			Traffic safety		No alternative diversion can be provided if a traffic accident is happened on the bridge that cause damages of traffic flow.	B
			Landscape		Symbolic aesthetic view can be provided with see-through web.	A
	Alt. 3B Composite steel trussed web (Separate two lane section)  Construction time*2: 2.8y, (1.0)y+2.8 y=3.8y		Cost (Mil. USD)	Super.	35.4	C+
				Sub.	12.6	
				Temp.	15.3	
				Total	63.3	
			Structure		Nearly 15 to 20% of self weight could be decreased by using steel pipe trussed web for superstructure as mentioned above however increase approx..15% of cost for provision of dualling separate sections.	A
			Workability		Cantilever erection minimizing the cast-in situ concrete work by using steel pipe truss for web of girder eliminates the installation for rebar, PC cables and form work however subject to installation of steel pipe trussed web requiring certain higher techniques.	A+
			Maintainability		High durability by using weathering steel except requiring maintenance for stay cables and care for connection between web (steel pipe) and bottom slab. Separate section requires additional maintenance.	C
			Traffic safety		Another bridge can be alternative for traffic diversion at closure of either bridge (however requiring control of traffic speed on narrow bridge).	A
			Landscape		Symbolic aesthetic view can be provided however duplicated stay cables extended from four towers annoying view of background (Kilimanjaro).	B

Note1: Above score indicates A+ (Most Superior) to C- (Most Inferior). Note2: The score is calculated by multiplying weight of each evaluation factor, estimated by AHP questionnaire survey, and average of ranking of evaluation factors (A+=1.0 to C-=0.2). The higher the score, the better the performance of the bridge type. Note2: The left figure indicates as construction time of first two lane bridge and right figure as that of total construction time including second two lane bridge.

Source: JICA Study Team

5.4 Preliminary Study on Safety Measures and Roadside Station at Kikafu Bridge

As discussed in the previous section, a number of traffic accidents occur at/around existing Kikafu Bridge, which contributes to many fatalities and injuries caused by the accident and therefore adverse economic and environmental impacts.

There is high potential that the newly built Roadside Station be a tourist attraction place since both Arusha and Moshi have a number of tourist destinations that includes National Game Parks and Mount Kilimanjaro therefore, and a new Roadside Station can provide a rest place for those tourists with a good scenery of Kikafu Bridge with the background of Mount Kilimanjaro.

A provision of quality infrastructure is one amongst current Japanese ODA policy and therefore installation of safety measure(s) and Roadside Station at/around existing Kikafu Bridge is encouraged to be part of the Project.

5.4.1 Safety Measures at/around Existing Kikafu Bridge

(1) Current Condition

1) Geometrical Condition

Curve radii Arusha side is about 660m and gradient is about 5 %. Curve radii Moshi side is about 260m and gradient is about 6 %. Although the speed limit of 50 km/h is applied in/around the existing Kikafu Bridge, since approach road of existing Kikafu Bridge is straight and S-curve around Kikafu Bridge, these is a tendency that actual travel speed be more than 80 km/h. The existing alignment doesn't satisfy the standards with the speed of over 80 km/h.

Table 5.4.1 Geometrical Condition in/around Existing Kikafu Bridge

	Arusha side	Moshi side	Design Standards in Tanzania
Design Speed/ Travel Speed	80 – 100km/h	80 – 100km/h	80 – 100km/h
Curve radii	660m	**260m**	230m – **400m**
Gradient	**5%**	**6%**	**4% - 3%**

Source: JICA Study Team

2) Current Safety Measure

Guardrails and speed limit signs are installed in some section.

Source: JICA Study Team

Figure 5.4.1 Current Safety Measure at Kikafu

3) Current Traffic Safety Issues

From the observation and site visit made by the Study Team and TANROADS officers, the following traffic safety issues are identified by road section.

< Moshi side >

Travel speed tends to be over design speed because of small curve radii and steep gradient in the section.

➔ Suppression of travel speed

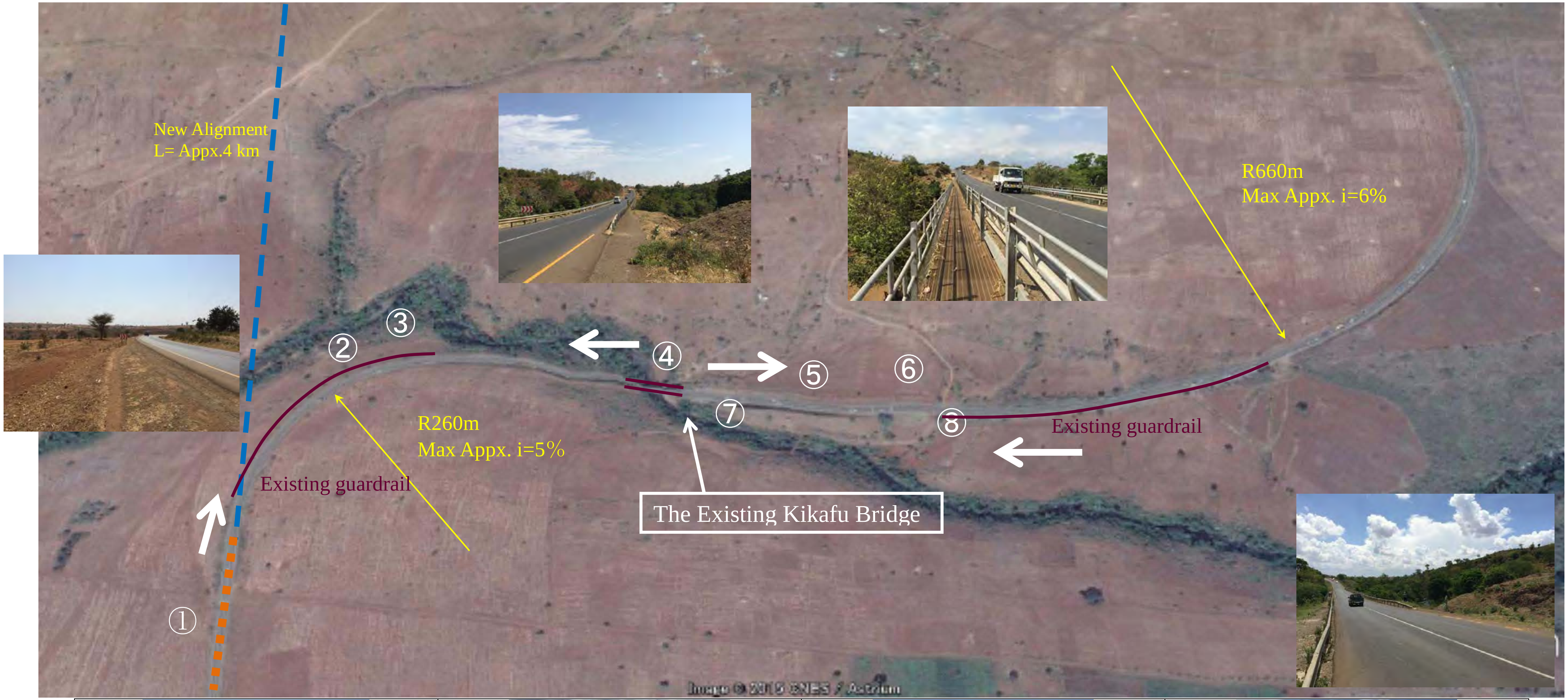
There is long distance steep slope and the bridge with narrow shoulder at the lowest point. Therefore, the bridge is located at the point where travel speed tends to be largest, and the risk of traffic accident tends to increase in the evening and the night time.

