THE EAST AFRICAN TRADE AND TRANSPORT FACILITATION PROJECT (EATTF)

AFRICAN DEVELOPMENT BANK GROUP

PREPARATION OF A TRANSPORT FACILITATION STRATEGY FOR THE EAST AFRICAN COMMUNITY

ANNEX A
FINAL REPORT

Bureau for Industrial Cooperation
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FINAL REPORT

ANNEX A 1

HARMONISATION OF STANDARDS AND SPECIFICATIONS

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1. INTRODUCTION

1.1 Background

As part of the on going EAC efforts to enhance trade among the Partner States and with outside world thereby improving the region's economy and competitiveness, BICO was contracted to work on transport facilitation component of the East African Trade and Transport Facilitation Project (EATTFP). The East African Trade and Transport Facilitation Project is a multi-national/multi sector program to support trade growth in the region through an effective Customs Union, reliable, efficient and safe road transport services. This will enhance faster transit time, promote trade facilitation, remove non - tariff barriers and simplification and harmonization of customs formalities.

The road transport facilitation component aims at making it possible to have reliable, efficient and safe road transport services in the region. As part of the contract, therefore BICO was required to review and harmonise the following thematic areas:

i. Standards and specifications
ii. Environmental standards and regulations
iii. Vehicle registration and licensing
iv. Road safety laws and regulations
v. Axle overload control regulations: weighbridge print out certificates, training curriculum, interconnection within the EAC region and development of legal instrument for overload control
vi. Legal and institutional frameworks

Annex A is part of the outputs of the harmonisation process of the above thematic areas, and more particularly Thematic Area 1: Harmonisation of Standards and Specifications. Several subcomponents were identified to fall under this thematic area as follows:

i. Harmonisation of road geometric design standards
ii. Harmonisation of pavement and bridge design standards
iii. Harmonisation of specifications for road and bridge works
iv. Harmonisation of road and bridge maintenance standards
v. Harmonisation of road signs, traffic signals and marking
vi. Harmonisation of vehicle safety and fitness
vii. Harmonisation of driver testing and training
viii. Harmonisation of vehicle dimensions and combinations
ix. Harmonisation of transportation of abnormal, awkward and hazardous loads

Therefore this Annex addresses the above subcomponents of thematic area one (1).

1.2 Terms of Reference and Scope of Work

The Terms of Reference (TOR) clearly outlined the background of the project, its description and key objectives. The scope of services to be provided by the consultant was detailed for each of the six thematic areas outlined in Section 1.1.

As outlined in the TOR, the study aimed to achieve the objects outlined under thematic area 1 as follows:

i. Review existing documents/ statutes and propose improvements to the same. The review shall include but not limited to the following:

a) Design standards, Maintenance Manuals and specifications for roads and bridges
b) Traffic signs including traffic signals, road signs and marking
c) Safety and fitness of vehicles
d) Dimensions of vehicles and vehicle combinations
e) Transportation of abnormal, awkward and hazardous loads

ii. Identify areas of commonality which lend themselves to harmonization
iii. Propose and implement the incorporation of areas unique to particular countries into the harmonized regimes
iv. Give an indication of the impact of harmonization
v. Conduct stakeholder workshops to gain consensus on the harmonization of different regulations and standards

1.3 Study Objective

The main objective of this chapter is to address the aforementioned study objects and make recommendations on potential harmonisation regimes and improvements for the EAC region based on the analysis results of existing initiatives, and practices within the EAC region as well as best state-of-the practices and state-of-the art.

1.4 Approach and Methodology

In order to adequately address the scope of work, the overall approach and methodology adopted for the project and therefore preparation of this working paper entailed the following activities:

- Visit EAC Partner States for the purpose of collecting documents from each partner state and to make initial contacts with the responsible officials. The visits involved one member of the consultant's team visiting the contact person in the respective partner state to identify and collect documents relevant to all thematic areas.
- Preparation of an Inception Report and submission of the same to the EAC Secretariat.
- Detailed documents review, situational analysis and preparation of draft working papers. In preparation of the draft working papers the approach was to adopt higher standards and specification from EAC Partner States provided they are in line with international state-of-the practices and state-of-the art, and vice versa.
- Collection of experts' views and comments on the draft working papers through experts meetings in each EAC Partner State. Meetings were held as follows:
    - Nairobi, Kenya – 4th July, 2011
    - Bujumbura, Burundi – 6th July, 2011
    - Kigali, Rwanda – 8th July, 2011
    - Kampala, Uganda – 11th to 12th July, 2011
    - Dar es Salaam, Tanzania – 15th July, 2011
    - Zanzibar, Tanzania – 25th July, 2011
- The process of collection of experts' views was concluded by revising the draft working papers and preparation of Working Papers for submission to the EAC Secretariat for comments by the EAC Task Force Review Meeting.
- The 1st EAC Task Force Review Meeting was held from 19th -23rd September, 2011 in Dar es Salaam as part of the harmonization process with a view to collect views and comments on the Working Papers. The views and comments were used to improve the papers which were upgraded and submitted as Draft Final Report.

- The 2nd EAC Task Force Review Meeting was held from 7th – 11th May, 2012 in Mwanza to review and give comments on the Draft Final Report.

1.5. Study Area

The study area covers all EAC Partner States, i.e. Burundi, Kenya, Rwanda, Tanzania (Mainland and Zanzibar), and Uganda. Additionally, reference is made to other countries and regional communities such as COMESA, and SADC.

1.6 Structure of the Report

This report constitutes Annex A of the Final Report. It is subdivided into ten chapters including the introduction. Each chapter addresses one of the nine (9) subcomponents under Thematic Area 1: Harmonisation of Standards and Specifications:

i. Chapter 2: Harmonisation of road geometric design standards
ii. Chapter 3: Harmonisation of pavement and bridge design standards
iii. Chapter 4: Harmonisation of specifications for road and bridge works
iv. Chapter 5: Harmonisation of road and bridge maintenance standards
v. Chapter 6: Harmonisation of road signs, traffic signals and marking
vi. Chapter 7: Harmonisation of vehicle safety and fitness
vii. Chapter 8: Harmonisation of driver testing and training manuals
viii. Chapter 9: Harmonisation of vehicle dimensions and combinations
ix. Chapter 10: Harmonisation of transportation of abnormal, awkward and hazardous loads

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ANNEX A 2

HARMONISATION OF ROADWAY GEOMETRIC DESIGN STANDARDS

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GLOSSARY AND ACRONYMS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transport Officials
ADT	Average Daily Traffic
CSIR	Centre for Scientific and Industrial Research (RSA)
DHV	Daily Hourly Volume
EAC	East African Community
GSI	Grade Separated Intersection
HCM	Highway Capacity Manual
LOS	Level of Service
MUTCD	Manual for Uniform Traffic Control Devices
NCHRP	National Cooperative Highway Research Program
NMT	Non Motorised Transport
ORN	Overseas Road Note
PCU	Passenger Car Unit
RSA	Republic of South Africa
TRL	Transport Research Laboratory
SADC	Southern African Development Cooperation
SATCC	Southern African Transport and Communications Commission
SU	Single Unit
WB	Wheel Base
UK	United Kingdom
USA	United States of America
VD	Design Speed

EXECUTIVE SUMMARY

The geometric design of roadways deals with the dimensions and layout of visible features of a roadway with the objective of creating the roadway facility to suit the characteristic and behaviour of drivers, vehicles, traffic and terrain. It is based on specified design standards and controls which depend on a wide range of roadway system factors such as roadway functional classification, terrain of the area that the roadway traverses, levels of service, road user characteristics, vehicle characteristics (length, width, height, wheelbase, weight, including acceleration and deceleration characteristics and maximum speed) and design hourly traffic volume, sight distances and choice of horizontal and vertical alignment, road cross section, intersection type, available funds, safety, and social and environmental factors. These factors are often interrelated and coupled with the basic characteristics of the driver, vehicle, and road are used to determine standards for the geometric characteristics of the roadway, such as cross sections and horizontal and vertical alignments. Appropriate geometric design standards should be selected to maintain a desired level of service for a given traffic mix. The use of geometric design standards fulfils three inter-related objectives. Firstly, standards are intended to provide minimum levels of safety and comfort for drivers; secondly, they provide the framework for economic design; and thirdly, they ensure a consistency of alignment.

Geometric Design Standards in the EAC Partner States

The study found that Burundi and Rwanda follows French geometric road design standards though the latter has recently shifted to American standards. It also found that Kenya, Tanzania and Uganda are using their own standards which were developed largely from the American and English practice. It was found that countries like Tanzania has adopted some design criteria from the geometric design standard of the Southern African Transport and Communications Commission (SATCC), which was also derived largely from the American and English practice. A review of these standards revealed a number of areas of commonality as well as areas unique to particular countries, which lend themselves into harmonisation regimes.

Road Classification and Access Control

There are various schemes for classifying roads and the definitions a class generally vary depending on the purpose of classification. The principal purposes of road classification are to establish logical integrated systems that, because of their particular service, should be administered by the same jurisdiction; relate geometric design control and other design standards to the roads in each class; and establish a basis for developing long range programmes, improvement priorities and financial plans. The two major considerations in classifying highway and street networks functionally are access and mobility. The conflict between serving through movement and providing access to a dispersed pattern of trip origins and destinations leads to the differences and gradations in the various functional types. The extent and degree of access control is thus a significant factor in defining the functional category of a street or highway. The Final Report by Africon Ltd (2011) on the EAC Transport Strategy and Regional Roads Sector Development Program, noted that the current road classifications that are in place within the EAC member states and which focus on high mobility roads (Class 1-3) are similar in terms of their function. The review by Africon Ltd (2011) of best practice in road classification by various international authorities is shown below.

Country		South Africa			USA				International		
Function	Class.	RCAM 2010	RAM 2005	RISFSA 2006	AASHTO 1973, 2004, FHA 1989; both urban and rural	ITE 1988, NCHRP 1992	TRB 2003: Access Manual: State	TRB 2003: Access Manual: Local	Great Britain ordnance survey, DfT, HA	Australia	New Zealand
Mobility (arterial)	1	Principal arterial	Principal arterial	Primary distributor	Principal arterial system, interstate	Freeway, expressway	1 Interstate highways and other freeways	1 Freeway	Motorway, trunk roads	Freeway, M-route, state road, national highway	National road
Mobility (arterial)	2	Major arterial	Major arterial	Regional distributor	Principal arterial system, other	Strategic arterial, principal arterial, primary arterial	2 Roadway of state-wide importance (strategic, principal arterials)	2 Major arterial	A-road, principal road	Major arterial, A-route, regional road	Primary (regional) arterial
Mobility (arterial)	3	Minor arterial	Minor arterial / activity arterial - spine	District distributor	Minor arterial road / street system	Secondary arterial	3 Roadway of regional importance (other arterials)	3 Minor arterial	B-road	Minor arterial, B-route, district road, sub-arterial	Secondary (district) arterial
Access / Activity (street)	4	Collector (street) *(4a commercial, 4b residential)*	Activity street	District collector	Major collector road / street system, minor collector roads	collector	4 District roadway (collector)	4 Major collector, 5 Minor collector	C-road, minor roads	Collector, C-route	Collector road
Access / Activity (street)	5	Local street *(5a commercial, 5b residential)*	Residential street	Access roads	local road / street system	local, cul-de-sac	5 Local	6 Local	Local street, alley, private road with or without public access	local, lane	Local road
Access / Activity (street)	6	Walkway *(6a pedestrian priority, 6b pedestrian only)*	Non-motorised	Non-motorised access ways	Terminal and transfer facilities				pedestrianised street with restricted access		

Notes: AASHTO 1973 A policy on Design of Urban Highways and Arterial Streets 1973 (red book)
AASHTO 2004 A policy on Design of Urban Highways and Arterial Streets 2004 (green book)
FHA 1989 Highway functional classification, USD-DOT, Federal Highway Administration, March 1989
GB, Aus, NZ Assessing the feasibility of a National Road Classification, Intergovernmental Committee on Surveying and mapping, Australia, Oct 2006
ITE 1988 Transportation and Land Use Development, ITE, Stover and Koepke, 1988
RAM 2005 National Guidelines for Road Access Management in South Africa, October 2005 draft
RCAM 2010 National Guidelines for Road Classification and Access Management in South Africa, 2010
RISFSA 2006 Road Infrastructure Strategic Framework for South Africa, October 2006 (earlier version 2002)
TRB 2003 Access Management Manual, Transportation Research Board of the National Academies, 2003
Source: RCAM 2010 National Guidelines for Road Classification and Access Management in South Africa, 2010, cited in Africon Ltd (2011)

It was found that EAC Partner States are using the same road classification system with very slight changes in the definition of lower classes of roads i.e. access/minor roads. This classification serves well road administrative purposes. But each partner state has its own road design classification system. It is therefore desirable to harmonise road classification system for design purposes based on the road function.

Basing on the international practice, Africon Ltd (2011) recommended to the EAC member states the categories of High Mobility-Class 1 and High Mobility – Class 2 for the EAC corridor road classification and numbering. High mobility – class 1 roads are roads which movement is dominant, through traffic is dominant, the majority of traffic does not originate or terminate in the immediate vicinity, the function of the road is to carry high volumes of traffic between urban districts, across and through member states; connecting member states; freight focused and freight supportive. Class 2 roads are corridor feeder roads with its main function that of supporting the EAC corridors, i.e. Class 1 roads.

In accordance with the international practice, mobility roads include principal arterial roads that provide the highest level of mobility and the highest speeds over the longest uninterrupted distance. They also include multilane, two-lane two ways arterials, divided or undivided roads which connect or support, as directly as practicable the principal arterial systems. Lower classes of mobility roads provide less mobility and therefore balance both functions of mobility and accessibility, whereas access roads provide limited mobility and are the primary access to residential areas, businesses, farms, and other local areas.

The following table presents proposed functional road classifications in the EAC region. It shows the interrelationship between the existing and standard classification naming conventions for administrative and functional purposes. The table also shows the proposed spacing for the respective road classes as suggested by Africon Ltd (2011).

Function	Class	Name conventions		Access control
Mobility roads	Class 1	International trunk roads National trunk road	Principal Arterial/ Freeway/ Expressway (2500 m spacing)	Full
	Class 2	National roads Provincial roads Regional roads District roads	Major arterial/Highway (800 spacing)	Full or partial
	Class 3	Regional roads District roads Secondary roads	Minor arterial (600 m spacing)	Full or partial
Access roads	Class 4	District roads Secondary roads	Collector roads (50 m spacing)	Partial
	Class 5	Minor roads Local streets	Local streets (15 m spacing)	Unrestricted

Road reserve

The width of right-of-way for complete development of an arterial road is influenced by traffic demand, terrain, land use, cost, intersection design, and the extent of ultimate expansion. It is the summation of the various cross sectional elements: through roadways, roadside clear zones, side slopes, drainage facilities, utility appurtenances, and retaining walls. It should be based on the preferable dimensions of each element to the extent practical in developed areas.

Right of way widths provided for by design manual for EAC Partner States are within the right of way widths proposed for use in other countries. However, it was noted that the road reserve of 60 m recommended for international trunk roads is below the minimum right of way width required for freeways. We therefore recommend that EAC Partner States should plan to acquire incrementally more space for right of way in the range of 79 – 100 m so as to allow for future expansion of existing international and national routes to the freeway and expressways standards. Additional land should be acquired at all designated places for rest areas.

Design Control and Criteria

Design Vehicle and Vehicle Characteristics

The principal dimensions affecting design are the wheelbase, width, height, and turning radius. In general, buses and heavy vehicles should be used as the design vehicle for cross section elements, with the car as the design vehicle for the horizontal and vertical alignment. For most major intersections along arterial roads or within commercial areas, it is common practice to accommodate rigid trucks and vehicle combinations. The occasional larger vehicle may encroach on adjacent lanes while turning but not on the sidewalk.

Though there are some similarities, design manuals used in the EAC region present different dimensions of design vehicles. Burundi and Rwanda followed French design standards while the source of the design vehicle dimensions specified in the Kenya design manual is not known but it was assumed that they represented the vehicle market conditions in Kenya. The Tanzania manual indicated different dimensions of design vehicle with SATCC Code of Practice for the Geometric Design of Trunk Roads acknowledged as the source for the dimensions while the Uganda manual indicated dimensions of design vehicles with AASHTO Geometric Design Manual of Highway and Streets acknowledged as the source for the dimensions.

Comparison of the width, height, wheelbase and minimum turning radius dimensions for the vehicles such as Interlinks to the respective dimensions for the design vehicles documented in the existing design manuals for EAC Partner States showed that the 22 m Interlink vehicles, which has recently been permitted to traverse EAC road network, will be accommodated by the existing intersections on the EAC road network. We therefore recommend that Design vehicle dimensions should be uniform across all EAC Partner States and dimensions shown in the table below should be considered for adoption.

Design vehicle type	Symbol	Dimension (m)				
		Length	width	Height	Minimum design turning radius	Wheelbase
Passenger cars						
Passenger car	DV1	5.8	2.1	1.3	7.31	3.35
Buses						
Single unit bus	DV2	12.1	2.6	4.1	12.8	7.6
Articulated bus	DV3	18.3	2.6	3.4	12.1	6.7 + 5.9
Trucks						
Single unit truck	DV4	9.1	2.6	4.6	12.8	6.1
Semi-trailer	DV5	18.2	2.6	4.6	12.65	5.5+10.0

Rigid truck and drawbar trailer combination	DV6	22.0	2.6	4.6	12.0	4.6+8.2
Interlink (with a short truck tractor)	DV7	22.0	2.6	4.6	12.0	4.1+6.52*+7.94
Semi-trailer with long trailer	DV8	21.0	2.6	4.6	13.7	6.1 & 12.8

**distance between SU rear wheels and trailer front wheels*

Terrain

Each EAC Partner States has adopted and defined different classes of terrain and the associated average ground slope. Kenya has three terrain types, Tanzania and Uganda have four. Interestingly, Tanzania combines hilly and mountainous terrains into one group in specifying design standards while the Uganda design manual uses only three terrain types to define the associated standards and the escarpment terrain is left out. On the other hand, the Trans-African Highway Network is classified into four (4) terrain categories of level, rolling, mountainous, and steep. We recommend to the EAC member countries to adopt terrain categories of flat with cross slope of 0 to 10 per cent, rolling with cross slope of more than 10 to 25 per cent, mountainous with cross slope of more than 25 to 60 per cent, and steep with cross slope of more than 60 per cent.

Perception-Reaction Time

Guidance on driver perception reaction time does not vary among EAC Partner States. We recommend that EAC Partner States should continue to use driver perception reaction time of 2.5 seconds.

Height of Driver's Eye

Driver eye height values vary between the EAC Partner States, but with reasonable narrow ranges. We propose that EAC Partner States should adopt a driver eye height of 1.05 m for sight.

Object Height

Object height design values vary between the EAC Partner States, but with reasonable narrow ranges. We recommend object height values shown in the table below for the EAC region.

Object height (m)		
Stopping sight distance	**Decision sight distance**	**Passing sight distance**
0.60	0.60	1.08

Pedestrians and cyclists performance

To effectively plan and design pedestrian facilities, it is necessary to understand the typical pedestrian. Likewise, to provide adequately for bicycle traffic, the designer should be familiar with bicycle dimensions, operating characteristics, and needs. These factors determine acceptable turning radii, grades, and sight distance. We recommend pedestrian speed of 1.2 m/s and adult typical bicycle performance criteria i.e. speed range of 13-24 km/h; coefficient for braking of 0.32, and minimum height of 1.0 m and width of 1.5 m for bicycle operation for the design of NMT facilities.

Traffic Characteristics

Traffic composition

It is not practical to design for a heterogeneous traffic stream and, for this reason; trucks and other types of vehicles are converted to equivalent Passenger Car Units (PCUs). The number of PCU's associated with a single vehicle type is a measure of the impedance that it offers to the passenger cars in the traffic stream. A comparative assessment of the vehicle equivalent factors given in the geometric design manuals for EAC Partner States revealed that these countries have recommended use of same vehicle equivalent factors, except Tanzania design manual which specified a factor of 3.0 as the vehicle equivalent factor for Medium Goods Vehicles in rolling terrain. Furthermore, PCU for motorcycles were reviewed and we recommend vehicle equivalent factors shown below.

Vehicle type	Terrain		
	Level	Rolling	Mountainous
	pcu		
Passenger cars	1.0	1.0	1.5
Light goods vehicles	1.0	1.5	3.0
Medium goods vehicles	2.5	5.0	10.0
Heavy goods vehicles	3.5	8.0	20.0
Buses	2.0	4.0	6.0
Motor cycles, scooters	0.5	1.0	1.5
Pedal cycles	0.5	0.5	NA

Projection of Future Demands

Right-of-way and grading may be considered to have a physical life expectancy of 100 years; minor drainage structures and base courses, 50 years; bridges, 25 to 100 years; resurfacing, 10 years; and pavement structure, 20 to 30 years, assuming adequate maintenance and no allowance for obsolescence. Bridge life may vary depending on the cumulative frequency of heavy loads. Pavement life can vary widely, depending largely on initial expenditures and the repetition of heavy axle loads. The decision is greatly influenced by economics. For example, a highway might be designed for traffic volumes 50 years hence with the expectation that the pavement structure would be restored in 20 to 25 years. However, if the added cost of a 50-year design over a design with a 25-year life expectancy is appreciable, it may be imprudent to make a further investment providing capacity that will not be needed for at least 25 years. The construction cost savings could be used to construct another currently needed highway project. Furthermore, the cost of increased maintenance for the larger highway would be avoided for at least 25 years. AASHTO (2011) suggests that the maximum design period should be in the range of 15 to 24 years and a period of 20 years is widely used as a basis for design of new facilities and 5 to 10 years for reconstruction or rehabilitation projects because of uncertainties of predicting traffic and funding constraints.

It was found that the draft design manual for Tanzania does not specify a specific design period, however, the pavement and materials design manual suggests 20 years. Uganda design manual recommends use of 10 – 20 years as design year.

We propose that projection of future traffic demand should be based on a period of 20 years for new roadway design and 10 years for reconstruction and rehabilitation in EAC Partner States.

Design hourly volume (DHV)

The design hourly volume is the projected hourly volume that is used for design. The DHV is the 30th HV highest hourly volume during the year. Geometric design manual for Uganda also estimates DHV as = AADT x K or ADT x K where K is estimated from the ratio of the 30th HV to the AADT from a similar site and in the absence of such factors the manual for Uganda estimates the 30th-highest DHV by applying the factors of 0.15 and 0.10 to the ADT for rural highways and urban roads, respectively. Other design manuals in the EAC region do not show factors that are used to compute the DHV.

We propose that field data should be used to estimate K factors for estimation of DHV and in the absence of such factors the 30th-highest DHV should be estimated by applying 0.15 and 0.10 to the ADT for rural highways and urban roads, respectively.

Design Speed

The most important factor in geometric design is the design speed. This was previously defined as the highest continuous speed at which individual vehicles can travel with safety on the road when weather conditions are favourable, traffic volumes are low and the design features of the road are the governing condition for safety. The current definition is simply states that the design speed is the speed selected as the basis for establishing appropriate geometric elements for a section of road.

Review of design speeds specified in the design manuals used in EAC Partner States revealed that the minimum design speed in Tanzania for flat terrain for Class A roads is higher compared to Kenya and Uganda. Apart from this observation, it was noted that the design speeds for various classes of roads in various terrain types varies among the Partner States. This is not surprising as there is some variances in the transverse slope values even within the same terrain classification. We recommend use of design speed values shown in the table below for the EAC road network.

Road class		**Design Speed (km/h)**			
		Flat terrain	**Rolling terrain**	**Mountainous**	**Steep**
Mobility roads	Class 1	120	100	60	60
	Class 2	110	80	50	50
	Class 3	100	80	50	40
Access roads	Class 4	80	60	40	40
	Class 5	60	40	30	30

Levels of Service

Levels of Service express the effectiveness of the road in terms of operating conditions. It is a qualitative measure of the effect of traffic flow factors, such as speed and travel time, interruptions, freedom of manoeuvre, driver comfort and convenience, and indirectly, safety and operation costs. The choice of level of service is generally based on economic considerations.

Guidelines for selection of design levels of service for the different classes of rural roads are also given in the design manuals for EAC Partner States. However, the level of service given varies between the roadway geometric design guides. The design guide for Uganda suggests design levels of service for rural roads for a given combinations of road functional class, road design class and terrain while levels of service selection guidelines are missing in the design guides for Kenya and Tanzania. We recommend that EAC should adopt guidelines shown in the table below for the selection of design levels of service for the different road classes.

Level of service for specified combinations of terrain and area type					
Function	**Class**	**Level terrain**	**Rolling terrain**	**Mountainous terrain**	**Steep terrain**
Mobility roads	Class 1	B	B	C	C
	Class 2	B	C	D	D
	Class 3	C	C	D	D
Access roads	Class 4	D	D	D	D
	Class 5	D	D	D	D

Access Control and Access Management

Regulation of public access rights to and from properties abutting the highway facilities is called access control. It is usually achieved though a number of controls which can be categorized as full control of access, partial control of access, access management, and driveway/entrance regulations. The principal advantages of controlling access are the preservation or improvement of service and the reduction of crash frequency and severity (AASHTO, 2011).

Access management addresses the basic questions of when, where, and how access should be provided or denied, and what legal or institutional changes are needed to enforce these decisions. In a broad context, access management is resource management, since it is a way to anticipate and prevent congestion and to improve traffic flow. Key elements of access management include defining the allowable access and access spacing for various classes of highways, providing a mechanism for granting variances when reasonable access cannot otherwise be provided, and establishing means of enforcing policies and decisions. These key elements, along with appropriate design policies, should be implemented through a legal code that provides a systematic and supportable basis for making access decisions.

We recommend that EAC member states should put in place statutes, land-use ordinances, geometric design policies, and driveway regulations for managing and controlling access

Horizontal Alignment

Stopping Sight Distance

This is the distance required by a vehicle driven by a below-average driver at or near the design speed to stop before reaching a stationary object in its path. It is basically a sum of two distances namely; a brake reaction distance and a breaking distance. The brake reaction time corresponds to the perception reaction time which has been harmonised at a value of 2.5 seconds. The breaking distance is the distance to bring the vehicle to zero speed and depends on the brake force coefficient. Therefore the stopping sight distance can be determined as: D = 0.278Vt + 0.039(V2/a) where; t = brake reaction time, 2.5 sec; V = design speed, km/h; and a = acceleration rate of 3.4 m/s2. We recommend the following minimum stopping sight distance (SSD) for the various design speeds.

Design Speed (km/h)	30	40	50	60	80	100	110	120
SSD (m)	35	50	65	85	130	185	220	250

Passing Sight Distance

Passing sight distance is an important criterion that affects the capacity as well as the quality of service provided by the road. A heavily trafficked road requires a higher percentage of passing sight distance than a lightly trafficked road to provide the same level of service. Passing sight distance is calculated on the basis of a distance required for a successful overtaking manoeuvre and makes adequate provision for an aborted manoeuvre in the case of a truck attempting to pass another truck. We propose the following rounded passing sight distances for the various design speeds.

Design Speed (km/h)	30	40	50	60	80	100	110	120
PSD (m)	200	270	345	410	540	670	730	775

The straight

Straight sections provide better visibility and more passing opportunities and hence enhance safety. However, long straight sections increase the danger from headlight glare and usually lead to excessive speeding. In hot climate areas, long tangents have been shown to increase driver fatigue and hence cause accidents. Thus, the design guides have attempted to limit the length of straight sections. Guidelines for determination of minimum and maximum length of straight sections given vary between the roadway geometric design guides for the EAC Partner States. Straights should not have length greater than (20xVD) m (where VD is design speed in km/h and a minimum length of (6xVD) m should also be adopted between circular curves following the same direction.

Minimum radius of horizontal curve

For a given design speed, minimum curve radius is limited by maximum allowable side friction, which is usually based on comfort standard, maximum superelevation rate for the curve, and the necessity to maintain stopping sight distance. Guidelines for determination of minimum radius for horizontal curve vary between the roadway geometric design guides for the EAC Partner States as well. We recommend EAC to adopt maximum superelevation rates of 4% on roads in urban areas and 10% on roads in the rural areas as well as side friction factors with corresponding design speed shown below.

Design speed (km/h)	Limiting value of *f*
30	0.17
40	0.17
50	0.16
60	0.15
70	0.14
80	0.14
90	0.13
100	0.12
110	0.11
120	0.09

Additionally, we recommend that the distribution of superelevation and side friction over a range of curves should be such that superelevation and side friction are proportional to the inverse of the curve radius.

Minimum and maximum lengths of curve

For small deflection angles, it is required that curves should be long enough to avoid the appearance of a kink. SATCC design guide recommends a minimum length of curve of 300 m and if space is limited this length may be reduced to 150 m. Likewise SATCC design guide recommends that the length of a curve should not exceed 1 000 m and the preferred maximum length is 800 m. This length applies also to multilane cross-section because, while passing opportunities do not pose a problem, tracking still remains an issue. We recommend that the length of curve should not exceed 1000 m while the minimum length should be 300 m for EAC network.

Transition curve

A transition curve give the driver a natural path that is easy to follow as a vehicle enters or leaves a circular horizontal curve and at the same time provides a suitable arrangement for superelevation runoff. The appearance of the road is also enhanced by the use of transition curves, since this avoids noticeable breaks at the beginning of circular curves, these breaks often being made more pronounced by the superelevation runoff. They are most appropriate for roadways with relatively high design standards, where large-radius curves are used. In horizontal curve design lateral acceleration is equivalent “centripetal acceleration”, the term lateral acceleration is used.

The design of the transition includes consideration of the changes of the alignment from tangent to circular as well as the cross section from normal to superelevated. An Euler spiral (clothoid) is the most common to be used in the design of a spiral transition from tangent to curve and curve to tangent. A spiral transition curve give the driver a natural path that is easy to follow as a vehicle enters or leaves a circular horizontal curve and at the same time provides a suitable arrangement for affecting the superelevation runoff. The appearance of the road is also enhanced by the use of spiral transition curves, since this avoids noticeable breaks at the beginning of circular curves, these breaks often being made more pronounced by the superelevation runoff.

AASHTO (2011) observe that a minimum length of transition curve is defined based on consideration of driver comfort and shifts in the lateral position of vehicles. Criteria based on driver comfort are intended to provide spiral length that allows for a comfortable increase in lateral acceleration as a vehicle enters a curve. The criteria based on lateral shift are intended to provide a spiral curve that is sufficiently long to result in a shift in a vehicle’s lateral position within its lane that is consistent with that produced by the vehicles natural spiral path. It is recommended that these two criteria can be used together to determine the minimum length of spiral which can be computed as

$$L_{s,\min} = \sqrt{24\left(p_{\min}\right)R} \text{ or } L_{s,\min} = 0.0214\frac{V^3}{RC}$$

Where:

$L_{s,\min}$ = minimum length of spiral, m

$p_{\min}$ = minimum lateral offset between the tangent and circular curve (0.20 m)

R = radius of circular curve, m

V = design speed, km/h

C = maximum rate of change in lateral acceleration (1.2 m/s^3)

AASHTO (2011) also reports that international experience indicates that there is a need to limit the length of spiral transition curve. Spirals should not be so long (relative to the length of the circular curve) that drivers are misled about the sharpness of the approaching curve. A conservative maximum length of spiral that can minimize the likelihood of such concerns can be computed as:

$$L_{s,\max} = \sqrt{24(p_{\max})R}$$

Where:

$L_{s,\max}$ = maximum length of spiral, m

$p_{\max}$ = minimum lateral offset between the tangent and circular curve (1.0 m)

R = radius of circular curve, m

Further, AASHTO (2011) provides desirable lengths of spiral transition curves as shown in the table below. These lengths correspond to 2.0 s of travel time at the design speed of the roadway. The travel time has been found to be representative of the natural spiral path for most drivers and thus the spiral lengths listed in Table 30 are recommended as desirable values for street and highway design. The use of longer spiral curve lengths that are less than $L_{s,\max}$ than is acceptable. However, where such longer spiral curve lengths are used, consideration should be given to increasing the width of the travelled way on the curve to minimize the potential for encroachments into adjacent lanes. Further, if the desirable spiral curve length shown in Table 30 is less than the minimum spiral curve length $L_{s,\min}$, the minimum spiral curve length should be used in design.