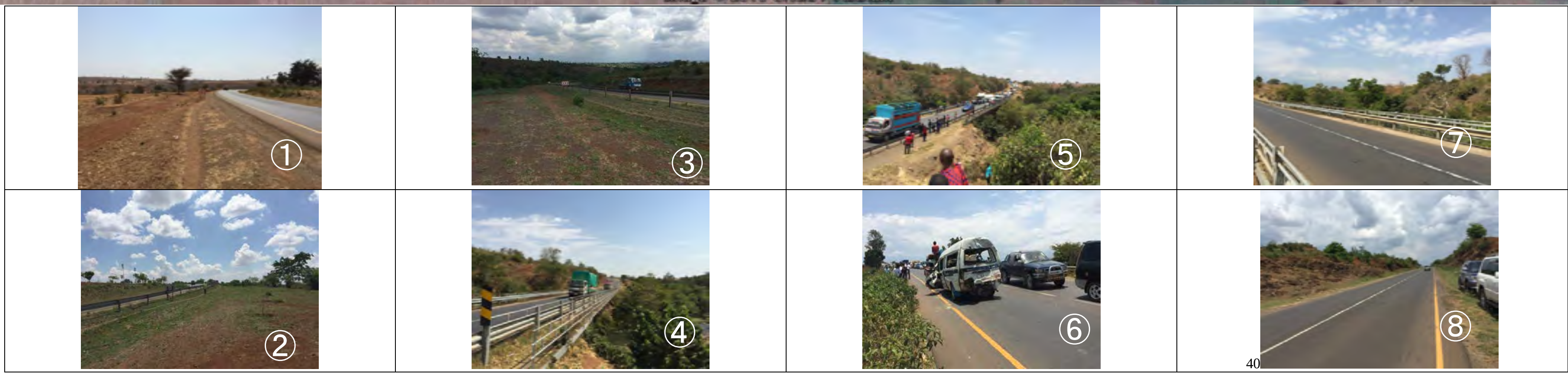
➔ Suppression of travel speed, improvement of visibility of bridge and widening of shoulder on bridge.

< Arusha side>

Travel speed of vehicles, especially trucks, to Arusha side decreases because of long distance steep slope. Therefore, overtaking traffic tends to occur, the risk of traffic accident become higher.

➔ Separation of the low speed vehicles (installation of the climbing lane) Suppression of travel speed, improvement of visibility of bridge and widening of shoulder on bridge.


New Alignment
L= Appx.4 km
R660m
Max Appx. i=6%
①
②
③
④
⑤
⑥
⑦
⑧
R260m
Max Appx. i=5%
Existing guardrail
Existing guardrail
The Existing Kikafu Bridge
Image © 2015 CNES / Astrium


①
③
⑤
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④
⑥
⑧

(2) Safety Countermeasure

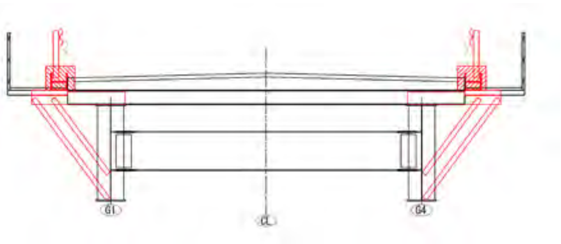
1) Proposed Countermeasures

Proposed safety countermeasures to traffic safety issues at/around existing Kikafu Bridge are listed below.

Table 5.4.2 List of Safety Countermeasures proposed at/around Existing Kikafu Bridge

Approach	Safety Countermeasures
Suppression of travel speed	➢ Warning Sign ➢ Regulatory sign ➢ Marking with Road Studs ➢ Channelizing Lame Marking ➢ V shape Lane Marking ➢ Curve Marker ➢ Delineator ➢ Rumble strip
Improvement of visibility of bridge	➢ Light ➢ Reflection sheet
Widening of shoulder on bridge	➢ Pavement of shoulder ➢ Setback of guardrail
Separation of the low speed vehicles	➢ Climbing Lane:

Source: JICA Study Team



Source: JICA Study Team

Figure 5.4.2 Image of Setback of Guardrail

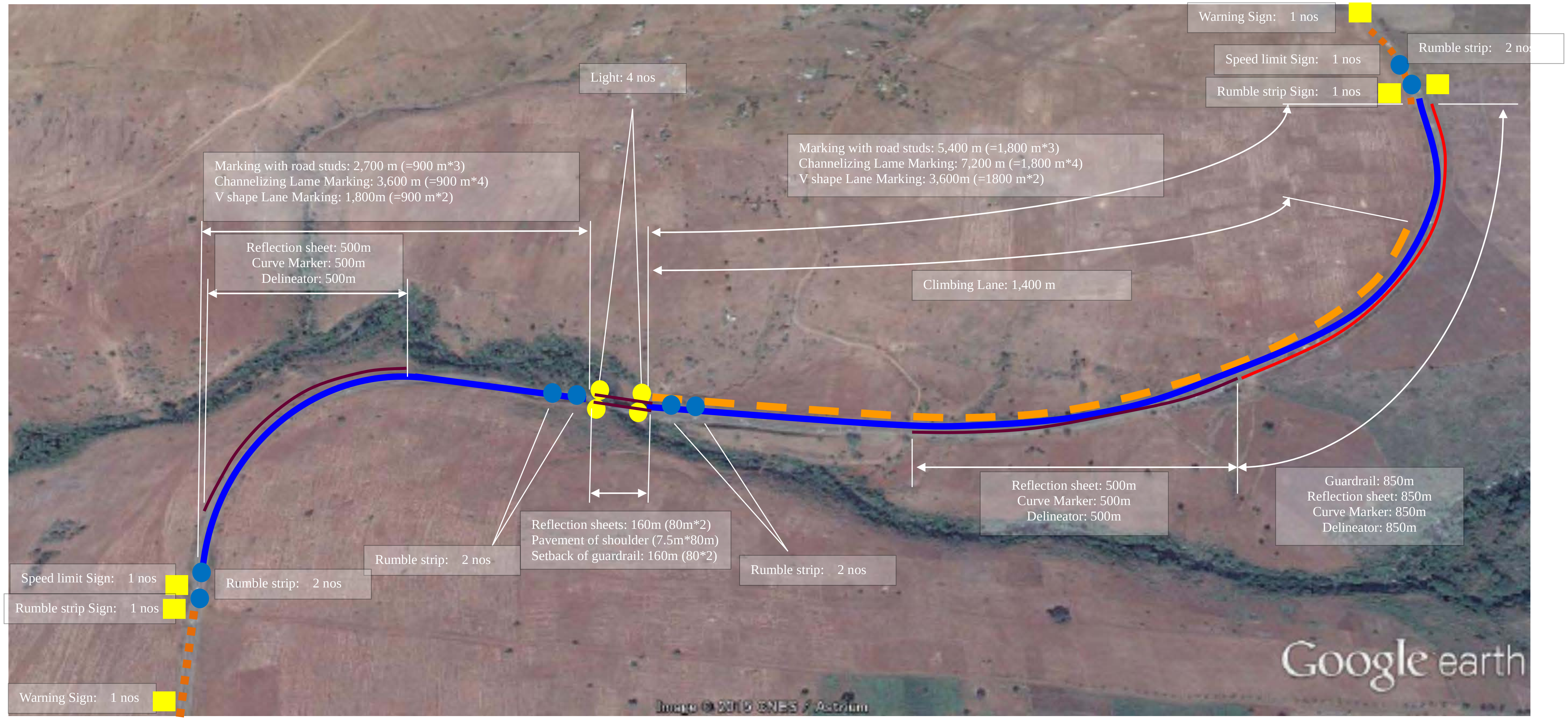
2) Preliminary Cost Estimate for Safety Countermeasures

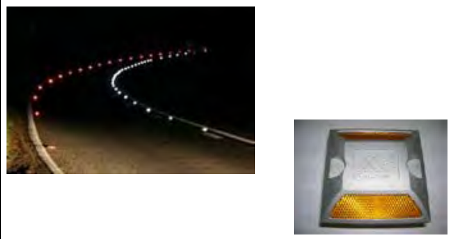
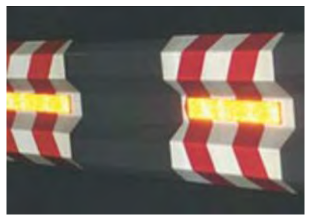
Direct cost for safety measures at/around existing Kikafu Bridge is summarized below and is estimated at around 1.5 million USD.