Desirable length of spiral curve transition

Design speed (km/h	20	30	40	50	60	70	80	90	100	110	120	130
Spiral length (m)	11	17	22	28	33	39	44	50	56	61	67	72

The maximum relative gradient between pavement edges (see table below) will determine the minimum length for the runout and runoff and hence the minimum length of the transition curves.

Maximum relative gradient

Design Speed (km/h)	30	40	50	60	80	100	110	120
Maximum relative gradient	1:133	1:143	1:154	1:167	1:200	1:227	1:244	1:263

In Kenya transition curves are not required for horizontal curves with radii greater than 2000 m. Design guides for Tanzania and Uganda recommend maximum relative gradient shown in Table 29 and the application of transition curves to circular curves under the following condition: $R < \frac{V^3}{432}$

where:

R = Radius of curve in metres,

V = Design speed in km/hr

We propose the maximum relative gradient given in table above for various speeds for determination of the minimum length for the runout and runoff. Additionally, the minimum length of transition curve should be based on consideration of driver comfort and shifts in the lateral position of vehicles and these two criteria can be used together to determine the minimum length of transition curve.

Vertical Alignment

Gradients

Maximum grades vary, depending on the type of facility, and usually do not constitute an absolute standard. The effect of a steep grade is to slow down the heavier vehicles and increasing operating costs (beyond an ascending gradient of 2.5 % each additional percentage point of gradient leads to a 12 % increase in fuel consumption compared and the extent to which any heavier vehicle is slowed depends on both the steepness and length of the grade. Maximum gradient guidelines range in complexity, with various countries considering some or all of the following factors: road classification, design speed and terrain. In Tanzania and Uganda, maximum grades are based on design speed and topography, and there is no much difference in the grades specified by the two design manuals.

To avoid standing water in side ditches, the minimum gradient for roads in cutting is recommended to be 0.5% for Kenya. It is further recommended to check for length of grades or otherwise climbing lane may be justifiable in some cases. For Tanzania, the draft guide proposed minimum grades on cut sections to be 0.5% unless special drainage treatments are provided while the recommended minimum gradient in cuttings in order to avoid standing water in the road side ditches is 0.3% - 0.5% in Uganda. We recommend a minimum grade of 0.5% and maximum grades shown in table below.

Terrain	Maximum gradient (%)
Flat	4
Rolling	4 – 8
Mountainous	7 – 12
Steep	12 - 18

Climbing lanes

The existing design guides for EAC Partner states show that each of the EAC Partner state is following different requirements for climbing lanes. Comparing these requirements with those prescribed in the AASHTO guide, it is found that AASHTO requirements for climbing lanes are simple to use because they require data (i.e. traffic volumes and speed only) which can easily be obtained for site assessment, and above all they include a safety criterion. We therefore recommend use of the following AASHTO requirements for climbing lanes.

- i) Upgrade traffic flow rate in excess of 200 vehicles per hour
- ii) Upgrade truck flow rate in excess of 20 vehicles per hour
- iii) One of the following conditions exists:
    - a) A 15 km/h or greater speed reduction is expected for a typical heavy truck
    - b) Level of service E or F exists on the grade
    - c) A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

In addition safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes

Vertical curves

Minimum lengths of crest vertical curves based on sight distance criteria generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as ramp exit gores, where longer sight distances and, therefore, longer vertical curves should be provided.

For sag curves, four different criteria for establishing lengths of sag curves are known; headlight sight distance, passenger comfort, drainage control and general appearance. Headlight sight distance is recommended by AASHTO for determining the length of sag curves. For overall safety on highways, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. As in the case of crest curves, it is convenient to express the design control in terms of the K rate for all values of A. Minimum lengths of vertical curves (i.e. 0.6 times the design speed in km/h) for the flat gradients also are recognised for sag conditions.

The basic formulas for length of a crest vertical curve in terms of algebraic difference in grade when stopping sight distance is taken as a control are:

When S is less than L,

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

When S is greater than L,

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}$$

Where: L = length of vertical curve, m

S = Stopping sight distance, m

A = algebraic difference in grades, %

h_1 = height of eye above roadway surface, m

h_2 = height of object above roadway surface, m

Thus, the major control for safe operation on crest curves is the provision of ample sight distances. Design guide for Kenya recommended that the length of both sag and crest curves should generally be not less than (2 x VD) metres where VD is the design speed in km/h. Design guides for Tanzania and Uganda suggest the vertical curves lengths in terms of rate of curvature K as given by L = KA where L = minimum length of vertical curve, K = rate of vertical curvature per change in grade given as meters per percent grade change, and A = algebraic difference between the gradient (%).

We recommend that minimum lengths of vertical curves should be based on stopping sight distance criteria and the length of a curve (in metres) should not be shorter than the design speed. For good aesthetics the minimum length (in m) should not be less than twice the design speed (in km/h). For crest vertical curves, the minimum length standard depends on the stopping sight distance, the height of the driver's eye measured from an eye height of 1.05 m, and the height of the object of 0.60 m to be seen over the crest of the curve. For sag curves, stopping sight distance is based on the distance illuminated by the headlight height of 0.6 m at night.

Cross Section Elements

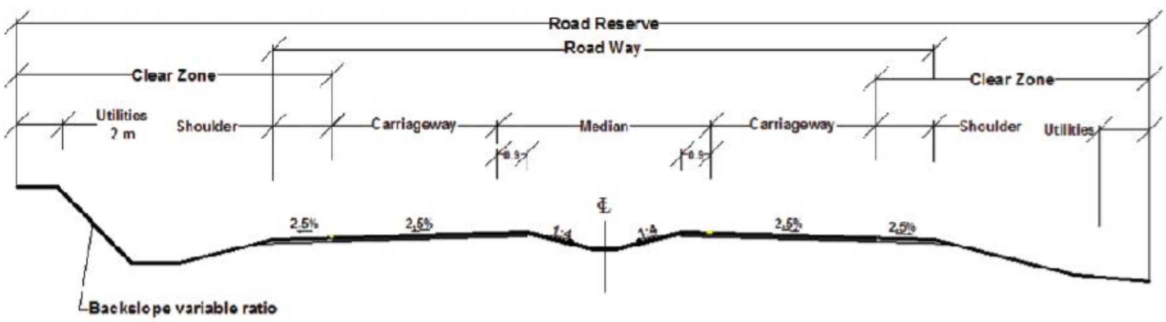
Lane Width

A fundamental feature of roadway cross section is the width of a travel lane, which must be sufficient to accommodate the design vehicle, allow for imprecise steering manoeuvres, and provide clearance for opposing flow in adjacent lanes. It was found that lane widths vary between EAC countries but with reasonable narrow ranges.

We recommend the following lane widths for EAC:

- For mobility roads classes 1 to 3, lanes should be 3.5 to 3.75 m wide
- For access roads classes 4 to 5, lanes should be 2.5 to 3.5 m wide

The actual width to be adopted shall be determined after considering the traffic volume and terrain. The figure below presents typical cross sectional elements of a dual carriageway.



Shoulders

Shoulders are used for emergency stopping, for parking of disabled vehicles, and for lateral support of the subbase, base, and surface courses of the travel roadway. In cases of paved road, lane width covers net width excluding width of edge strip, edge line and centreline markings. Shoulders should be wide enough to adequately fulfil their purpose, but excessive width encourages drivers to use them as an additional travel lane. It was found that almost all countries use narrower shoulders for classifications of roadways with lower design speeds and that several studies have found that wider shoulders produce significant safety benefits. It was generally noted that there is no international consensus on appropriate shoulder width. As expected, it found that there is a slight variation in shoulder widths between the EAC Partner States, but almost all widths are within the ranges of widths specified by SATCC guide as well as AASHTO. We recommend the following shoulder widths for EAC:

a) *For mobility road classes 1 to 3 in flat terrain, minimum width for shoulders should be 2.0 m.*
b) *For roads in rolling and mountainous terrain shoulders should be 1.0 - 1.5 m wide.*
c) *For access roads classes, shoulders should be 0.6 to 1.0 m wide*

Normal Cross Fall

A cross slope is used on traffic lanes to promote drainage of surface water. Divided roadways can be treated as two separate roadways, each having a centre crown bordered by slopes to the outside of the pavement, or each direction of travel can be sloped unidirectionally. Review of normal cross fall used in other countries revealed that 2.0 to 2.5% cross slope is the most widely accepted value for design, and that there is little or no difference by roadway class. Additionally, some countries shoulders are sloped at the same rate as the adjacent roadway lane, while in others the shoulder slope can be as much as 2% greater than the adjoining lane. Kenya manual recommended that for rural roads with bituminous pavements the minimum cross fall should be 2.5% and for rural roads with gravel pavements 4.0%. The shoulder should have the same slope as the carriageway. The Tanzania manual proposed the cross fall for asphalt concrete surfaces 2.5%, surface dressing surfaces 2.5%, stone paved surfaces 3.0%, and gravel and earth surfaces 4.0%. Uganda manual recommends that the normal cross fall for paved carriageway on tangent sections and on very flat curves with larger radius, should be 2.5% maximum. For areas of intense rainfall the cross fall of 3% may be accepted. On high type two-lane carriageway the crown slope of as low as 2 percent is accepted for all other conditions. The normal cross fall for unpaved roads should be 4%.

We recommend the following cross fall for EAC roads network:

- *2.5% for asphalt concrete surfaces (or 3% in areas where heavy rainfall is common)*
- *Surface dressing surfaces 2.5%*
- *Stone paved surfaces 3.0%*
- *Gravel and earth surfaces* 4.0%

Side Slope and Back Slope

Side slopes should be designed to ensure roadway stability and provide a reasonable opportunity for recovery for an out-of-control vehicle. Three regions of the side slopes are important to safety: the top of the slope (hinge point), the fore slope, and the toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch). The recommended maximum rates of side slopes for embankment (fills) and for back slope are the same in the Tanzania and Uganda. In Kenya, the values vary between 1:0.10 to 1:4 depending on the height of cut and the type of materials. We recommend that for side slopes steeper than 1V:3H, roadside barrier should be used; Back slopes should be flatter than 1V:3H to accommodate maintenance equipment; and that slopes steeper than 1V:3H should be evaluated for slope stability and traffic safety.

Clear Zone

The term clear zone is used to designate the unobstructed, relatively flat area provided beyond the edge of the roadway for the recovery of errant vehicles. It includes any shoulders or auxiliary lanes. Clear zone widths are related to speed, volume, embankment slope, and horizontal geometry. The need for clear zones increases with speed and curvature. Clear zone widths vary throughout the world depending on land availability and design policy. Design guides for Tanzania and Uganda suggested clear zone widths design values in relation to speed limits. The design guide for Kenya reported a clear zone of 3 m or more from the edge of the carriageway. At existing pipe culverts, box culverts and bridges, both manuals proposed clearance not to be less than the roadway width. We recommend that clear zone width design values shown in the table below should be adopted for EAC and the values should be increased at sharp bends on high speed roads by a correction factor.

Speed limit	Standard	
	Desired (m)	Minimum (m)
70	5	3
80	6	4
100	9	6

Median

A median is the portion of a roadway separating opposing directions of the travelled way. Medians are highly desirable on multilane divided arterials carrying four or more lanes. Median width is expressed as the dimension between the edges of travelled way and includes the left/right shoulder (depending on whether driving is on the right/left, respectively), if any. The main functions of a median are to separate opposing traffic, provide recovery area for out-of-control vehicles, provide a stopping are in case of emergencies, allow space for speed changes and storage of left/right-turning (depending on whether driving is on the right/left, respectively) and U-turning vehicles, minimise headlight glare, provide width for future lanes, and in urban areas to offer an open green space, refuge area for pedestrians crossing the street, and control the location of intersection traffic conflicts. It may be depressed, raised, or flush with the travelled way surface.

Review of standards showed that consensus median width values have not been reached for any category of roadway. It was found that minimum freeway median width ranges from 1.5 m to over 4.5 m, while for arterials ranges from 1.2 to over 20.0 m. In Tanzania, installation of a median barrier is required where the median is less than 9.0 m wide and the speed is high. The minimum width of median is as narrow as 1.2 to 1.8 m. In Uganda a minimum median width is required to allow the provision of right-turning lanes outside of the adjacent carriageway. Four lanes and divided roads are required when the design traffic volume is sufficient to justify their use. The minimum width of median is as narrow as 1.2 to 1.8 m. Where provision for right turn lane is required the minimum width of median will be 4.8 – 5.0 m. In some cases for future upgrading of the road a central reserve of minimum 12.0 m could be introduced to serve as median in Uganda. The manual for Kenya does not cover minimum median width design values. The above values are within the recommended median widths by AASHTO. We recommend median widths shown in table below for speed limits of 80 km/h and 100 km/h.

Speed limit	Median Widths	
	Desirable (m)	Minimum (m)
100	12	6
80	9	4

Service Roads

A service road (also known as access road or frontage roads) serves numerous functions, depending on the type of arterial they serve and the character of the surrounding area. They may be used to control access to the arterial, function as a street facility serving adjoining properties, and maintain circulation of traffic on each side of the arterial. Service roads segregate local traffic from the higher speed through-traffic and intercept driveways of residences and commercial establishments along the highway. AASHTO (2004) recommends a minimum spacing of about 50 m between the arterial and the service roads in urban areas. The design of service road is influenced by the type of service it is intended to provide. Design manuals for Tanzania and Uganda proposed that for the larger trading centres and towns service roads should be provided and typically be of 6.0 m wide while this design aspect is not covered in Kenya design guide. We propose service roads of 6.0 m wide for larger trading centres and towns, however, lower widths may be adopted depending on the intended function of the service road.

Headroom and Lateral Clearance

Since the SADC harmonised guidelines for vehicle dimensions specify the maximum height allowed for vehicles is a double-deck bus not exceeding 4.65 m, the vertical clearance should be above this figure, with some allowance for roadway resurfacing. Though there are some similarities, design manuals used in the EAC showed some variances in headroom and lateral clearance standards. We recommend

- 5.5 m as headroom under bridge structures, 7.0 m as headroom under high-power cables and 6.0 m under low-power cables.
- minimum lateral clearance (i) width of 0.6 m on each side of a bridge for traffic volume of less than 400 veh/day, (ii) width of 1.0 to 1.2 m on each side of a bridge for traffic volume of 400 to 2000 veh/day, and (iii) width of approach roadway width for traffic volume of over 2000 veh/day (and for bridges over 30 m span, the width should be increased by 1 m).

Emergency Escape Ramps

Emergency escape ramps are not covered by the existing design manuals for the EAC members states. The design and construction of an emergency escape ramp at an appropriate location is desirable to provide a location for out-of-control vehicles, particularly trucks, to slow and stop away from the main traffic stream. Out-of-control vehicles are generally the result of a driver losing braking ability either through overheating of the brakes due to mechanical failure or failure to downshift at the appropriate time. Each grade has its own unique characteristics. Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. For existing highways, operational concerns on a downgrade will often be reported by law enforcement officials, truck drivers, or the general public. A field review of a specific grade may reveal damaged guardrail, gouged pavement surfaces, or spilled oil indicating locations where drivers of heavy vehicles had difficulty negotiating a downgrade. For existing facilities, an escape ramp should be provided as soon as a need is established. Crash experience (or, for new facilities, crash experience on similar facilities) and truck operations on the grade combined with engineering judgment are frequently used to determine the need for a truck escape ramp. Often the impact of a potential runaway truck on adjacent activities or population centres will provide sufficient reason to construct an escape ramp.

We recommend that an escape ramp or ramps, if the conditions indicate the need for more than one, should be located wherever grades are of steepness and length that present a substantial risk of runaway trucks and topographic conditions permit construction.

Motorcycle Lanes

There are two types of motorcycle track: restricted track and exclusive track. Restricted motorcycle track can be developed within the carriageway of an existing road. It is usually sited on the left side of the road, preferably between the kerbs and the parking lane. Some form of physical barrier or pavement markings defines the corridor set aside for the cyclists and route markings are necessary to define the route and reduce potential conflicts. However, at crossings and intersections, this kind of cycle track ceases as a separate mode and conflicts may occur with other forms of movement. An exclusive motorcycle track is a complete separate right-of-way established for the sole use of cyclists. This kind of cycle track separates the cyclists from other motorists. This type of exclusive cycle track differs from the restricted cycle track in that it normally has a wide right-of-way and is not developed from existing carriageway of a wide road. It helps to separate conflicts at crossings and intersections with the provision of underpasses and other related facilities.

Thus, the following warrants are recommended for determining the need for an exclusive cycle lane: the total volume of traffic exceeds the provided lane capacity, and the volume of motorcycles exceeds 20% of the total volume of traffic as indicated in Annex A Chapter 2.

Intersections

The Distance between Intersections

For safe and efficient traffic flow, intersections should not be placed too close together. Drivers accelerating away from one intersection are not expecting to encounter traffic slowing for another intersection, for example. Also a queue of vehicles from one intersection that blocks another intersection, called spill-back, will cause congestion to propagate and cause extra delay. The design manuals for Tanzania and Uganda recommends that the minimum distance between consecutive junctions shall preferably be equal to (10 x VD) meters; where VD is the major road design speed in km/h. The manuals also indicate that the minimum spacing for signalised intersections is 400 m.

Since proper intersection spacing, particularly for signalized intersections, is critical for providing coordinated signal timing, optimal timing progression for two-way movements should allow travel time between intersections to be about half of the cycle length. We therefore recommend that signal spacing should be at least 400 m in urban areas and 800 m in suburban areas for optimum two-way progression, and for unsignalised intersections the minimum distance should be at least 10^*V_D *meters.*

Location

Intersections are safer in some locations than others. This aspect of location of intersection is not covered by the existing design manuals for EAC Partner States. We suggest that intersections should neither be on a horizontal curve if possible nor near a crest vertical curve.

Angle

The angle of intersection is important to operations. A right angle intersection provides the most favourable conditions for intersecting and turning traffic movements. Specifically, a right angle (90°) provides (a) the shortest crossing distance for motor vehicles, bicycles, and pedestrians, and (b) sight lines which optimize corner sight distance and the ability of drivers to judge the relative position and speed of approach vehicles. Minor deviations from right angles are generally acceptable provided that the potentially detrimental impact on visibility and turning movements for large trucks can be mitigated. However, large deviations from right angles may decrease visibility, hamper certain turning operations, and will increase the size of the intersection and therefore crossing distances for bicyclists and pedestrians. This element of intersection design is not covered by the existing design manuals for EAC Partner States. We suggest that angle of existing intersections on EAC road network should be checked for possible implementation of retrofitting improvement strategies, if necessary.

Warrant for Turning Lanes

Turning lanes provide room for left-turning or right-turning vehicles to decelerate before their turns and/ or to queue while waiting to turn. They are particularly effective at reducing delay and collisions by getting those vehicles out of the way of through vehicles. At busy signalized intersections, dual and triple turning lanes are used effectively to reduce the time that those vehicles need the right-of-way. Dual left-turn (left side driving) lanes are also used at some intersections. The drawbacks to using turning lanes include higher right-of-way costs and longer crossing distances for pedestrians.

A comparison of the above criteria reveals that AASHTO criteria for installing turning lanes are simple and require data which is easily obtainable at site and can be directly applied for site assessment, whereas criteria that are currently used in the Tanzania and Uganda require conversion of daily volumes into annual average daily volumes and information concerning passenger car equivalent units. We therefore recommend use of the following AASHTO criteria for provision of turning lanes.

Opposing volume (veh/h)	Advancing volume (veh/h)			
	5% left turns	10% left turns	20% left turns	30% left turns
60 km/h operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
80 km/h operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
100 km/h operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Grades

Steep grades hamper traffic operations at intersections. Steep downgrades on an intersection approach increase stopping distances and make turning more difficult. Steep upgrades on an intersection approach make idling difficult for vehicles with manual transmissions and make acceleration slower for all vehicles, which in turn increases necessary gap sizes and sight distances for crossing and turning movements. Review of international practice revealed that grades under 2 percent do not cause many operational problems, grades from 2 to 4 percent begin to introduce noticeable problems, and grades over 4 percent should be avoided where practical. This element of intersection design is not covered by the existing design manuals for EAC Partner States. We recommend that intersections should be at locations that have grades of less than 4%.

Grade Separations and Interchanges

Warrants for grade separations and interchanges

With increasing traffic volumes, a point will be reached where all the options of temporal separation of conflicting movements at an at-grade intersection have been exhausted. The elimination of bottlenecks by means of interchanges can be applied to any intersection at which demand exceeds capacity and is not necessarily limited to arterials. Under these circumstances, it is necessary to weigh up the economic benefits of increased safety, reduced delay and reduced operating and maintenance cost of vehicles against the cost of provision of the interchange. The latter includes the cost of land acquisition and the cost of construction.

Generally, the justification of an interchange at a given location is difficult due to the wide variety of site conditions, traffic volume, highway types and interchange layouts. AASHTO (2011) provides six warrants that should be considered when determining if an interchange is justified at a particular site. The design guides for EAC also covers these warrants. Thus, we recommend to EAC members states use of the AASHTO warrants and undertaking of benefit cost analysis for justification for grade separation.

Spacing

If an interchange is warranted for any of the listed reasons, interchange spacing is an additional consideration in the decision-making process. Interchange spacing has a pronounced effect on freeway operations. In areas of concentrated urban development, proper spacing usually is difficult to attain because of traffic demand for frequent access. Minimum spacing of arterial interchanges (distance between intersecting roads with ramps) is determined by weaving volumes, signal progression, and lengths of speed-change lanes. AASHTO (2004) gives a general rule of thumb for minimum interchange spacing of 1.5 km in urban areas, and 3.0 km in rural areas (between freeway-to-freeway interchanges and local roads interchanges). In urban areas spacing of less than 1.5 km may be developed by grade-separated ramps or by adding collector-distributor roads. This element of intersection design is not covered by the existing design manuals for EAC Partner States. We recommend a rule of thumb for minimum interchange spacing of 1.5 km in urban areas and 3.0 km in rural areas.

Railway – Highway Crossing

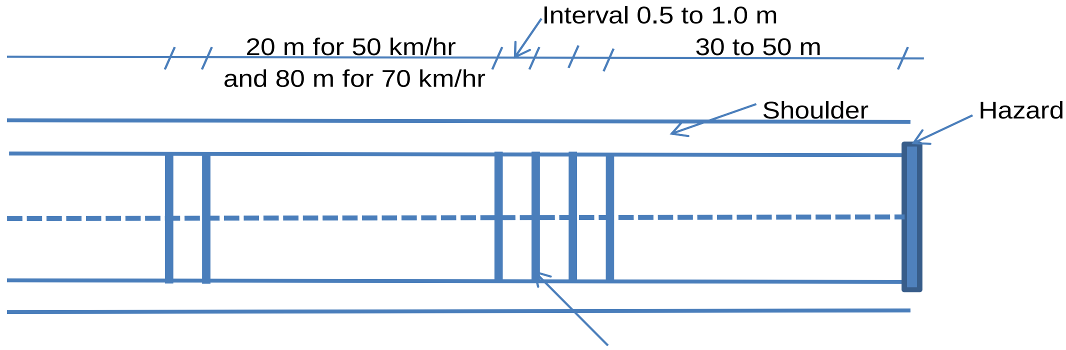
A railroad-highway crossing, like any highway-highway intersection, involves either a separation of grades or a crossing at-grade. The geometrics of a highway and structure that involves the overcrossing or undercrossing of a railroad are substantially the same as those for a highway grade separation without ramps. The horizontal and vertical geometrics of a highway approaching a railroad grade crossing should be constructed in a manner that facilitates drivers' attention to roadway conditions. We recommend that the decision to grade separate a highway-rail crossing is primarily a matter of economics. Investment in a grade separation structure is long-term and impacts many users. Such decisions should be based on long-term, fully allocated life-cycle costs, including both highway and railroad user costs, rather than on initial construction costs. Such analysis should consider the following:

1) Eliminating train/vehicle collisions (including the resultant property damage and medical costs and liability).
2) Savings in highway-rail grade crossing surface and crossing signal installation and maintenance costs.
3) Driver delay cost savings.
4) Costs associated with providing increased highway storage capacity (to accommodate traffic backed up by a train).
5) Fuel and pollution mitigation cost savings (from idling queued vehicles).
6) Effects of any "spillover" congestion on the rest of the roadway system.
7) Benefits of improved emergency access.
8) Potential for closing one or more additional adjacent crossings.
9) Possible train derailment costs.

Speed Management

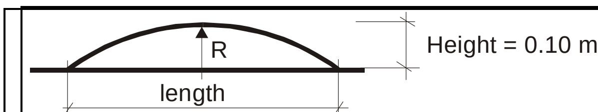
Speed management encompasses a range of measures aimed at balancing safety and efficiency of vehicle speeds on a road network. It aims to reduce the incidence of driving too fast for the prevailing conditions, and to maximize compliance with speed limits. Speed management aims to reduce the number of road traffic crashes and the serious injury and death that can result from them. Numerous practices have been implemented in urban areas to reduce speeds.

The design manuals for Tanzania and Uganda recommend the use of gates, humps, and rumble strips for speed control. Both manuals give the same design standards for gates, humps (circular and plateau/flat-topped), and rumble strips. The following figure and principles are to be observed when using rumble strips:

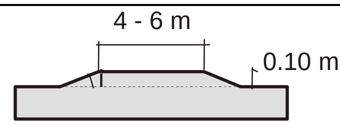


- Rumble strips should normally be in groups of 4 strips
- The height of the strips shall be no more than 10 – 15 mm
- The strip width should be 0.5 m
- One set of rumble strips is usually enough within 50km/h sections
- The last or only strip should be located 30 to 50 m before the hazard
- Pre-warning sets can, if used, be located 20 to 80 m before the hazard depending on speeds
- Rumble strips should preferably have yellow thermoplastic lines across the top for better visibility
- Strips should continue across the full width of the carriageway, including the shoulders but be terminated so that they do not interfere with drainage.

Road bumps are 75 – 100 mm high and 300 – 900 mm long and are typically used in parking lots and on private roads. To pass over road bumps without doing damage to the vehicle or causing discomfort, the driver must slow down almost to a complete stop. They are suitable for residential areas but are not acceptable and should not be used on national roads i.e. Trunk and Regional roads. Road humps are 75 – 100 mm high and 4.0 – 9.5 m long. They may be used on National Trunk Roads and Regional Roads where proven to be absolutely necessary. Road humps should not be used on International Trunk Roads. There are two main types of road humps i.e. circular, which are intended for traffic speed reduction only and flat-topped humps, which are intended for speed reduction and for use as a pedestrian crossing. The figures below illustrate the recommended standards for speed humps.



Vehicle speed(km/h)	Radius (m)	Length (m)
20	11	3.0
25	15	3.5
30	20	4.0
35	31	5.0
40	53	6.5
45	80	8.0
50	113	9.5
55	180	12.0



Vehicle speed(km/h)	Grade i (%)
	14
25	12.5
30	10
35	7.5
40	6
45	5
50	4
55	3
60	2.5

In addition, we recommend that other speed control measures other than the speed humps and rumble strips should be tried on major roads to reduce vehicle speeds. Such measures may include narrowings, raised zebra crossings, and use of combination of measures.

Pedestrian and Cyclists Facilities/ NMT Facilities

Pedestrians facilities

Pedestrian facilities include sidewalks, crosswalks, traffic control features, bus stops or other loading areas, sidewalk on grade separations, and the stairs, escalators, or elevators related to these facilities, and curb cuts (depressed curbs and ramped sidewalks) and ramps for older walkers and persons with mobility impairments.

In the past the needs of pedestrians were largely ignored, and this may be one reason why so many pedestrians are killed and injured on our roads. Pedestrians have as much right to use the road as motorists, and roads must be designed with their needs in mind. The first step is to identify major pedestrian generators (e.g. markets, shops, and schools) and determine which the most important pedestrian routes are. The aim should be to develop a network of pedestrian routes and crossing facilities that is convenient to use and avoids conflicts with vehicular traffic.

Shoulders and footways

Both manuals for Tanzania and Uganda recommended a shoulder of 1.5 m wide for pedestrian in rural areas as pedestrian can walk on the road shoulders. The conventional view is that pedestrians in rural areas can walk on the road shoulders. The shoulder should be at least 1.5 m wide, though 1 m is just acceptable if there are constraints. The surface must be well drained and be as smooth as the traffic lanes – if not, pedestrians may prefer to walk in the traffic lane. The implication of this is that low-cost chip seal shoulders may not be a good investment. Letting pedestrians use the shoulders is not entirely satisfactory, as there is nothing to protect the pedestrian from speeding traffic. This is of particular concern on high-speed and / or high volume roads. In these situations it is preferable to provide a separate footway several metres beyond the edge of the shoulder – and separated from it by a grass strip. Some criteria for the provision of footways are given in Table 46 but these should be used with caution – in some circumstances footways can be justified at lower pedestrian flows.

Location of footway	Average daily vehicle traffic	Pedestrian flow per day	
		Speed limit of 60 – 80 km/h	Speed limit of 80 – 100km/h
One side only	400 to 1,400	300	200
	> 1,400	200	120
Both sides	700 to 1,400	1,000	600
	> 1,400	600	400

Standard footway widths are:
Absolute minimum: 1 m (two persons cannot pass each other)
Desirable minimum: 1.8 m (two persons can pass each other closely)
Light volume: 2.25 m (two persons can pass each other comfortably)
Heavy volume: 3.5 m+ (space for three persons)

In urban areas, however, the manual for Tanzania recommends a 150-200 mm high barrier kerbs and that higher kerbs of 250 mm can also be used in order to deter vehicles from parking on the footway (but are not recommended for general use) while the manual for Uganda recommends a barrier kerbs of 100-150 mm high with higher kerbs of 200 mm. AASHTO suggests use of kerbs of 100 – 200 mm high.

Pedestrian bridges and underpasses

The manual for Tanzania recommends dimensions for underpasses which are different from those documented by the manual for Uganda. Tanzania recommends a minimum height of 2.5 m and 3.0 m for short (< 15 m) and long (> 15 m) underpasses, respectively, while Uganda recommends heights of 2.3 m and 2.6 m for the respective underpasses. Moreover, Tanzania recommends a vertical clearance of 5.5 m (and absolute minimum of 5.2 m) for pedestrian bridge above the carriageway surface while Uganda recommends a clearance of 5.0 m. The recommended flights of stairs (between landings) is limited to 12 steps for the case of Tanzania while for Uganda the number of steps for flights of stairs (between landings) are limited to 20 steps (9 steps where there are significant numbers of disabled persons). It is recommended that the headroom under pedestrian bridges should be 5.5 m.

Cycle facilities

The conventional view is that cyclists in rural areas can use the shoulders, and this is acceptable provided that the combined volume of pedestrians and cyclists is low (<400 per day) and the shoulder is at least 1.5 m wide. But, with heavier flows, and especially if there is high-speed traffic and / or a high proportion of heavy goods vehicles, it will be better to provide a separate cycleway or a combined cycleway and footway. The design manuals for Tanzania and Uganda recommend similar dimensions for widths for cycle facilities and clearances to wall, fence, barrier or other fixed object. They specify minimum clearance to edge of traffic lane for various speed limits. It is also recommended to provide separate cycleway or combined cycleway and footway if there is high-traffic and the combined flow of pedestrians and cyclists is more than 400 per day. Cycle ways need to have a smooth surface with good skid resistance. It is recommended that cycleways of width shown in the table below be provided as separate cycleway or combined cycleway and footway if there is high-traffic and the combined flow of pedestrians and cyclists is more than 400 per day. It is also recommended that minimum clearances shown in the table below be adopted for cycleways.