Table 5.4.3 Cost Estimates of Safety Measures at/around Existing Kikafu Bridge

Item			Unit	Quantity	Unit Cost USD(2016)	Total Amount (USD 2016)
Climing lane	Cut		m3	8,400	4.0	33,600
	Pavement		m2	5,950	48.0	285,600
	Drainage		m	1,400	150.0	210,000
Ancillary road works	Warning sign		No	4	198.6	794
	Speed limit Sign/Rumble strip sign		No	4	2,105.7	8,423
	Marking with road studs		-	-	-	-
	Makrking		m	8,100	1.5	12,150
	Road studs		No	8,100	13.4	108,540
	Channelizing Lane Marking	1.424m2/m	m2	15,379	10.4	159,942
	V shape Lane Marking	0.458m2/m	m2	2,473	10.4	25,719
	Guardrail		m	850	70.3	59,755
	Curve Marker	0.1 no/m	No	270	201.6	54,432
	Delineator	0.2 no/m	No	540	31.1	16,794
	Reflection sheets	0.8 no/m	No	928	95.1	88,253
	Rumble strip	8.5m*5/nos	m	340	12.7	4,318
	Light		No	4	6,000.0	24,000
Exisitng Kikaf Bridge	Surfacing		m3	30	103.5	3,105
	Excavating existing pavment		m3	28	12.7	356
	Setback of guardrail		m	160	2,500.0	400,000
Total Amount						**1,495,781**

Source: JICA Study Team

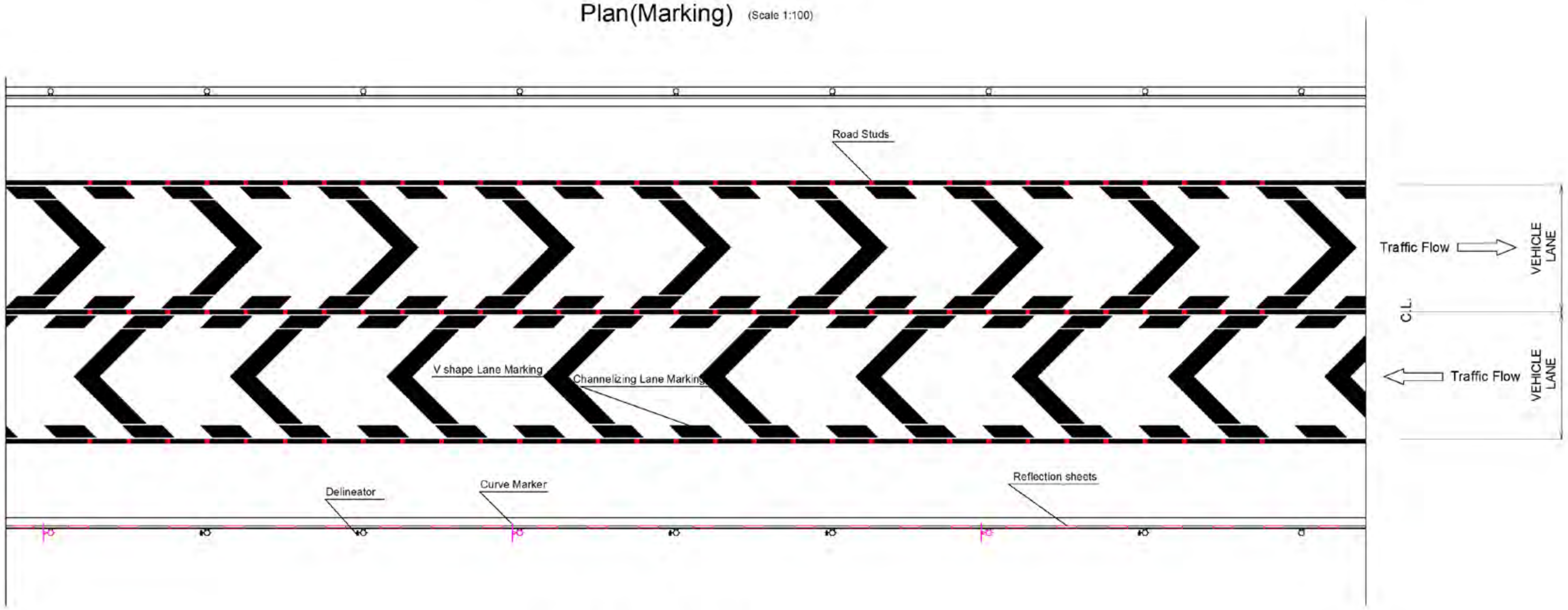


					
Warning sign	Speed limit Sign	Marking with Road Studs	Channelizing Lame Marking	V shape Lane Marking	Curve Marker
					
Delineator	Reflection sheet	Rumble strip	Light	Climbing Lane	Pavement of shoulder

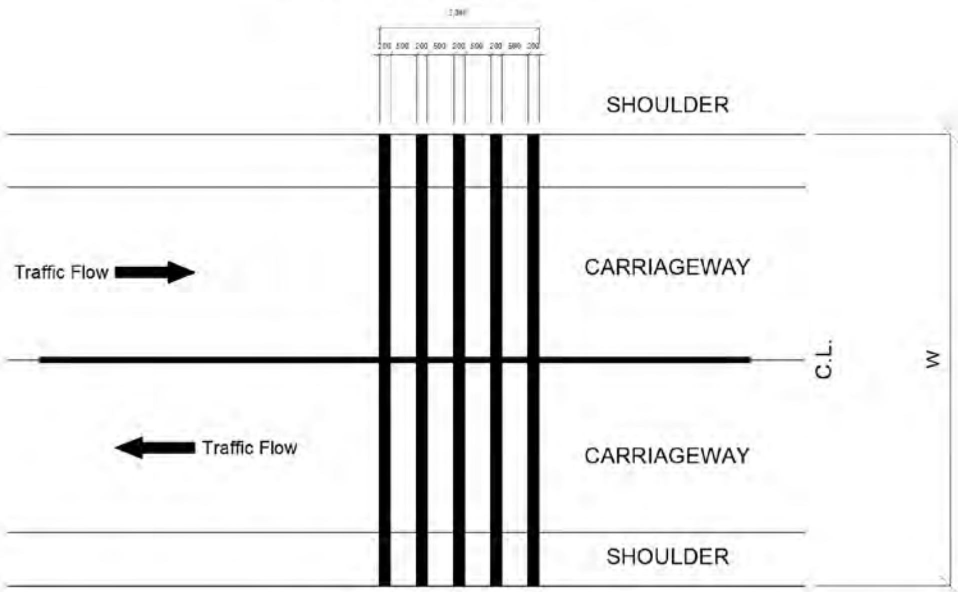
Source: JICA Study Team

SAFETY MEASURES

Plan(Marking) (Scale 1:100)



Plan (Rumble strip) (Scale 1:100)



TANROADS
TANZANIA NATIONAL ROADS AGENCY
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APPROVED:
PROJECT ENGINEER
CHIEF EXECUTIVE

JAPAN INTERNATIONAL COOPERATION AGENCY
CONSULTANT
International Development Center of Japan Inc.
Oriental Consultants Global CO., LTD.