Recommended widths for cycle facilities

Type	Minimum width (m)	Standard width (m)	Width for heavy usage (m)
Cycleway (separate from carriageway)	2.0	2.5	3.5
Combined cycleway and footway	2.0	3.0	4.5
Cycle lane (one way)	1.5	2.0	2.5

Recommended clearances for cycle facilities

Type	Recommended clearance [m]
Minimum overhead clearance	0.50
Clearance to wall, fence, barrier or other fixed object	0.50
Clearance to unfenced drop-off, e.g. embankment, river, wall	1.0
Minimum clearance to edge of traffic lane for speed limit of:	
50 km/h	0.50
80 km/h	1.00
100 km/h	1.50

Traffic control devices and other road furniture

Traffic control devices and other road furniture like guardrails and marker posts are intended to improve the driver's perception and comprehension of the continually changing appearance of the road and to control and guide traffic. Traffic control devices include traffic signs, road markings and traffic signals which are described and harmonised in Annex A6.

Other Facilities

Marker posts

They are intended to make drivers aware of potential hazards. Only the draft design manual for Tanzania recommends the marker posts to be sited at 0.25 m outside the edge of the shoulder. We recommend that marker posts should be sited at 0.25 m outside the edge of the shoulder.

Kilometre Posts

These are blocks or pillars of concrete set up beside major roads to show distances from that point to town centres or major settlements along the road. They show the distances to the destination and that of the origin placed in such a way that the road user will only immediately see the distance to the destination. Only the design manual for Tanzania included kilometre posts. We recommend that kilometre posts should be installed along the whole road at an interval of distance of 5 kilometres from each other and they should be placed in stagger thus forming a 10 km interval on each side of the road.

Road Reserve Marker Posts

Numerous unauthorised accesses tend to develop within the road reserve area. This unwanted development can be limited by providing proper demarcation of the boarder of the road reserve. Road reserve marker posts are recommended by the draft design manual for Tanzania. The posts are recommended to be erected on both sides of the road at intervals of 100 m from each other when traversing inhabited areas and at 300 m on other areas. Whenever new villages are formed along the roads, additional road reserve marker posts should be erected to meet the 100 m interval.

We recommend that road reserve marker posts should be erected on both sides of the road at intervals of 100 m from each other when traversing inhabited areas and at 300 m on other areas. Whenever new villages are formed along the roads, additional road reserve marker posts should be erected to meet the 100 m interval.

Rest Areas

Rest areas are of three types of rest areas: major, minor and truck parking bays. Major rest areas are designed for long rest breaks, offering a range of facilities and separate parking areas for heavy and light vehicles. These are designed to allow drivers to take rest and sleep breaks required under current driving hours regulations. Minor rest areas are designed for shorter rest breaks, and at a minimum should provide sufficient parking space for both heavy and light vehicles. While it is not anticipated that these stops will be used for long rest breaks/sleep opportunities, separate parking areas for heavy and light vehicles may be required at some locations. Truck parking bays areas are primarily designed to allow drivers of heavy vehicles to conduct short, purpose-based stops including load checks, completing logbooks and addressing associated operational needs. Tanzania design manual provides requirements which should be taken into account when planning for rest areas.

We therefore recommend that major rest areas should be located at maximum intervals of 100 – 120 km, minor rest areas should be located at maximum intervals of 50 – 60 km, and truck parking bays should be located at maximum intervals of 30 – 40 km. additionally, planning for rest areas should take into account the following requirements, as elaborated in Annex A Chapter 2.

a) Detailed rest area strategy
b) Spacing intervals
c) Proximity to a town
d) Location
e) Proposed Layout
f) Pedestrian Access and Visibility
g) Speed
h) Access
i) Minimum Signage Requirements
j) Minimum Facilities
k) Additional Facilities
l) Cross-Border Compatibility

Impact of harmonisation and Implementation strategy

Implementation of the above recommendations will ensure uniform roadway geometric standards and hence the optimum balance between road infrastructure construction cost and road user cost is obtained, considering road safety issues and natural and human environmental aspects in the EAC Region. Other impacts of the above harmonised regimes include improved efficiency of the road transport system by minimisation of road crashes and energy consumption. It should be noted that this study also revealed that the width, height, wheelbase and minimum turning radius dimensions of vehicles such as the 22 m Interlinks, which have recently been permitted to traverse EAC road network, conform to the respective dimensions of the design vehicles documented in the existing design manuals for EAC member countries and therefore will be accommodated by the existing intersections on the EAC road network. On the flip side, all countries will need to check whether headroom under bridge structures complies with the recommended value of 5.5 m and make improvements, if necessary. Additionally, it is proposed that the recommended standards be implemented in all new road designs, and road rehabilitation, reconstruction and widening projects.

2 HARMONISATION OF ROADWAY GEOMETRIC DESIGN STANDARDS

2.1 Introduction

2.2.1 General

The geometric design of roadways deals with the dimensions and layout of visible features of a roadway with the objective of creating the roadway facility to the characteristic and behaviour of drivers, vehicles, traffic and terrain. Geometric design is based on specified design standards and controls which depend on a wide range of roadway system factors such as roadway functional classification, terrain of the area that the roadway traverses, levels of service, road user characteristics, vehicle characteristics (length, width, height, wheelbase, weight, including acceleration and deceleration characteristics and maximum speed) and design hourly traffic volume, sight distances and choice of horizontal and vertical alignment, road cross section, intersection type, available funds, safety, and social and environmental factors.

These factors are often interrelated. For instance, design speed depends on the functional classification which is usually related to expected traffic volume. The design speed may also depend on the terrain, particularly in cases where limited funds are available. In most cases, the principal factors used to determine the standards to which a particular roadway will be designed are the level of service to be provided, expected traffic volume, design speed, and the design vehicle. These factors, coupled with the basic characteristics of the driver, vehicle, and road, are used to determine standards for the geometric characteristics of the roadway, such as cross sections and horizontal and vertical alignments. Appropriate geometric design standards should be selected to maintain a desired level of service for a known proportional of different types of vehicles.

The use of geometric design standards fulfils three inter-related objectives. Firstly, standards are intended to provide minimum levels of safety and comfort for drivers; secondly, they provide the framework for economic design; and thirdly, they ensure a consistency of alignment. As noted above, the design standards adopted must take also into account the environmental road conditions, traffic characteristics, and driver behaviour.

It is therefore expected that a code of practice for or manual on roadway geometric design should include the following important aspects for proper roadway geometric design.

- Roadway functional classification.
- Design controls and criteria: these cover the characteristics of vehicles, road users, and traffic that act as criteria for the optimisation or improvement in design of the various road classes.
- Design elements: these are principal design elements which include sight distance, superelevation, horizontal and vertical alignments, and other elements of geometric design. They are joined together to create a facility that serves the traffic in a safe and efficient manner, consistent with the facility's intended function. Thus, each alignment element should complement others to produce a consistent, safe, and efficient design.
- Cross section elements.
- Intersections.
- Road furniture and other facilities.

2.1.2 Study Objectives

The overall objective of the assignment is to make it possible to have a reliable, efficient and safe road transport services.

This chapter gives recommendations about the roadway geometric design standards for the EAC region. It discusses established roadway geometric design standards within the EAC member countries as well as applicable SADC and other international standards, and makes recommendations for the EAC.

2.2 Geometric Design Standards in the EAC Partner States

The fundamental principles of geometric design were discussed in engineering textbooks as early as 1912; however, it was not until 1940 when the American Association of State Highway Officials (AASHO), later the American Association of State Highway and Transportation Officials (AASHTO), published seven documents, formally recognizing policies on certain aspects of geometric design. The 1940 AASHO policies were revised and amended in 1954, 1965, and 1971. They also were revised and amended by AASHTO in 1984, 1990, and 1994. AASHTO geometric design policies are based on the laws of physics and conservative assumptions in regard to the driver, vehicle, and roadway. Although some assumptions have changed, most of the basic models are the same as those in the 1940s.

The study found that Burundi and Rwanda follows French geometric road design standards though the latter has recently shifted to American standards. It also found that Kenya, Tanzania and Uganda are using their own standards which were developed largely from the American and English practice. Table 1 provides a summary of design standards that are used by EAC Partner States. It should also be noted that countries like Tanzania has adopted some design criteria from the geometric design standard of the Southern African Transport and Communications Commission (SATCC), which was also derived largely from the American and English practice.

Table 1: Geometric Design Standards in EAC Partner States

Country	Standard	Year	Remarks
Burundi	French geometric design standards		This study reviewed 1) Se'tra (2006) Understanding the principal geometric design parameters for roads; 2) Se'tra (1998) The design of interurban intersection major roads: at-grade intersections; and 3) Directorate of roads (2000) National instruction on technical design requirements for rural motorways
Kenya	Kenya road design manual: Part 1: Geometric Design of Rural Roads	1979	A new design manual is currently under preparation but was not available for this study.
Rwanda	French (AFNOR) and AASHTO geometric design standards		This study reviewed 1) Se'tra (2006) Understanding the principal geometric design parameters for roads, 2) Se'tra (1998) The design of interurban intersection major roads: at-grade intersections, 3) Directorate of roads (2000) National instruction on technical design requirements for rural motorways, and 4) AASHTO design guide (2011).
Tanzania (Mainland and Zanzibar)	Tanzania road geometric design manual (Draft): Part 1: Trunk and Regional Roads	2010	Draft manual to replace the 1989 design manual
Uganda	Uganda road design manual: Vol. 1: Geometric Design	2005	-

A review of the documents listed in Tables 1 revealed a number of areas of commonality as well as areas unique to particular countries. These areas are discussed in detail in the following section.

2.3 Potential Areas for Harmonisation and Improvement

2.3.1 Road Classification and Access Control

The classification of roads into different operational systems, functional classes or geometric types is necessary for communication between engineers, administrators and the general public. Classification is the tool by which a complex network of roads can be subdivided into groups having similar characteristics. A single classification system, satisfactory for all purposes, would be advantageous but has not been found to be practicable. Moreover, in any classification system the division between classes is often arbitrary and, consequently, opinions differ on the best definition of any class. There are various schemes for classifying roads and the class definitions generally vary depending on the purpose of classification. The principal purposes of road classification are to:-

- Establish logical integrated systems that, because of their particular service, should be administered by the same jurisdiction;
- Relate geometric design control and other design standards to the roads in each class, and
- Establish a basis for developing long range programmes, improvement priorities and financial plans

Classification of roads by design types based on the major geometric features (e.g. freeways) is the most helpful one for road location and design purposes. Classification by route numbering is the most helpful for road traffic operational purposes, whilst administrative classification is used to denote the level of government responsible for, and the method of, financing road facilities. Functional classification, the grouping of roads by the character of service they provide, was developed for transportation planning purposes.

Roads have two functions: to provide mobility and to provide land access. However from a design standpoint, these functions are incompatible. For mobility, high or continued speeds are desirable and variable or low speeds undesirable; for land access, low speeds are desirable and high speeds undesirable. Given a functional classification, design criteria can be applied to encourage the use of the road as intended. Design features that can convey the level of functional classification to the driver include width of roadway, continuity of alignment, spacing of intersections, frequency of access points, building setbacks, alignment and grade standards, and traffic controls. Thus, the functional classification system is most commonly used as a design type of highways. The three primary road classifications within this functional hierarchy are arterials (principal and minor arterials), collectors, and local roads and streets. There are also varying stages within each of these classifications and more particularly within the arterial class. On the basis of this understanding, the most widely accepted design type criteria are those developed by AASHTO which classify a road system into:

- Arterial roads
- Collector roads
- Local streets and roads

For each classification, specific design standards and criteria and other policies have been developed and are applied. Within the arterial classification there are principal and minor arterials while there are major and minor collectors as well. Freeways are not listed as a separate functional class since they are generally classified as part of the principal arterial system. However, they have a unique geometric criteria that require special design consideration and hence their inclusion as a separate class.

Generally, freeways are principal arterial roads with full control of access, i.e., access to and egress from these facilities is permitted only at controlled locations such as an entrance and exit ramps. They are intended to provide for high levels of safety and efficiency in the movement of large volumes of traffic at high speeds and therefore crossing at grade and direct private driveway connections is prohibited. Expressways are multilane, divided principal arterial roads with partial control of access, i.e., access or egress may also be permitted directly from or to abutting land or via a limited number of at-grade intersections. Highways are multilane undivided arterials while minor arterials are two-lane roads. Collectors combines aspects of both arterials and local streets, they collect traffic for movement between arterial streets and local roads and provide access to abutting land. A tow-lane road system constitutes an important part of rural collectors. Local roads or streets provide access to farms, residences, businesses or other abutting properties. They may be two-lanes or single lane.

The two major considerations in classifying highway and street networks functionally are access and mobility. The conflict between serving through movement and providing access to a dispersed pattern of trip origins and destinations leads to the differences and gradations in the various functional types. The extent and degree of access control is thus a significant factor in defining the functional category of a street or highway. The Final Report by Africon Ltd (2011) on the EAC Transport Strategy and Regional Roads Sector Development Program, noted that the current road classifications that are in place within the EAC member states and which focus on high mobility roads (Class 1-3) are similar in terms of their function. The review by Africon Ltd (2011) of best practice in road classification by various international authorities is shown in Table 2.

Basing on the international practice, Africon Ltd (2011) recommended to the EAC member states the categories of High Mobility-Class 1 and High Mobility – Class 2 for the EAC corridor road classification and numbering. High mobility – class 1 roads are roads which movement is dominant, through traffic is dominant, the majority of traffic does not originate or terminate in the immediate vicinity, the function of the road is to carry high volumes of traffic between urban districts, across and through member states; connecting member states; freight focused and freight supportive. Class 2 roads are corridor feeder roads with its main function that of supporting the EAC corridors, i.e. Class 1 roads.

In accordance with the international practice, mobility roads include principal arterial roads that provide the highest level of mobility and the highest speeds over the longest uninterrupted distance. They also include multilane, two-lane two ways arterials, divided or undivided roads which connect or support, as directly as practicable the principal arterial systems. Lower classes of mobility roads provide less mobility and therefore balance both functions of mobility and accessibility, whereas access roads provide limited mobility and are the primary access to residential areas, businesses, farms, and other local areas.

Table 2: Road Classification Systems in other Countries

Country		South Africa			USA				International		
Function	Class.	RCAM 2010	RAM 2005	RISFSA 2006	AASHTO 1973, 2004, FHA 1989: both urban and rural	ITE 1988, NCHRP 1992	TRB 2003: Access Manual: State	TRB 2003: Access Manual: Local	Great Britain ordnance survey, DfT, HA	Australia	New Zealand
Mobility (arterial)	1	Principal arterial	Principal arterial	Primary distributor	Principal arterial system, interstate	Freeway, expressway	1 Interstate highways and other freeways	1 Freeway	Motorway, trunk roads	Freeway, M-route, state road, national highway	National road
	2	Major arterial	Major arterial	Regional distributor	Principal arterial system, other	Strategic arterial, principal arterial, primary arterial	2 Roadway of state-wide importance (strategic, principal arterials)	2 Major arterial	A-road, principal road	Major arterial, A-route, regional road	Primary (regional) arterial
	3	Minor arterial	Minor arterial / activity arterial - spine	District distributor	Minor arterial road / street system	Secondary arterial	3 Roadway of regional importance (other arterials)	3 Minor arterial	B-road	Minor arterial, B-route, district road, sub-arterial	Secondary (district) arterial
Access / Activity (street)	4	Collector (street) *(4a commercial, 4b residential)*	Activity street	District collector	Major collector road / street system, minor collector roads	collector	4 District roadway (collector)	4 Major collector, 5 Minor collector	C-road, minor roads	Collector, C-route	Collector road
	5	Local street *(5a commercial, 5b residential)*	Residential street	Access roads	local road / street system	local, cul-de-sac	5 Local	6 Local	Local street, alley, private road with or without public access	local, lane	Local road
	6	Walkway *(6a pedestrian priority, 6b pedestrian only)*	Non-motorised	Non-motorised access ways	Terminal and transfer facilities				pedestrianised street with restricted access		

Notes: AASHTO 1973 A policy on Design of Urban Highways and Arterial Streets 1973 (red book)
AASHTO 2004 A policy on Design of Urban Highways and Arterial Streets 2004 (green book)
FHA 1989 Highway functional classification, USD-DOT, Federal Highway Administration, March 1989
GB, Aus, NZ Assessing the feasibility of a National Road Classification, Intergovernmental Committee on Surveying and mapping, Australia, Oct 2006
ITE 1988 Transportation and Land Use Development, ITE, Stover and Koepke, 1988
RAM 2005 National Guidelines for Road Access Management in South Africa, October 2005 draft
RCAM 2010 National Guidelines for Road Classification and Access Management in South Africa, 2010
RISFSA 2006 Road Infrastructure Strategic Framework for South Africa, October 2006 (earlier version 2002)
TRB 2003 Access Management Manual, Transportation Research Board of the National Academies, 2003
Source: RCAM 2010 National Guidelines for Road Classification and Access Management in South Africa, 2010, cited in Africon Ltd (2011)

It can be noted from Table 3 that EAC Partner States are using the same functional classification system with very slight changes in the definition of classes C and D roads. This classification serves well road administrative purposes. In Kenya, roads are classified into five functional classes based on their functional characteristics. For design purposes the roads are classified into seven design classes upon which typical road cross-sections are defined, which in turn are classified based on AADT or DHV in year 10 in passenger car units (pcu). In Tanzania, there are three functional classes and five design classes of roads. The division into road design classes is partly governed by design traffic in the design year and the functional classification of the road. Further, geometric road cross section parameters and road reserve widths are specified for each road design class. On the other hand there are five functional classes and seven design classes of roads in Uganda. The division into road design class is governed by the design speeds in level, rolling and mountainous terrains as well as capacity in pcu per day. Further, other geometric design parameters including cross section dimensions are specified for each design class (Table 3).

But each partner state has its own road design classification system. It is therefore desirable to harmonise road classification system for design purposes. EAC road network can be classified as shown in Figure 1 in the order of increased accessibility and reduced speeds. It can be noted from the figure that as the accessibility of road increases, the speed reduces and as the speed increases mobility increases. EAC roads may be classified as mobility roads (class 1, class 2, and class 3) and access roads (class 4 and class 5).

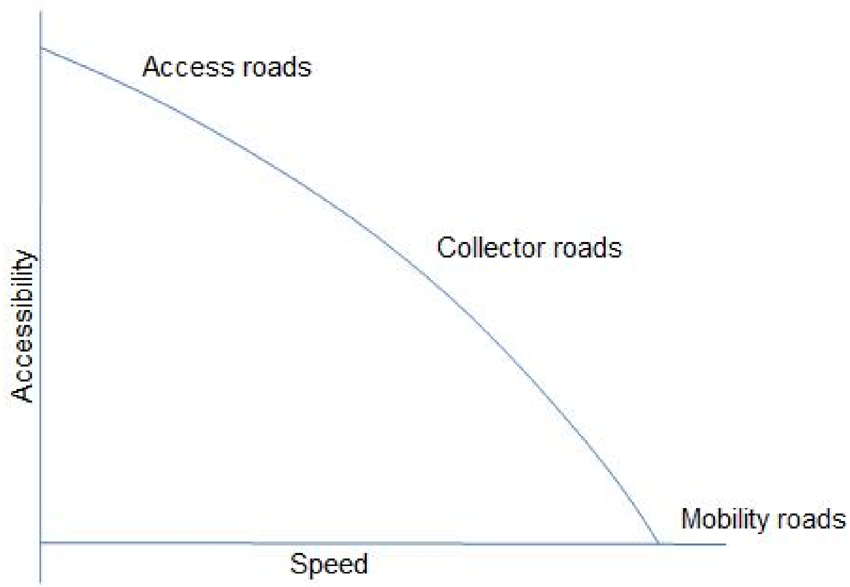


Figure 1**: Speed vs accessibility**

Table 3: Road Functional Classification in EAC Partner States

Level of access control	**Kenya**			**Tanzania (Mainland and Zanzibar)**			**Uganda**		
	Road class	**Design class**	**Road reserve**	**Road class**	**Design class**	**Road reserve**	**Road class**	**Design class**	**Road reserve**
Full	A: International trunk roads			A: International trunk roads	1, 2 & 3	60	A: International trunk roads	Ia, Ib, II, & III	50 – 60
Full	B: National trunk roads			B: National trunk roads	1, 2, 3, 4 & 5	60	B: National trunk roads	Ib, II, III & A gravel	40 – 60
Full or partial	C: Primary roads			C: Regional roads	3, 4 & 5	60	C: Primary roads	II, III & B gravel	40 – 50
Partial	D: Secondary roads			D: District roads	-		D: Secondary roads	A gravel & B gravel	30 – 40
Partial or unrestricted	E: Minor roads			E: Minor roads	-		E: Minor roads	B gravel & C gravel	30

Thus, Table 4 presents proposed road classifications in the EAC region. It shows the interrelationship between the existing and standard classification naming conventions for administrative and functional purposes. The table also shows the proposed spacing for the respective road classes as suggested by Africon Ltd (2011).

Table 4: Proposed functional classification of roads for the EAC

Function	Class	Name conventions		Access control
Mobility roads	Class 1	International trunk roads National trunk road	Principal Arterial/ Freeway/ Expressway (2500 m spacing)	Full
	Class 2	National roads Provincial roads Regional roads District roads	Major arterial/Highway (800 spacing)	Full or partial
	Class 3	Regional roads District roads Secondary roads	Minor arterial (600 m spacing)	Full or partial
Access roads	Class 4	District roads Secondary roads	Collector roads (50 m spacing)	Partial
	Class 5	Minor roads Local streets	Local streets (15 m spacing)	Unrestricted

Recommendation

For design purposes, we recommend to the EAC member states to adopt the following functional classification of EAC road network:

i) Mobility roads; classes 1, 2, and 3, and

ii) Access roads; classes 4 and 5, as shown in Table 4 above.

2.3.2 Road reserve

The width of right-of-way for complete development of an arterial road is influenced by traffic demand, terrain, land use, cost, intersection design, and the extent of ultimate expansion. It is the summation of the various cross sectional elements: through roadways, roadside clear zones, side slopes, drainage facilities, utility appurtenances, and retaining walls. It should be based on the preferable dimensions of each element to the extent practical in developed areas. Right-of-way widths in urban areas are governed primarily by economic considerations, physical obstructions, or environmental concerns. However, one factor that significantly affects the location of highways in urban areas is the cost of acquiring right of way. This cost is largely dependent on the predominant land use in the right of way of the proposed roadway. Costs tend to be much higher in commercial areas, and landowners in these areas are often unwilling to give up their property for road construction. Along any arterial route, conditions of development and terrain vary, and accordingly, the availability of right-of-way varies. For this reason, the right of way on a given roadway should not be a fixed width predetermined on the basis of the most critical point along the facility. Instead, every opportunity should be taken to provide a desirable right-of-way width along most, if not all, of the roadway.

It should also be noted that wide right-of-way permit the construction of gentle slopes, resulting in greater safety for the motorists and provide for easier and more economical maintenance. The acquisition of sufficient right-of-way, at the time of initial construction, permits subsequent widening of the roadway and the widening and strengthening of the pavement at reasonable cost as traffic volume increase. Freeways and expressways in urban areas have been placed on continuous elevated structures in order to avoid the acquisition of right of way and the disruption of commercial and residential activities. This method of design has the advantage of minimal interference with existing land use activities, but it is usually objected to by occupiers of adjacent land because of noise or for aesthetic reasons. The elevated structures are also very expensive to construct and therefore do not completely eliminate the problem of high costs.

AASHTO (2004) suggests that the width of right-of-way for a two-lane urban collector street should generally range from 12 to 18 m, depending on the conditions listed above. It also observes that typical range in widths of outer separations or border, which provide a buffer zone between the freeway and its adjacent area, is 25 to 45 m, but much narrower widths may be used in urban areas if retaining walls are employed. It is further noted that median widths of about 15 to 30 m are common on rural freeways. Garber and Hoel (2010) observes that the desirable minimum right of way for two-lane arterials is 25 m, 19 to 33 m for undivided four-lane arterials, whereas for divided arterials they range from about 36 to 90 m, depending on the numbers of lanes and whether frontage roads are included. The minimum right of way for freeways depends on the number of lanes and the existence of a frontage road. Levinson (1999) compared street spacing/design of city streets and suburban highways and suggested approximate right of way for freeway, strategic arterial, arterial, continuous collector, local access collector, and local street as 76 – 100 m, 46 – 61 m, 27 – 37 m, 18 – 27 m, 18 – 24 m, 15 – 18 m, respectively.

Right of way widths provided for by design manual for EAC Partner States (Table 2) are within the right of way widths recommended above by Levinson, and Garber and Hoel. However, it can be noted that the road reserve of 60 m recommended for international trunk roads is approximately below the minimum right of way width required for freeways.

Recommendation

EAC Partner States should plan to acquire incrementally more space for right of way in the range of 79 – 100 m so as to allow for future expansion of existing international and national routes to the freeway and expressways standards. Additional land should be acquired at all designated places for rest areas.

2.3.3 Design Control and Criteria

2.3.3.1 Design Vehicle and Vehicle Characteristics

The physical characteristics of vehicles and the proportions of the various sizes of vehicles using a road are positive controls in design and define several geometric design elements. In identifying a design vehicle considerations should take into account recent trends in motor vehicle sizes in the market and it should represent a composite of vehicles currently in operation. However, the dimensions of design vehicle are intended to represent vehicle sizes that are critical to geometric design. The dimensions used to define design vehicles are not averages or maxima, nor are they legal limiting dimensions. They are, in fact, typically the 85th percentile or 15th percentile value of any given dimension. The design vehicles are therefore hypothetical vehicles, selected to represent a particular vehicle class.

The principal dimensions affecting design are the wheelbase, width, height, and turning radius. The boundaries of the turning paths of each design vehicle for its sharpest turns are established by the outer trace of the front overhang and the path of the inner rear wheel. This assumes that the outer front wheel follows the circular arc defining the minimum centreline turning radius as determined by the vehicle steering mechanism. Trucks and buses generally require more generous geometric designs than do passenger vehicles. Single unit trucks and buses have smaller minimum turning radii than most combination vehicles, but because of their greater offtracking, the longer combination vehicles need greater turning path widths.

In general, buses and heavy vehicles should be used as the design vehicle for cross section elements, with the car as the design vehicle for the horizontal and vertical alignment. For most major intersections along arterial roads or within commercial areas, it is common practice to accommodate the semi trailer. The occasional larger vehicle may encroach on adjacent lanes while turning but not on the sidewalk.

Though there are some similarities, design manuals used in the EAC region present different dimensions of design vehicles. Burundi and Rwanda followed French design standards while the source of the design vehicle dimensions specified in the Kenya design manual is not known but it was assumed that they represented the vehicle market conditions in Kenya. The Tanzania manual indicated different dimensions of design vehicle with SATCC Code of Practice for the Geometric Design of Trunk Roads acknowledged as the source for the dimensions while the Uganda manual indicated dimensions of design vehicles with AASHTO Geometric Design Manual of Highway and Streets acknowledged as the source for the dimensions. Table 5 presents a summary of design vehicle dimensions used in the EAC region. As it can be seen from the table, design vehicle dimensions adopted by the EAC Partner States vary from one state to another.

As the result of using different sizes of design vehicles dimensions, each design manual presented templates for establishing the layout of intersections and median openings that correspond to their design vehicles. However, in constricted situations where templates are not appropriate, the capabilities of the design vehicle become critical. The Tanzania manual specified the use of SATCC minimum turning radii and AASHTO values were adopted by Uganda manual for the outer side of the vehicle to be used in constricted situations where the templates are not appropriate and the radii are only to crawl speeds. A comparison of minimum turning radii for the design vehicles is shown in Table 6.

Table 5: Comparison of the design vehicle characteristics

Vehicle	Wheel base (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car		2.9	3.1	3.4	
Single unit truck		6.5	6.1	6.1	
BUS			7.6	7.6	
Semi-trailer combination		4.8+8.4 = 13.2	6.7+3.4*+ 6.1	6.1 & 9.1	
WB-15			6.5+9.4		
Interstate semi-trailer				6.1 & 12.8	

Vehicle	Front overhand (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car		0.9	0.7	0.9	
Single unit truck		1.5	1.2	1.2	
BUS			2.1	2.1	
Semi-trailer combination		1.2	1.2	0.9	
WB-15			0.9		
Interstate semi-trailer				1.2	

Vehicle	Rear overhang (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car		1.2	1.0	1.5	
Single unit truck		3.0	1.8	1.8	
BUS			2.6	2.4	
Semi-trailer combination		0.6	1.8	0.6	
WB-15			0.6		
Interstate semi-trailer				0.9	

*distance between SU rear wheels and trailer front wheels. Note: blank cells mean that the dimensions were not available or are not covered by the manual

Table 5: (Continues)

Vehicle	Width (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car (P)		1.9	1.8	2.1	
Single unit truck (SU)		2.5	2.5	2.6	
BUS			2.6	2.6	
Semi-trailer combination		2.5	2.5	2.6	
WB-15			2.5		
Interstate semi-trailer				2.6	

Vehicle	Height (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car (P)		-		1.3	
Single unit truck (SU)		3.8		4.1	
BUS				4.1	
Semi-trailer combination		3.8		4.1	
WB-15					
Interstate semi-trailer				4.1	

Vehicle	Length (m) Burundi	Kenya	Tanzania (SATCC)	Uganda (AASHTO)	Rwanda
Passenger car (P)		5.0		5.8	
Single unit truck (SU)		11.0		9.1	
BUS				12.1	
Semi-trailer combination		15.0		16.7	
WB-15					
Interstate semi-trailer			21.0	21.0	

Table 6: Comparison of minimum turning radii for the design vehicle outer side

Vehicle	Minimum design turning radius (m)		
	Kenya	Tanzania (SATCC)	Uganda (AASHTO)
Passenger car	6.0	6.8	7.3
Single unit truck	12.5	10.0	12.8
BUS		11.5	12.8
Semi-trailer combination	12.5	11.0	13.7
Interstate semi-trailer		-	13.7

As suggested by AASHTO design guidelines, for intersection of local streets and park roads, a single unit truck may be selected as a design vehicle. For intersections of rural highways and urban streets that serve buses with relatively few large trucks, a city transit bus may be used. For intersections of highways and low volume rural roads or local roads with less than 400 ADT, either a large school bus 12 m long (84 passengers) or 65 passengers conventional bus 11 m long may be used. Also, for intersections of freeway ramp terminals and arterial crossroads, and at intersections of rural highways and industrialised streets that carry high volumes of traffic, the minimum size of the design vehicle should be WB-20. It should be noted, however, that the minimum turning radii for single unit truck, city transit bus, large school bus, conventional school bus, and WB-20 (interstate semitrailer) are 12.8 m, 12.8 m, 12.0 m, 11.9 m, and 13.7 m, respectively. Of these vehicles, the longest one is WB-20 which is 21.0 m long.

It can be noted from Tables 5 and 6 that Uganda follows AASHTO vehicles dimensions and SATCC and design manuals for other EAC Partner States acknowledge AASHTO standards as well. Further, it can be noted that wheelbases and minimum turning radii proposed by the vehicles dimensions study (Annex B10) and found by a study of axle loads; turning circles; and lane widths for freight vehicles in Sub-Saharan Africa by CISR (2010) worthy further consideration as follows.

Comparison of the dimensions for wheelbases and minimum design turning radii given in Tables 5 and 6 to the wheelbases and minimum turning radii of the above vehicles (Figures 2 to 4) reveals that geometric design for trucks and buses require much more generous designs than for passenger vehicles. Trucks and buses are wider and have longer wheelbases and greater minimum turning radii. These are the principal characteristic dimensions affecting intersections and horizontal roadway design. The longer single-unit trucks and buses require greater minimum turning radii than most vehicle combinations, but because of their greater offtracking, the longer vehicle combinations also require greater widths of turning paths. This characteristic is also quite evident in AASHTO design vehicle dimensions. Furthermore, comparison of the width, height, wheelbase and minimum turning radius dimensions for the above vehicles (Figures 2 to 4) to the respective dimensions for the design vehicles documented in the existing design manuals for EAC Partner States shows that the 22 m Interlink vehicles, which has recently been permitted to traverse EAC road network, will be accommodated by the existing intersections on the EAC road network. Therefore the dimensions of the above vehicles are still within the range of design vehicle dimensions that recommended by the existing design manuals for the EAC Partner States.