Designed
Drawn
Checked
Ref.:SMRO

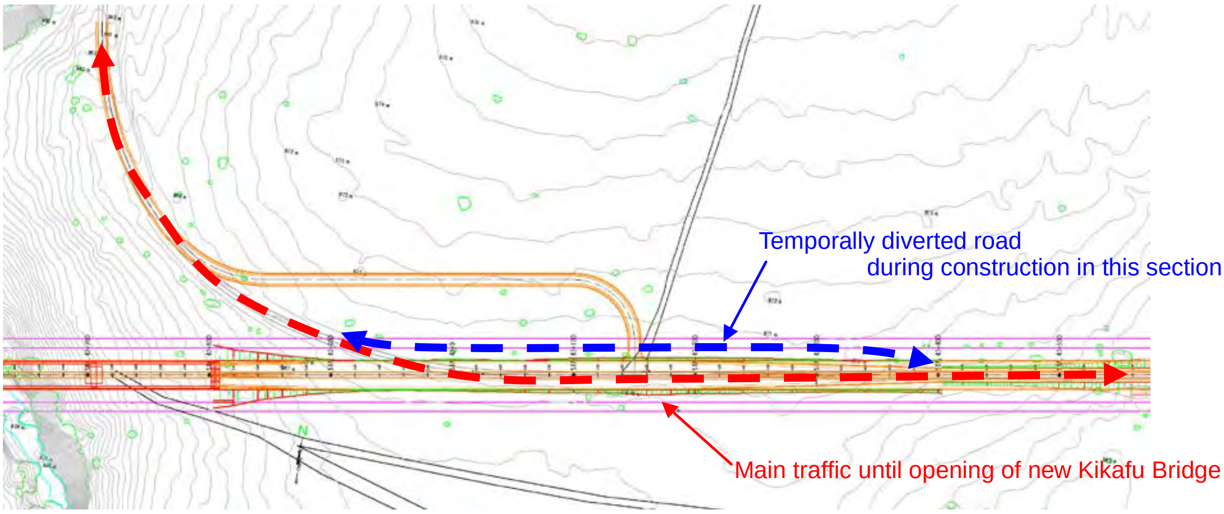
REVISIONS
DATE | DESCRIPTION

PROJECT
Supplemental Study for Implementation of Arusha-Holili Road Improvement Project

DWG TITLE: Safety Measures
SCALE: A3 1:20
DATE:
Dwg. No.
Rev

(3) Management during Construction

Existing roads and Kikafu Bridge are planned to function to accommodate main traffic until opening of new Kikafu Bridge. Accordingly, safety measures proposed above should be installed, considering the alignment of temporally diverted road during construction and new approach road after construction of new Kikafu Bridge.

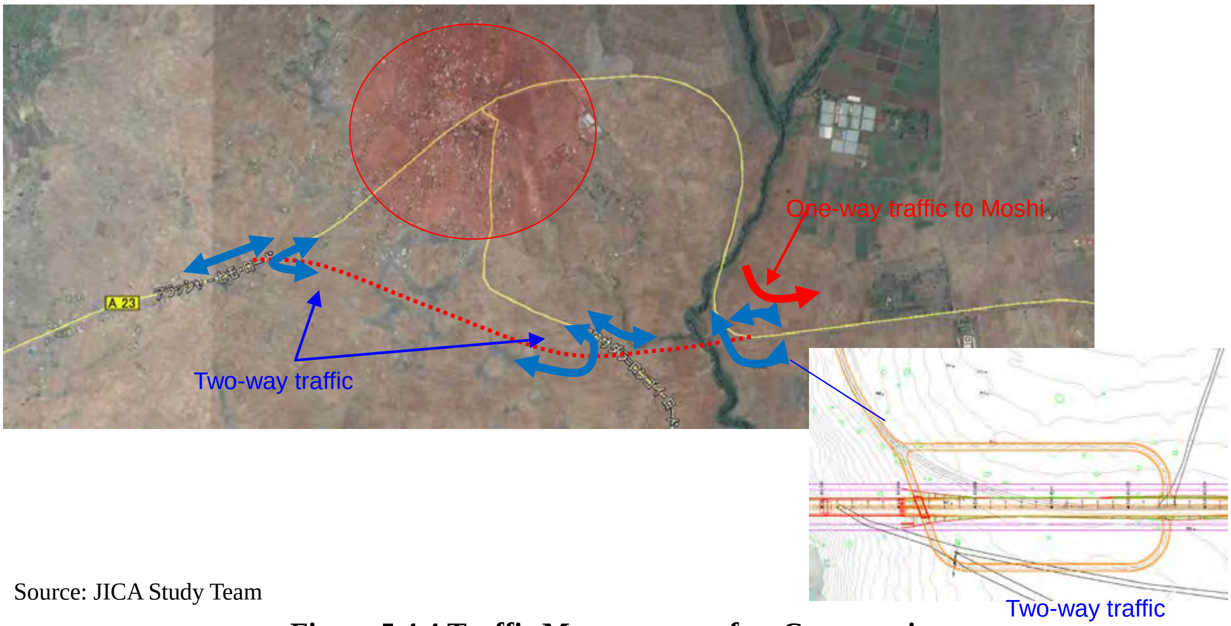


Source: JICA Study Team

Figure 5.4.3 Traffic Management during Construction

(4) Management after Construction

There is a community located in the west side of existing Kikafu Bridge. In order to provide local access and accommodate the traffic generated to/from the community, two intersections are planned to be installed, both located in west side of new/existing Kikafu Bridge. For the intersection in east side of new/existing Kikafu Bridge, there are two optional traffic management: (i) one-way treatment, allowing local traffic only to Moshi side in order to discourage usage of existing Kikafu Bridge and reduce collision between local traffic and main traffic and (ii) two-way treatment to provide a better access to/from said community which requires installation of box culvert under main road.



Source: JICA Study Team

Figure 5.4.4 Traffic Management after Construction

5.4.2 Roadside Station

(1) Proposed Location of Roadside Station

Roadside Station is suggested to be constructed at/around new Kikafu Bridge which is located between Arusha and Holili.

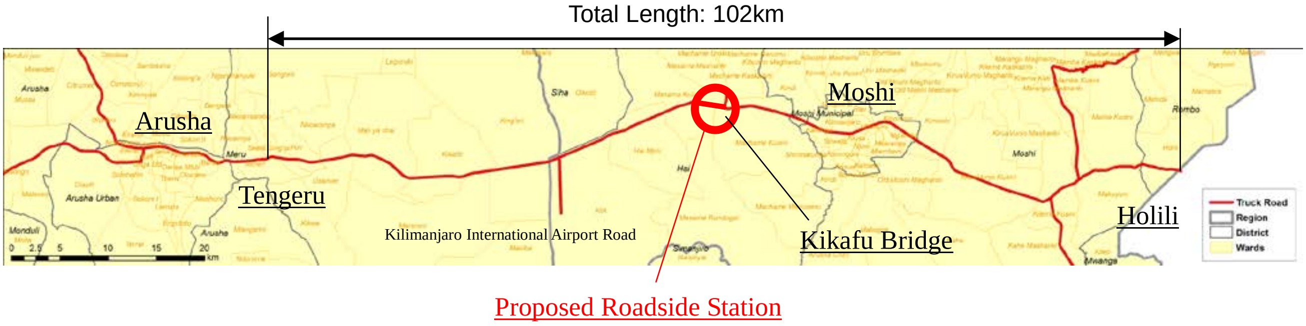


Figure5.4.5 Proposed Location of Roadside Station

(2) Function of Roadside Station

Roadside Station is expected to have various usage function as it is expected to be a public facility. However, such initiatives require involvement of the private sector, local communities and residents. Accordingly, the involvement of the private sector and locals might take considerable time and the step wise development is proposed for development of Roadside Station, as an initial development functions minimum, including parking and rest room.