- Articulated vehicle with 10 m wheelbase semi-trailer, long truck: the above studies found that the minimum turning radius for 90 and 180 degrees turn was 12.648 m

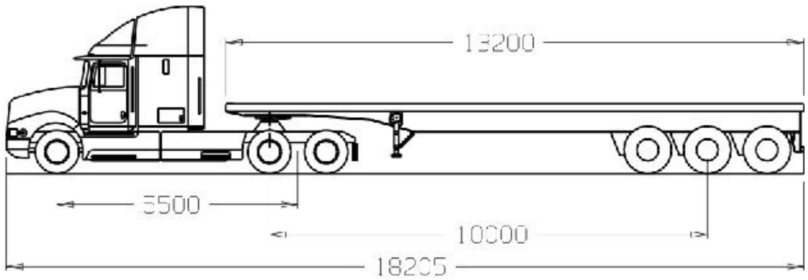


Figure 2: Articulated vehicle with 10 m wheelbase semi-trailer, long truck (Source: CISR, 2010)

- 22 m rigid truck and drawbar trailer combination: the above studies found that the minimum turning radius for 90 and 180 degrees turn was 12.0 m

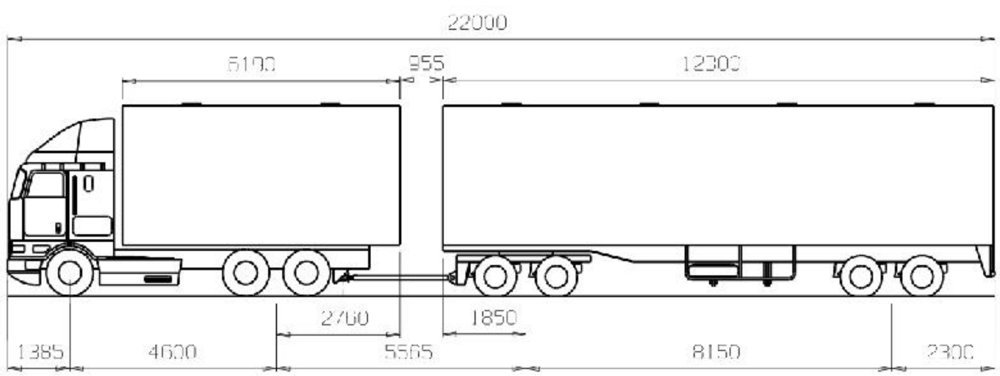


Figure 3: A 22 m rigid truck and drawbar trailer combination (Source: CISR, 2010)

- 22 m interlink (with a short truck tractor): the above studies found that the minimum turning radius for 90 and 180 degrees turn was 12.0 m.

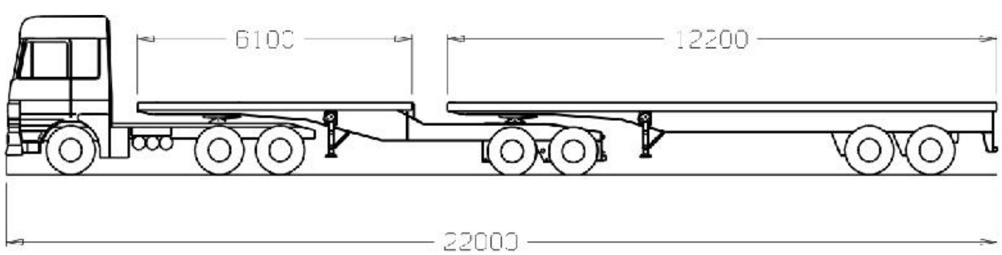


Figure 4: A 22 m interlink (with a short truck tractor) (Source: CISR, 2010)

Recommendation
EAC member states should adopt uniform design vehicle dimensions shown in Table 7.

Table 7: Proposed design vehicle dimensions

Design vehicle type	Symbol	Dimension (m)				
		Length	width	Height	Minimum design turning radius	Wheelbase
Passenger cars						
Passenger car	DV1	5.8	2.1	1.3	7.31	3.35
Buses						
Single unit bus	DV2	12.1	2.6	4.1	12.8	7.6
Articulated bus	DV3	18.3	2.6	3.4	12.1	6.7 + 5.9
Trucks						
Single unit truck	DV4	9.1	2.6	4.6	12.8	6.1
Semi-trailer	DV5	18.2	2.6	4.6	12.65	5.5+10.0
Rigid truck and drawbar trailer combination	DV6	22.0	2.6	4.6	12.0	4.6+8.2
Interlink (with a short truck tractor)	DV7	22.0	2.6	4.6	12.0	4.1+6.52*+7.94
Semi-trailer with long trailer	DV8	21.0	2.6	4.6	13.7	6.1 & 12.8

**distance between SU rear wheels and trailer front wheels*

2.3.3.2 Terrain

The next important factor that affects roadway geometric design is the terrain. It is easier to construct roads with required standards for a plain terrain. Generally, construction costs will be greater as the terrain becomes more difficult and higher standards will become less justifiable or achievable in such situations than for roads in either flat or rolling terrain. Drivers also expect lower standards in such conditions and therefore adjust their driving accordingly, so minimising crash risk. Design speed will therefore vary with terrain as well.

Each EAC Partner States has adopted and defined different classes of terrain and the associated average ground slope. Kenya has three terrain types, Tanzania and Uganda have four (see Table 6). Interestingly, Tanzania combines hilly and mountainous terrains into one group in specifying design standards while the Uganda design manual uses only three terrain types to define the associated standards and the escarpment terrain is left out. With the exception of hilly and escarpment terrains which are just mentioned but are not tied to any design standards of some of the design guides, other terrain types are widely used in the existing design guides of EAC Partner States as shown in Table 8.

Table 8: Comparison of transverse terrain slope categories

Terrain	Kenya	Tanzania	Uganda
Flat	Around 5%	0% - 10%	Around 5%
Rolling	Around 20%	10% - 25%	5%<slope≤20%
Hilly	na	25% - 60%	na
Mountainous	Up to 70%	Above 60%	20%<slope≤70%
Escarpment	na	na	Slope >70%

On the other hand, the Trans-African Highway Network is classified into four (4) terrain categories of level (L) with cross slope of 0 to 10 per cent, rolling (R) with cross slope of more than 10 to 25 per cent, mountainous (M) with cross slope of more than 25 to 60 per cent, and steep (S) with cross slope of more than 60 per cent.

Thus, EAC road terrain categories need to be defined clearly and harmonised so as to remove ambiguity in classifying transverse slopes and to omit terrain categories which are just mentioned in the design guides but are not tied to any design standards.

Recommendation

We propose EAC member countries to adopt terrain categories of flat with cross slope of 0 to 10 per cent, rolling with cross slope of more than 10 to 25 per cent, mountainous with cross slope of more than 25 to 60 per cent, and steep with cross slope of more than 60 per cent.

2.3.3.3 Driver Performance

Perception-Reaction Time

It takes time to process information. After a person's eyes detect and recognize a given situation, a period of time elapses before muscular reaction occurs. Reaction time is appreciable and differs between persons. It also varies for the same individual, being increased by fatigue, drinking, or other causes. The AASHTO brake reaction time for stopping has been set at 2,5 s to recognize all these factors for conditions that are more complex than simple conditions used in laboratory and road tests. AASHTO also makes provision for a reaction time of 5.7 to 10.0 seconds for more complex-choice situations. These extended times make provision for the case where more than one external circumstance must be evaluated, and the most appropriate response selected and initiated. NCHRP Synthesis 299 (NCHRP, 2001) reported about a study by Lerner (1995) which evaluated the perception reaction time for intersection sight distance, stopping sight distance, and decision sight distance, especially for older drivers. The study found that differences in perception reaction time between age groups were trivial and the current AASHTO perception reaction time values are still adequate. French guide reports the physiological perception-reaction time of 1.3 to 1.5 s and the mechanical delay before the brakes start to act of 0.5s. For calculation purposes, it recommends the perception-reaction time of 2 s irrespective of the speed, even though it is accepted that under conditions of sustained attention (speeds in excess of 100 km/h or prolonged high speed traffic) it may be reduced to 1.8s. However, the guide observes that a 0.2 s change has little effect on stopping distance (e.g. 5 m at 90 km/h) and studies from other countries tend to estimate this value at between 2 and 2.5 s.

Guidance on driver perception reaction time does not vary among EAC Partner States as shown in Table 9. Since this is an area of commonality it lends itself to harmonization.

Table 9: Driver characteristics

Parameter	**Design guide**							
	AASHTO	**SATCC**	**TRL ORN 6**	**Burundi (AASHTO)**	**Kenya**	**Tanzania**	**Uganda**	**Rwanda (French guide)**
Perception reaction time (secs)	2.5	2.5	2.00	2.5	Na	2.5	2.5	2.5

Recommendation

We EAC Partner States should continue to use driver perception reaction time of 2.5 seconds.

Height of Driver's Eye

AASHTO recommends driver's eye height of 1.08 m, for large trucks the driver eye height ranges from 1.8 m to 2.4 m and the recommended value for truck driver eye height for design is 2.33 m above the roadway surface. AASHTO observes that the truck driver is able to see the vertical features of the obstruction from substantially further because of the higher driver eye height. However, separate stopping sight distances for trucks and passenger cars are not generally used in highway design.

Driver eye height values vary between the EAC Partner States, but with reasonable narrow ranges (Table 10). French design guide, which is followed by Burundi and Rwanda, recommends that truck stopping sight distances should be checked at potentially hazardous locations. In the measurement of stopping sight distance, the guide suggests the driver's eye height as being at 1.00 m and at a distance of 2.0 m from the edge of the carriageway. The design guide for Kenya proposed an eye height of 1.10 m when determining sight distance on horizontal curves. The draft design manual for Tanzania reported eye height of 1.15 m while, for Uganda eye height value is 1.07 m. Further, in the Southern Africa region research by Pretorius (1976) and Brafman Bahar (1983) indicated that 95% of passenger car drivers have an eye height of 1,05 m or more, and 95% of bus or truck drivers an eye height of 1,8 m or more. In the absence of data on the same and eye height within the EAC region, it is reasonable to borrow these values with some minor adjustment based upon the experience of using values other than the AASHTO values in the region.

Table 10: Driver eye height

Parameter	**Design guide**							
	AASHTO	**SATCC**	**TRL ORN 6**	**Burundi (French guide)**	**Kenya**	**Tanzania**	**Uganda**	**Rwanda (French guide)**
Driver eye height (m)	1.08	1.05	1.05	1.00	1.10	1.15	1.07	1.00

Recommendation

We propose that EAC Partner States should adopt a driver eye height of 1.05 m for sight.

Object Height

Object heights controls are used to compute sight distances and are reviewed in terms of the following sight distances:

- Stopping sight distances needed for stopping, which are applicable on all highways.
- Passing sight distance, this is applicable only on two-lane roads and is required for substantial portions of the length of these roads.
- Decision sight distance needed for decisions at complex locations where, for example, a driver must be able to see and respond to road markings.

- Headlight sight distance, typically applied to sag vertical curves, and

For calculation and measurement of stopping sight distance, AASHTO (2004) recommends height of object to be 0.6 m, equivalent to the taillight height of a passenger car, above the road surface. The value is considered to be representative of an object that involves risk to drivers and can be recognised by a driver in time before reaching it. Use of longer stopping sight distances increases the margin of safety for all drivers and, in particular, for those who operate at or near the design speed. AASHTO observed that object heights of less than 0.6 m for stopping sight distance calculations would result in longer crest vertical curves without documented safety benefits and it could substantially increase construction costs because of the additional excavation needed to provide the longer curves. Additionally, in calculating braking distance of a vehicle on a level roadway at the design speed of the roadway AASHTO observed that studies documented in the literature show that most drivers decelerate at rate greater than 4.5 ms/s2 when confronted with the need to stop for an unexpected object in the roadway. It found that approximately 90% of all drivers decelerate at greater than 3.4 m/s2, a decoration rate which is within the driver's capability to stay within his or her lane and maintain steering control during braking manoeuvre on wet surfaces. Therefore 3.4 m/s2is recommended as thee deceleration threshold for determining stopping sight distance. Calculated stopping sight distances must be rounded up for design purposes.

French design standard, which is followed by Burundi and Rwanda, recommends that the designer should adopt an object height based on the probability of a particular object occurring on the roadway. For stopping sight distance, a conservative tail light height of 0.60 m is recommended. If fallen trees or rocks are a real risk, an object height of 0.15 m is recommended. In this context, French design guide observes that research has established that the probability of a collision involving an object of a height of 0.15 m or less is infinitesimally small. Table 11 provides a summary of values suggested by the French design guide.

Table 11: Object Height Design Domain

Object Height (m)	Applicability
1.30	Passing sight distance for top of car
	Intersection sight distance
0.60	Vehicle tail or brake light
0.15	Risk of fallen trees or rocks
	Risk of log or construction debris fallen from truck
	Risk of fallen person
0.00	Risk of road washouts
	Pavement markings in critical condition

Source: French design guide

On a bend, French guide suggests that the increase in stopping distance must be taken into account. This increase occurs because braking must be more energetic on a bend, and it is therefore accepted to increase the braking distance by 25% on bends with a radius of less than 5 times design speed.

SATCC guide recommended the use of an object height of 0.15 m in computing and measuring stopping sight distance. The guide noted that measuring sight distance to the road surface would substantially increase the length of the vertical curve and hence the earthworks required. Object height design values vary between the EAC Partner States, but with reasonable narrow ranges as shown in Table 12. The Kenyan manual proposed an object height of 0.10 m when determining sight distance on horizontal curves. The design manuals for Tanzania and Uganda reported an object height of 0.15 m.

Table 12: Comparison of values used in estimating sight distances

	Stopping sight distance	Decision sight distance	Passing sight distance
Country/standard	**Object height (m)**	**Object height (m)**	**Object height (m)**
Kenya	0.10	-	1.10
Tanzania	0.15	-	-
Uganda	0.15	-	1.30
AASHTO	0.60	0.60	1.08
SATCC	0.15	0.00	1.30

It is also evident that there are many locations where it would be prudent to provide longer sight distances. In such circumstances, decision sight distance provides the greater length that drivers need. If the driver can see what is unfolding far enough ahead, he or she should be able to handle almost any situation. Critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance include:

- Approaches to interchanges and intersections;
- Changes in cross-section such as at toll plazas and lane drops;
- Design speed reductions, and
- Areas of concentrated demand where there is apt to be "visual noise", e.g. where sources of information, such as roadway elements, opposing traffic, traffic control devices, advertising signs and construction zones, compete for attention.

In computing and measuring decision sight distances, AASHTO recommends the use of 1.08 m eye height and 0.6 m object height criteria used for stopping sight distance while the 1.05 m eye height and 0.00 m object height have been adopted by the SATCC guide. Further, SATCC recommends drivers perception reaction time to be 7.5 s which is roughly the mean of values quoted in American practice.