< First Stage >
Parking
Rest Rooms

<Second Stage>
Restaurant
Shops
Information Board

<Future Stage>
Community room
Recreation area
Information centre

Figure5.4.6 Proposed Functions of Roadside Station

(3) Scale of Roadside Station

Future traffic volume will be about 25,000 PCU/day, therefore, a scale of roadside station in this Project is determined based on Michinoeki Development Guideline. In D/D stage, function and scale of roadside station will be decided based on regional needs.

< First Stage(Initial)>
Parking: 5,000m^2
(50 spaces: 10 large vehicles / 40 small vehicles based on the future traffic volume[1])
Restrooms:130 m^2 (14 stalls)
Green area: 7,500 m^2

< First Stage(Extended)>
Parking: Additional 5,000 m^2 (Additional 50 spaces (total 100 spaces): + 10 large vehicles (total 20 large vehicles / and + 40 small vehicles (total 80 for small vehicles) based on future traffic volume[1])

< Second Stage>
Buildings: 1,200 m^2
(Market 400 m^2, Restaurants 300 m^2, Processing center 400 m^2, Information space 100 m^2)

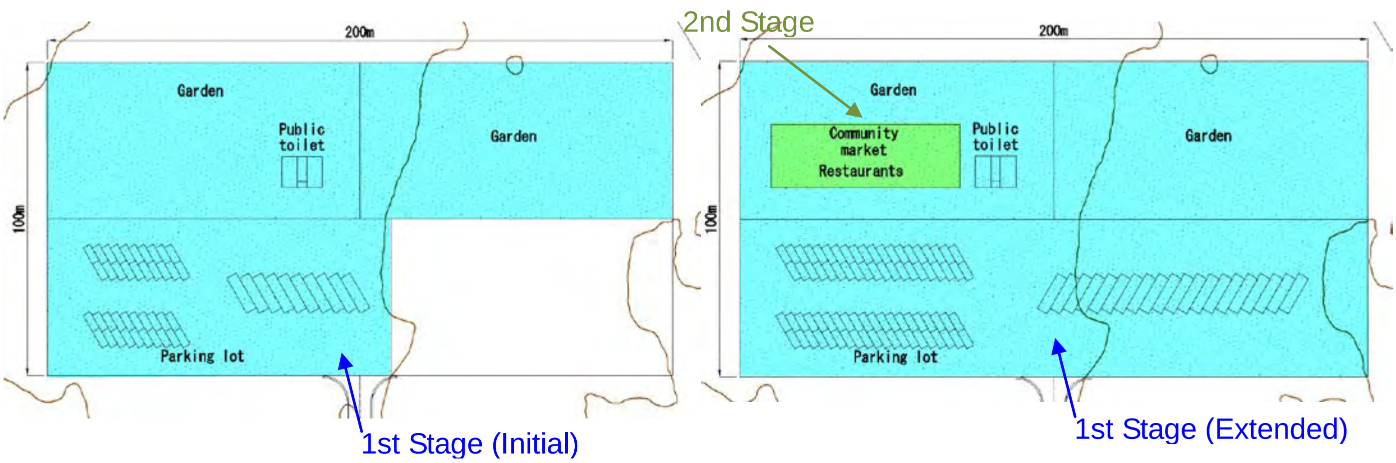


Figure 5.4.7 Stage Development of Roadside Station

Table 5.4.4 Facility Size Specification

Facility		Side	Comments
Parking lot		100 spaces 50 large vehicles/ 50 passenger cars Area : 5,000 m2	1 : 1 ratio assumed. The required area includes access routes.
Buildings	Restrooms	Male (urinals) : 10 Male (stalls) : 4 Female : 14 stalls	Male to female ratio is assumed to be 1 : 1.
	Direct sales (market)	200 - 400 m2	Depending on items processed and production vlume.
	Restaurants	200 - 300 m2 (70 - 100 seats)	General size
	Public servise facilities	50 - 100 m2	Depending on purpose of use and side of the local community.
	Meeting hall, etc. for residents		
	Exihibits, etc. for traverlers	300 - 1000 m2	Depending on content and Purpose.
Green planted areas		5,000 - 10,000 m2	Depending on amount of land available.

Remarks: These calculation assume a michinoeki along a major highway with traffic volume of about 25,000 vehicles per day, where the ratio of vechicles stopping is not particularly high.

Source: Guidelines for Roadside Stations ”Michinoeki”

[1] The traffic volume along new Kikafu Bridge is projected at 13,600 PCU in 2025 and 25,600 PCU per day in 2035.

(4) Cost Estimate of Roadside Station

Cost of Roadside Station at Kikafu is estimated using unit price by facilities. Unit price of each facility refers to those used for the pilot study in Kenya explored in "Guidelines for Roadside Stations "Michinoeki" and those in previous F/S and D/D considering price escalation. The following tables summarize cost estimates of Roadside Station, required for First Stage and Second Stage development of Roadside Station.

Table 5.4.5 Preliminary Cost Estimate of Roadside Station

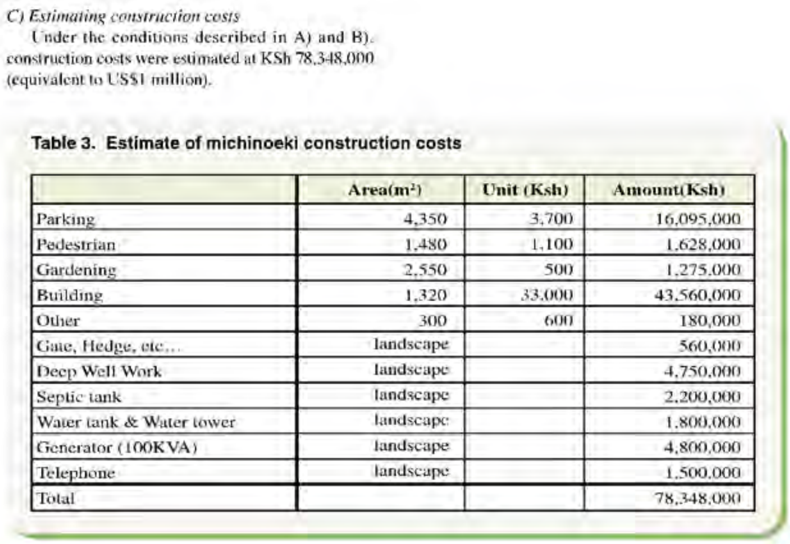
<1st Stage>

	Area(m2)	Unit(USD:2004)	Unit(USD:2016)	Amount(USD:2016)
Parking (Initial)	5,000	48	113	562,517
Pedestrian	1,170	15	35	41,134
Gardening	7,500	7	16	123,051
Building	130	422	989	128,582
Lighting	10		6,000	60,000
Accsess road	4,300		48	206,400
Other	20,000	10	23	468,764
Total				1,590,447
Total(round)				1,600,000
Parking (Extended)	5,000	48	113	562,517
Total(round)				2,200,000

<2nd Stage>

	Area(m2)	Unit(USD:2004)	Unit(USD:2016)	Amount(USD:2016)
Parking		48	113	0
Pedestrian		15	35	0
Gardening		7	16	0
Building	1,200	422	989	1,186,910
Lighting		0	6,000	0
Accsess road		0	48	0
Other	20,000	10	23	468,764
Total				1,655,674
Total(round)				1,700,000

Source: JICA Study Team



C) Estimating construction costs

Under the conditions described in A) and B), construction costs were estimated at KSh 78,348,000 (equivalent to US$1 million).

Table 3. Estimate of michinoeki construction costs

	Area(m²)	Unit (Ksh)	Amount(Ksh)
Parking	4,350	3,700	16,095,000
Pedestrian	1,480	1,100	1,628,000
Gardening	2,550	500	1,275,000
Building	1,320	33,000	43,560,000
Other	300	600	180,000
Gate, Hedge, etc…	landscape		560,000
Deep Well Work	landscape		4,750,000
Septic tank	landscape		2,200,000
Water tank & Water tower	landscape		1,800,000
Generator (100KVA)	landscape		4,800,000
Telephone	landscape		1,500,000
Total			78,348,000

Source: Guidelines for Roadside Stations "Michinoeki"

Figure5.4.8 Sample Cost Estimate of Roadside Station in Kenya

(5) Image of Roadside Station at Kikafu

< First Stage >

< Second Stage >

Source: JICA Study Team

Figure5.4.9 Image of Roadside Station at Kikafu

CHAPTER 6 CONSTRUCTION AND IMPLEMENTATION PLAN

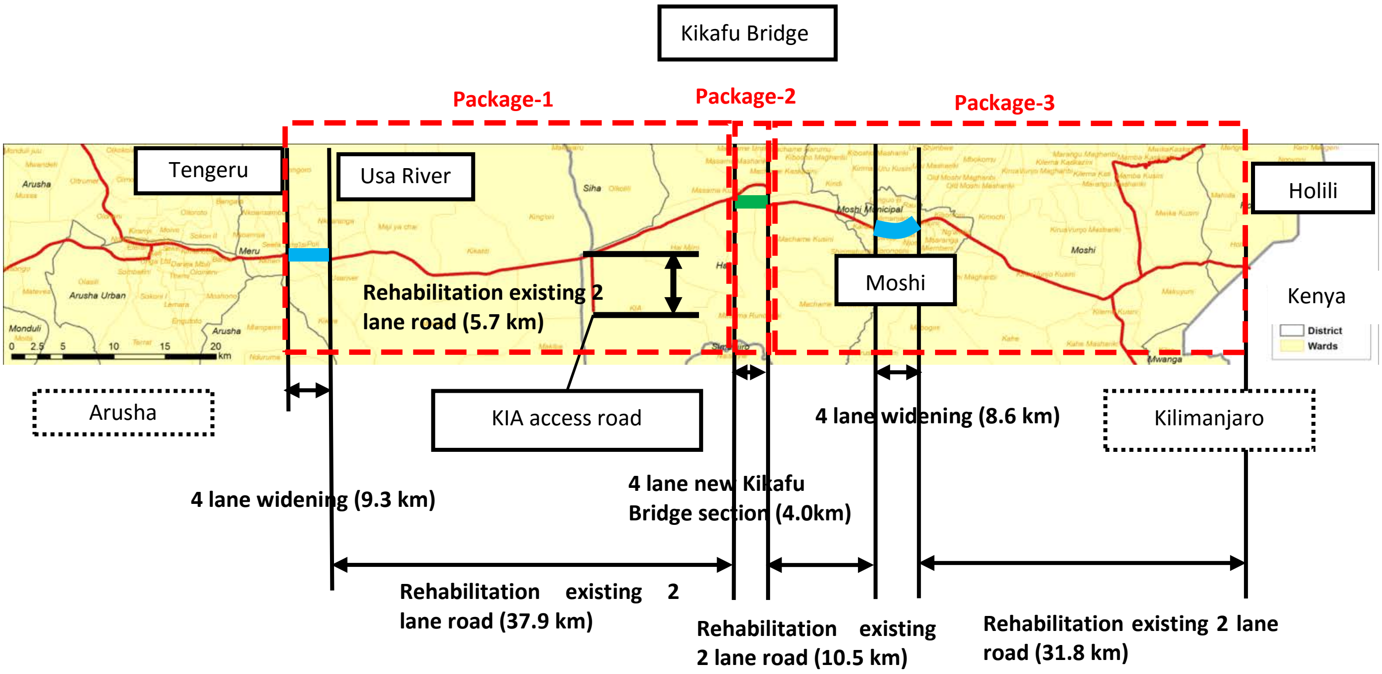
6.1 Formation of Construction Package (Contract Package)

The Project includes widening and rehabilitation of road section, totalling to 107.9 km (including four lane widening 21.9 km) and 560 m long Kikafu Bridge. Rehabilitation of existing road, especially widening section in urban areas, requires diversion of main traffic, ensuring traffic safety. 560 m long PC Extradoesed Kikafu Bridge itself requires complex construction method. Considering manageable size of the Project, the following three packages are proposed.

Package-1: 47.23km Tengeru - West Kikafu and 5.75km KIA Access Road

Package-2: 4.0km West Kikafu –East Kikafu including 560m Kikafu Bridge, and Roadside Station

Package-3: 50.89km East Kikafu- Holili



Source: JICA Study Team

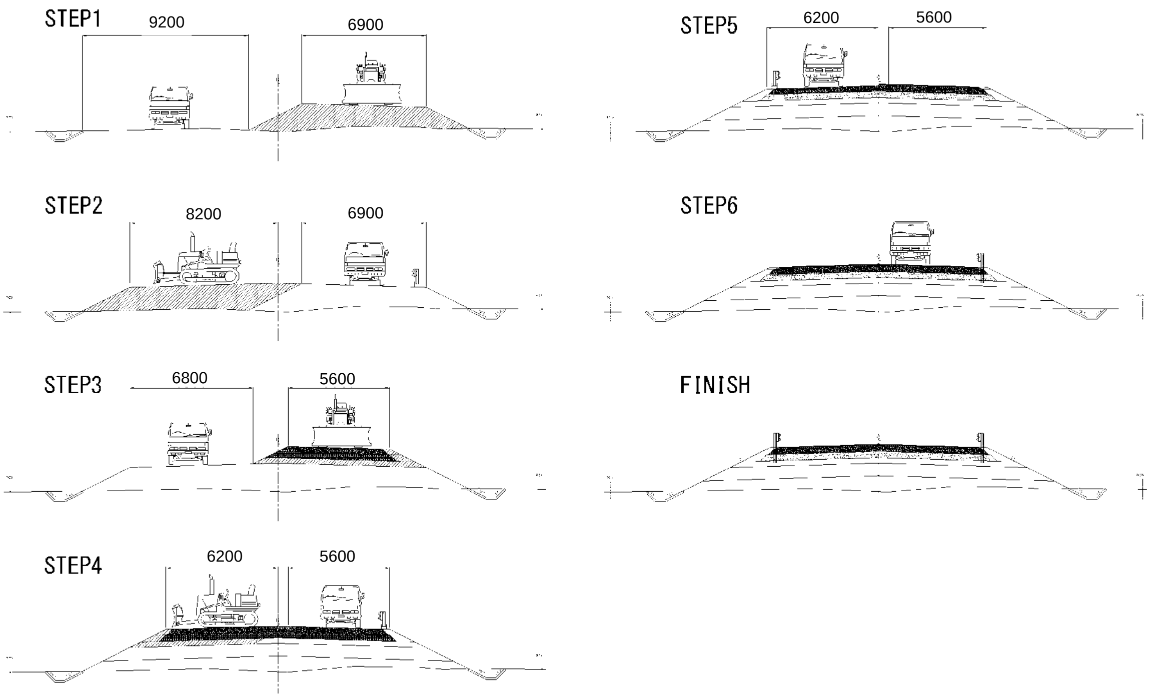
Figure 6.1.1 Proposed Construction Package of the Project

In addition, installation of safety measures at/around existing Kikafu Bridge is proposed as Package-4.

6.2 Construction Plan

6.2.1 Temporary Diversion of Existing Roads

The rehabilitation of existing road requires temporarily detour roads. The following figures show images of diversion in earth works section.



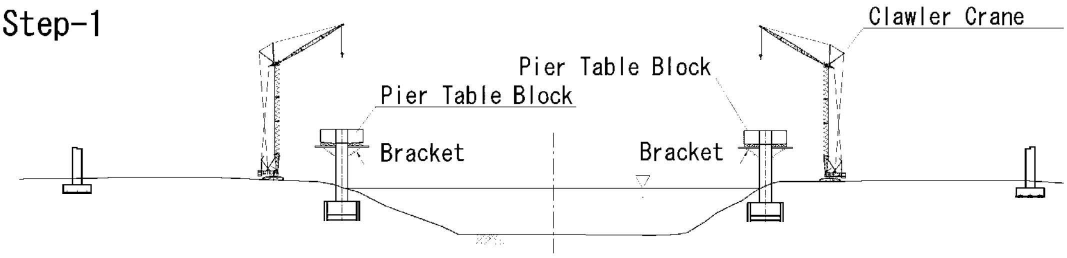
Source: JICA Study Team

Figure 6.2.1 Diversion of Existing Roads (Earth Works Section)

6.2.2 Construction Method of New Kikafu Bridge

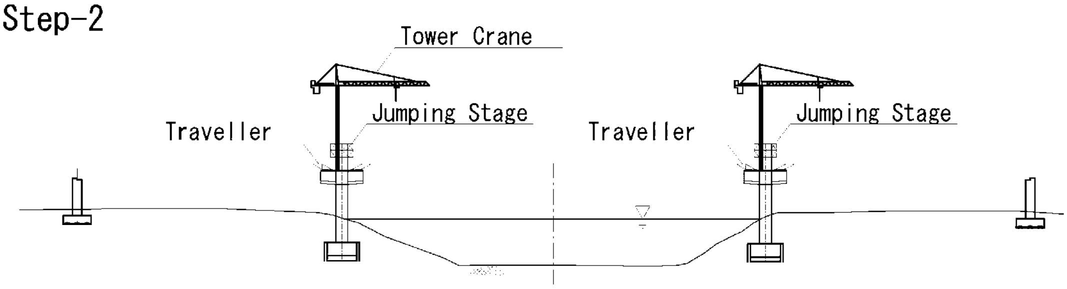
A cantilever method would be adopted for PC Extradosed Kikafu Bridge as a construction method, being erected using the balanced cantilever with sliding formwork. The following figures shows erection step for construction of Extradosed Bridge.

Step-1



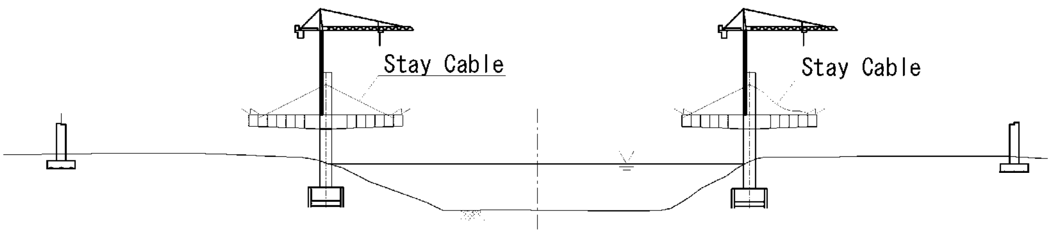
- Assemble bracket on the side of pier.
- Construct pier table block on the bracket by cast-in-situ method.

Step-2



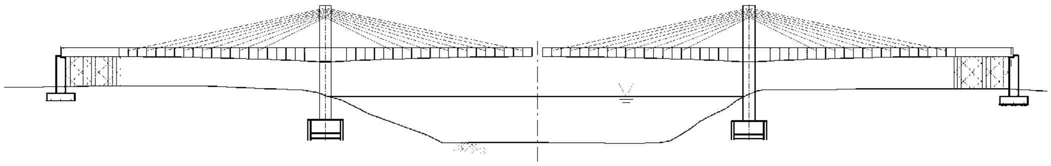
- Construct pylones by jumpinge stage and tower crane.
- Assemble travellers on the pier table

Step-3



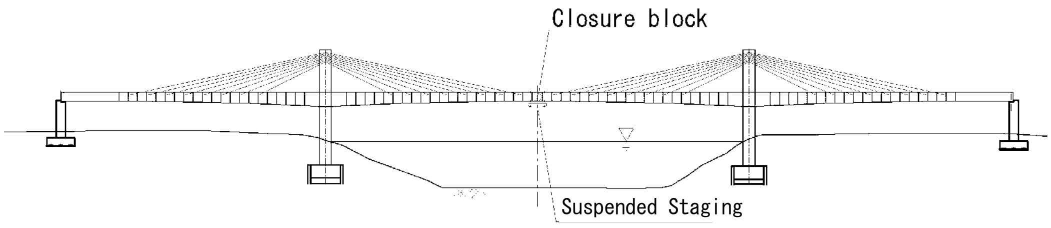
- Alternateing erect PC Box girder blocks with balanced cantilever method using sliding form.
- Set the first stay cable, and prestresing

Step-4



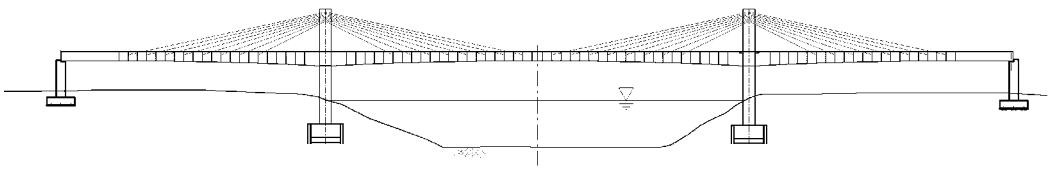
・The side of girder is casted by all staging method.

Step-5



・ Setting a closure block on suspended staging.

Step-6



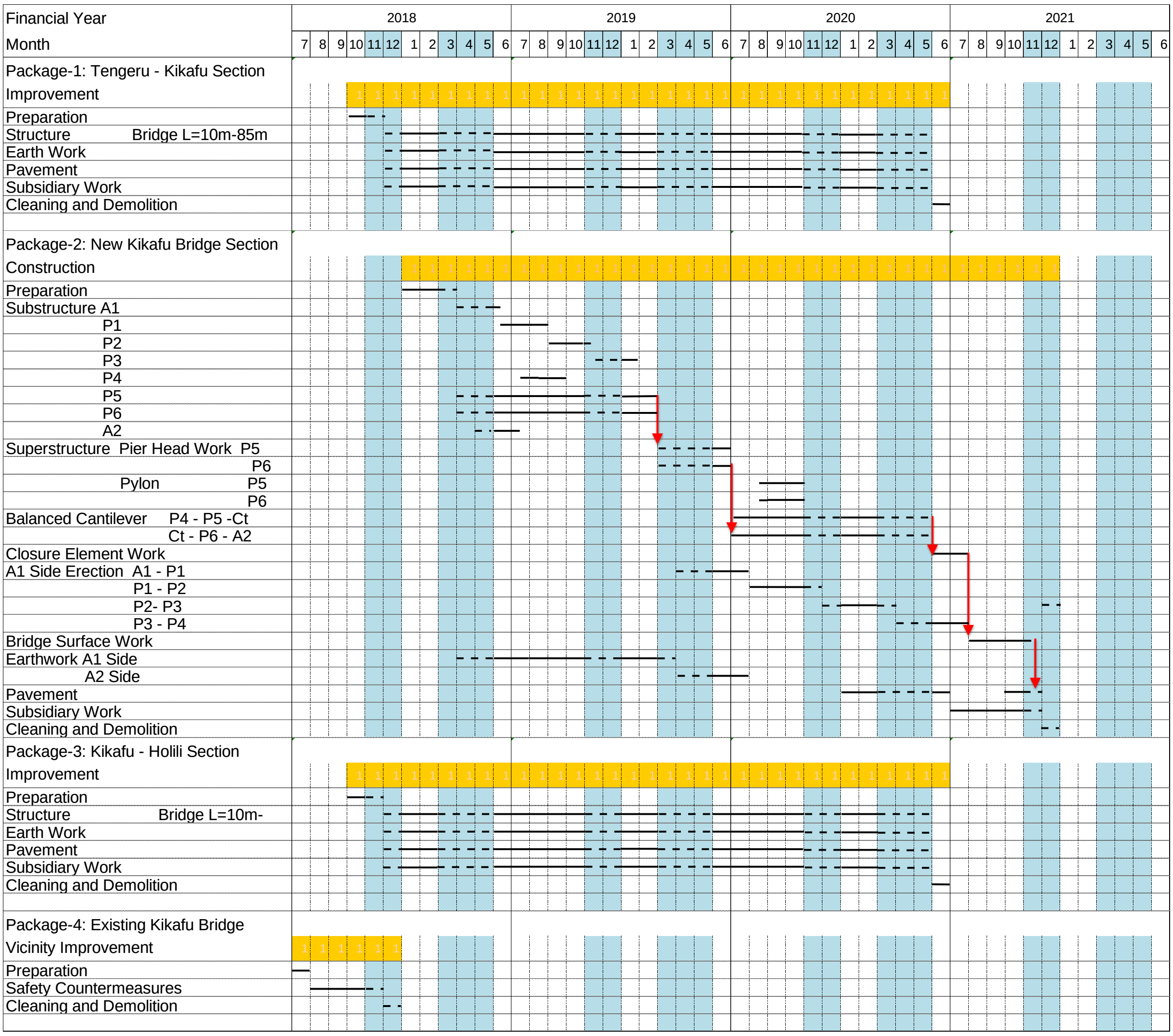
・Election is completed

Source: JICA Study Team

Figure 6.2.2 Erection Steps for Extradosed Bridge

6.2.3 Construction Schedule

Construction schedule of the Project is proposed, considering total volumes of each work item and allowable input/output volumes required for each work item and occupancy rate (considering rainy season and holidays). The following figure illustrates construction schedule of four Packages.



Source: JICA Study Team

Figure 6.2.3 Construction Schedule of the Project

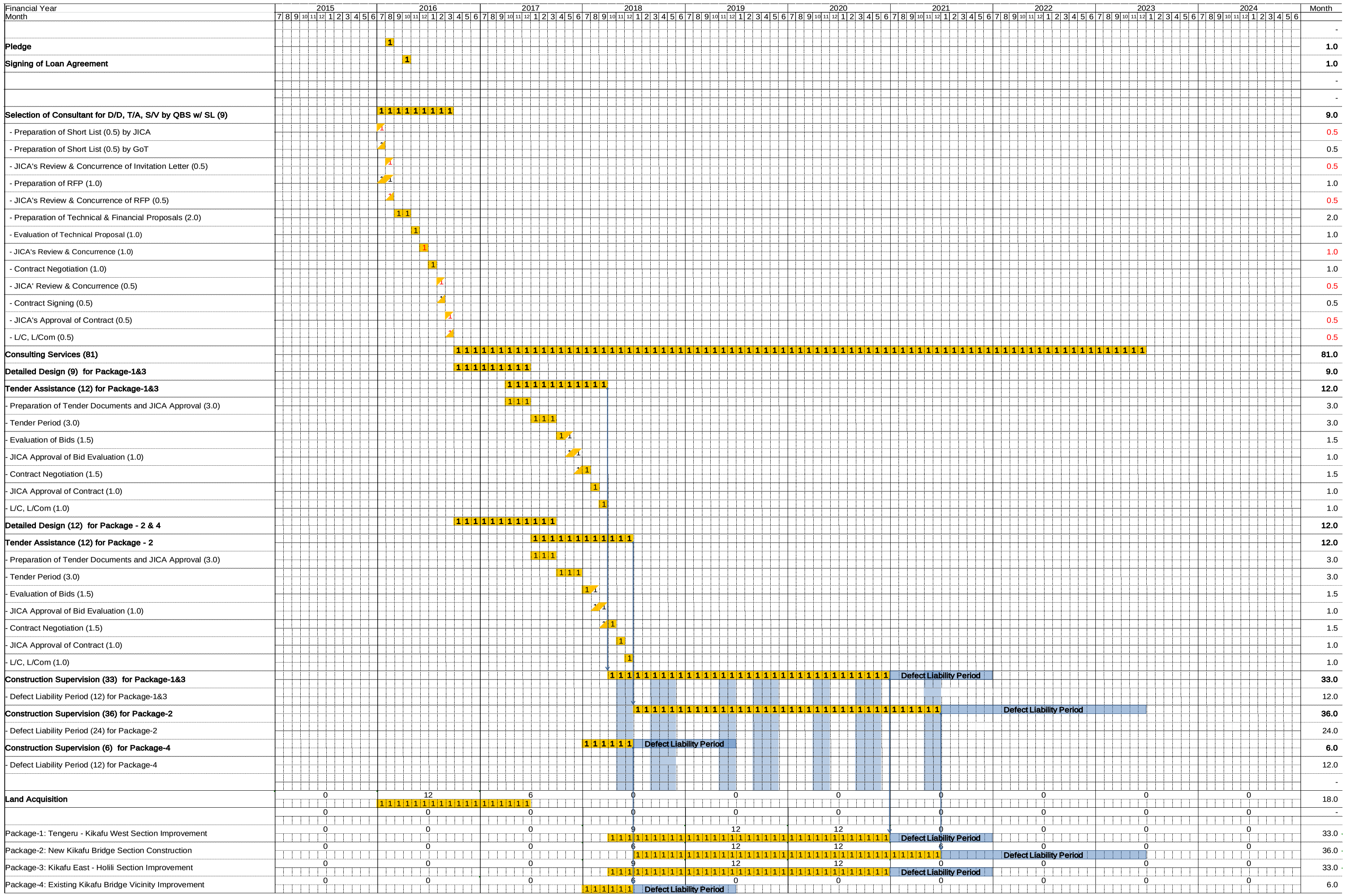
6.3 Implementation Schedule

The implementation plan for the Project is proposed based on the assumptions listed in the following table. The implementation schedule is shown in Figure 6.3.1.

Table 6.3.1 Basic Assumptions for Implementation Schedule

Item	Assumption
Loan Agreement	October, 2016
Procurement of D/D Consultant	9 months
Detailed Design	9 months for package 1 and 3 12 months for package 2 and 4
Tender Assistance	12 months
Civil Works	33 months for package 1 and 3 36 months for package 2 6 months for package 4
Land Acquisition	Completed before construction

Source: JICA Study Team



Source: JICA Study Team

Figure 6.3.1 Implementation Schedule of the Project