Headlight sight distance is typically used in establishing the rate of change of grade for sag vertical curves. At speeds above 80 km/h, only large, light coloured objects can be perceived at the generally accepted stopping sight distances. A five-fold light increase is necessary for a 15 km/h increase in speed and a 50 per cent reduction in object size. For night driving on highways without lighting, the length of visible roadway is that which is directly illuminated by the headlights of the vehicle. This length is typically shorter than the minimum sight distance. When headlights are operated on low beam, the reduced candlepower at the source and the downward projection angle significantly restrict the visible length of roadway surface. For crest vertical curves, the area beyond the headlight beam point of tangency with the roadway surface is shadowed and receives only indirect illumination. Also, a general limit of 120 to 150 metres sight distance is all that can be safely assumed for visibility of an unilluminated object on a bitumen surfacing. This corresponds to a satisfactory stopping sight distance for 80 to 90 km/h or a decision time of about 5 seconds at 100 km/h. Since the headlight mounting height (typically about 0.6 m) is lower than the driver eye height (1.05 m for design), sight distance is controlled by the height of the vehicle headlights and a one degree upward divergence of the light beam from the longitudinal axis of the vehicle. Any object within the shadow zone must be high enough to extend into the headlight beam to be directly illuminated. French guide suggests that height of object as the height of the most easily perceived rear light of a stationary vehicle on the carriageway (statutory minimum: 0.3 m) but for special cases such as areas subject to rock falls this height may be reduced to 0.15 m. On 2 x 2 lane roads with independent grading of the carriageways on the grounds that the main risk involves the presence of stationary vehicle on the carriageway, the ICTAAL, based on a specific study of vehicles travelling on roads of this type, selected a height of 0.60 m instead of the customary 0.35 m.

Passing sight distance is applicable to two-way, 2-lane single carriageway roads. On a two-lane rural road, the passing manoeuvre is one of the most significant yet complex and important driving tasks. The process is relatively difficult to quantify, primarily because of the many stages involved, the relative speed of vehicles and the lengthy section of road needed to complete the manoeuvre. Road safety, capacity and service levels are all affected by the passing ability of faster vehicles. This ability is influenced by a variety of factors, including traffic volumes, speed differentials, road geometry and human factors. The minimum sight distance required by a vehicle to overtake safely on two-lane single carriageway roads is the distance which will enable the overtaking driver to pass a slower vehicle without causing an oncoming vehicle to slow below the design speed. It should be pointed out that there are a variety of models defined for the overtaking manoeuvre. The distances usually given are those required to enable an overtaking driver to complete or abort a manoeuvre already commenced, with safety. In addition to this distance, the Austroads approach introduces a distance that is needed for the driver to identify a length of road as a potential overtaking zone. This "establishment" distance is considerably longer than the overtaking manoeuvre distance. Passing manoeuvres involving trucks require longer distances and therefore this must take this into account for roads where significant percentages of heavy vehicles are expected in the traffic stream. As mentioned above, the designer should seek opportunities to introduce passing lanes on two lane roads, particularly where the terrain limits sight distance.

AASHTO recommends calculation of passing sight distances on the basis of object height of 1.08 m and object height of 1.33 m which is based a vehicle height that represents 15th percentile of vehicle heights in the car population, less an allowance of 0.25 m that represent a near-maximum value for the portion of the vehicle height that needs to be visible for another driver to recognise a vehicle as such. The object height is also considered adequate for night conditions because headlight beam of an opposing vehicle generally can be seen from a greater distance than a vehicle can be recognised in the daytime (AASHTO, 2004). The choice of an object height equal to 1.08 m makes passing sight distance design reciprocal.

A report on a review and evaluation of research studies (Brewer et al., 2001) concluded that passing and climbing lane installations reduce collision rates by 25 per cent compared to untreated two lane sections. Passing lanes provide safer passing opportunities for drivers who are uncomfortable in using the opposing traffic lane and for those who become frustrated when few passing opportunities exist, owing to terrain or traffic density. Sections with adequate passing sight distance should be provided as frequently as possible. The appropriate frequency is related to operating speed, traffic volumes and composition, terrain and construction cost. As a general rule, if passing sight distance cannot be economically provided at least once every 2 km, passing lanes should be considered. The 2+1 cross section currently in vogue in Europe has some merit. This three-lane cross-section has two lanes in one direction and a single lane in the opposing direction. Brewer et al. noted that, at about two to three kilometre intervals, the second lane is allocated to movement in the opposite direction. A minimum shoulder width is required for this option to function properly.

For passing sight distance, the French guide observed that an object height of 1.30 m will allow the driver to discern the top of an oncoming car. A zero object height is recommended where road washouts are a serious risk. It is also recommended for pavement markings in situations such as at intersections or interchanges, where these provide essential guidance. We therefore recommend object height values shown in Table 13 for the EAC region.

Table 13: Recommended object height values

Object height (m)		
Stopping sight distance	**Decision sight distance**	**Passing sight distance**
0.60	0.60	1.08

2.3.3.4 Pedestrians and cyclists performance

Pedestrians

To effectively plan and design pedestrian facilities, it is necessary to understand the typical pedestrian. AASHTO guide notes that a pedestrian's age is an important factor that may explain behaviour that leads to collisions between motor vehicles and pedestrians. Because pedestrians have a broad range of walking speeds, the speed at which they may cross a street is significant in design. AASHTO observes that the average pedestrian walking speeds range from approximately 0.8 to 1.8 m/s. The Manual on Uniform Traffic Control Devices (MUTCD) uses a normal walking speed of 1.2 m/s. Older people will generally walk at speeds in the lower end of this range. AASHTO guide also suggest that a speed of 0.9 m/s should be used for design of areas where there are many older people. Knoblauch et al. (2000) noted that walking speeds are influenced by environmental, traffic, and pedestrian characteristics. The effects of terrain on walking speeds are unknown, although it can be expected that the elderly would be affected more when walking up or down a grade than the young. Also, pedestrian speed on sidewalks and crosswalks is strongly related to the number of pedestrians in the flow. The relationship between speed, flow, and space occupied (i.e., density) for a representative population group has therefore been examined in literature. On space requirements, NCHRP (2006) reports that for standing area design a simplified body ellipse of 50 cm by 60 cm, with a total area of 0.3 m^2 and a body buffer zone of 0.8 m^2 for walking. The minimum width that best serves two pedestrians walking together or passing each other is 1.8 m.

Walking speeds vary from a 15th percentile speed of 1.2 m/s to an 85th percentile of 1.8 m/s, with an average of 1.4 m/s. The design manual for Tanzania recommends a 15th percentile speed for design purposes. The manual also notes that pedestrians' age is an important factor that may explain behaviour that leads to collisions and therefore it recommends that older pedestrians be accommodated by using simple designs that minimize crossing widths and assume lower walking speeds. Where complex elements such as channelization and separate turning lanes are featured, it is recommended that the designer should assess alternatives that will assist older pedestrians. Pedestrian safety is enhanced by the provision of median refuge islands of sufficient width at wide intersections, and lighting at locations that demand multiple information gathering and processing.

Cyclists

To provide adequately for bicycle traffic, the designer should be familiar with bicycle dimensions, operating characteristics, and needs. These factors determine acceptable turning radii, grades, and sight distance. In many instance, design features of separate bike facilities are controlled by the adjoining roadway and by the design of the highway itself. None of the existing design manuals for EAC Partner States makes recommendation for these design factors. However, guidance can be obtained from the AASHTO guide for development of bicycle facilities. Figure 5 shows different types of bicycles with their over lengths. AASHTO recommends the following critical physical dimensions for bicycle operation (Figure 6), which can accommodate an adult bicyclist standing upright on the pedals.

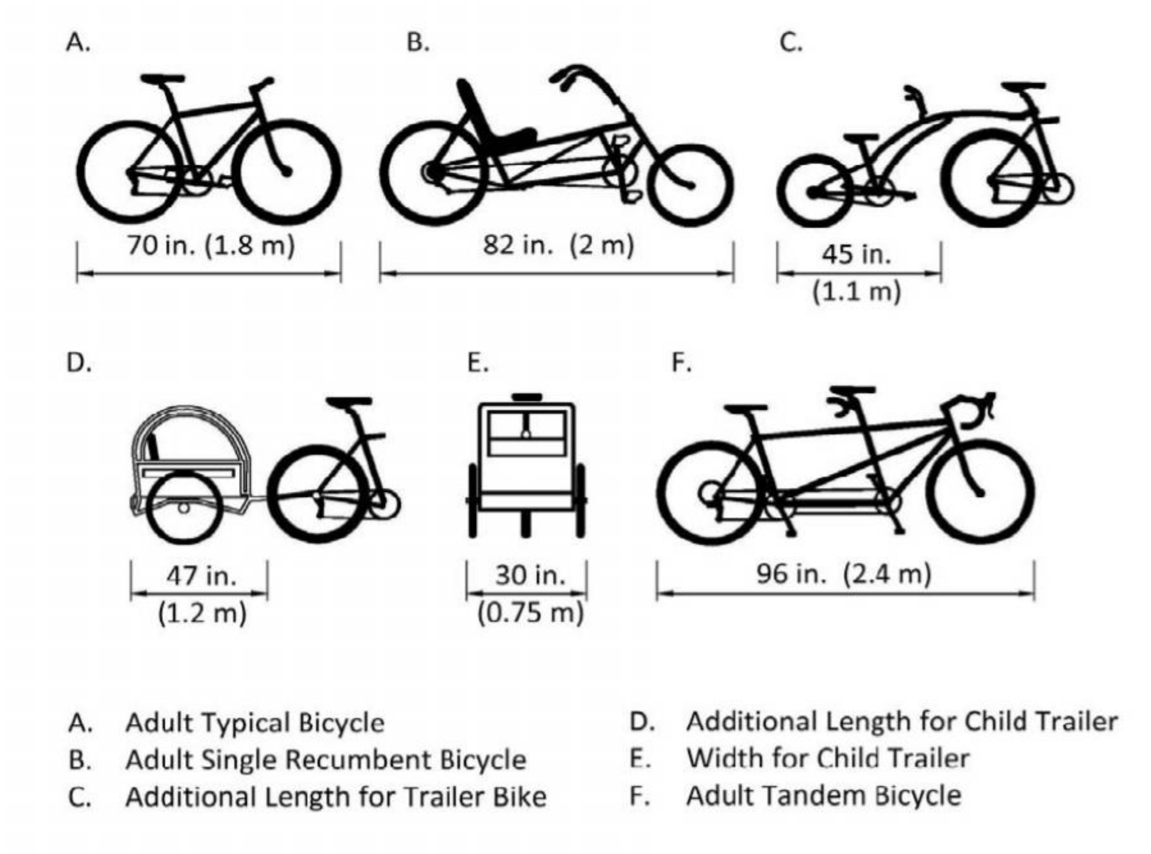


Figure 5: Typical bicycle types and dimensions (Source: AASHTO)

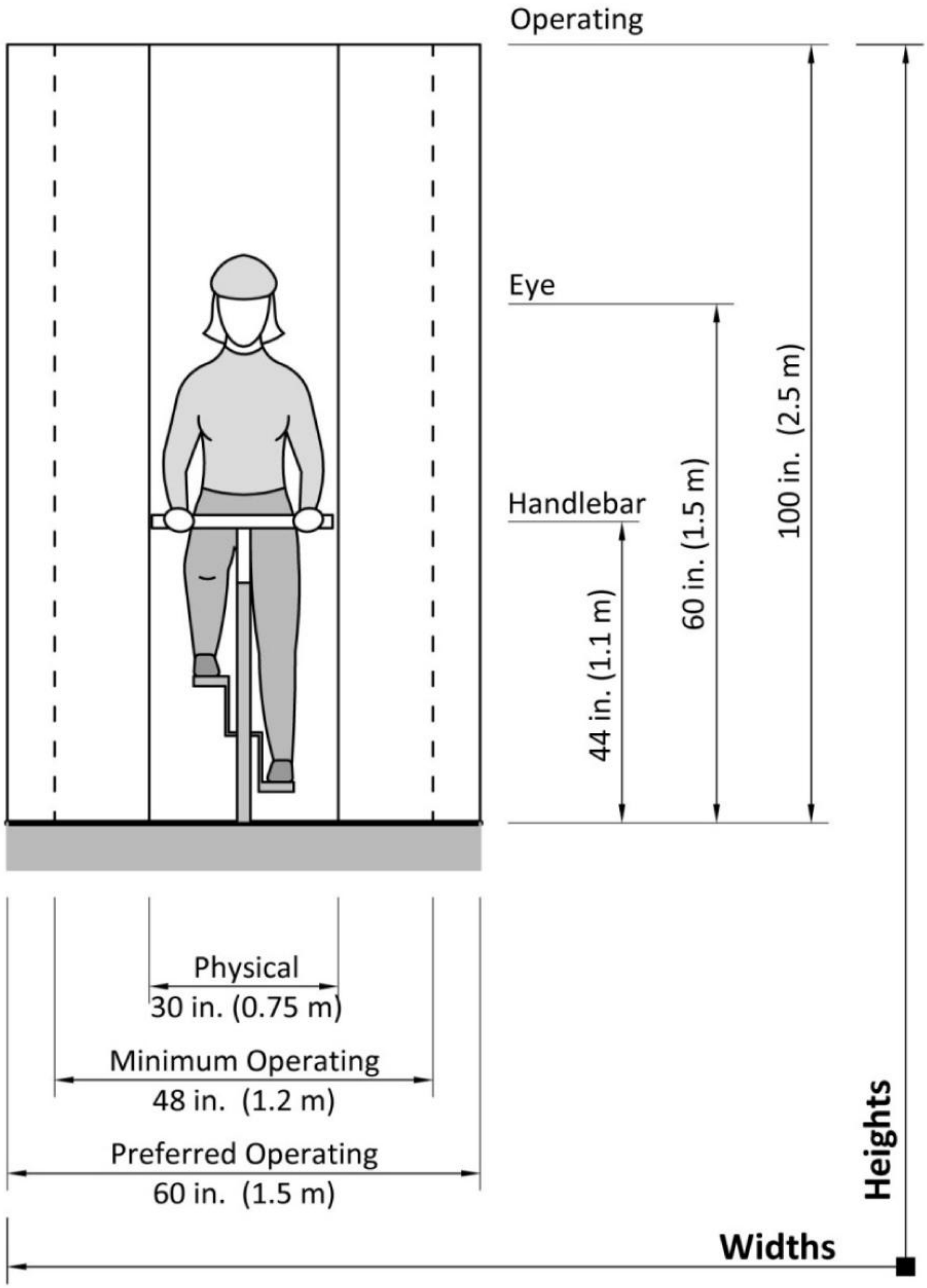


Figure 6: Bicycle operating space (Source: AASHTO)

Furthermore, Table 14 presents key bicycle performance features and values.

Recommendation

We recommend pedestrian speed of 1.2 m/s and adult typical bicycle performance criteria (speed 13-24 km/h, coefficient of friction for braking of 0.32, and minimum height of 1.0 m and width of 1.5 m for a bicycle operation) shown in Figure 7, 8 and Table 13 for the design of NMT facilities.

Table 14: Key bicycle performance criteria

Bicyclist Type	Feature	Value	
		US Customary	Metric
Typical upright adult bicyclist	Speed, paved level terrain	8-15 mph	13-24 km/h
	Speed, downhill	20-30 plus mph	32-50 plus km/h
	Speed, uphill	5-12 mph	8-19 km/h
	Perception reaction time	1.0-2.5 s	1.0-2.5 s
	Acceleration rate	1.5-5.0 ft/s^2	0.5-1.5 m/s^2
	Coefficient of friction for braking, dry level pavement	0.32	0.32
	Coefficient of friction for braking, wet level pavement	0.16	0.16
	Deceleration rate (dry level pavement)	8.0-10.0 ft/s^2	2.4-3.0 m/s^2
	Deceleration rate for wet conditions (50-80% reduction in efficiency)	2.0-5.0 ft/s^2	0.6-1.5 m/s^2
Recumbent bicyclist	Speed, level terrain	11-18 mph	18-29 km/h
	Acceleration rate	3.0-6.0 ft/s^2	1.0-1.8 m/s^2
	Deceleration rate	10.0-13.0 ft/s^2	3.0-4.0 m/s^2

(Source: AASHTO)

2.3.3.5 Traffic Characteristics

Factual information on expected traffic volumes is an essential input to design. This indicates the need for improvements and directly affects the geometric features and design. Traffic flows vary both seasonally and during the day. The designer should be familiar with the extent of these fluctuations to enable him or her to assess the flow patterns. The directional distribution of the traffic and the manner in which its composition varies are also important parameters. A thorough understanding of the manner in which all of these behave is a basic requirement of any realistic design.

Traffic composition

Vehicles of different sizes and weights have different operating characteristics. Trucks have a higher mass/power ratio and occupy more roadway space than passenger cars. Consequently, they constitute greater impedance to traffic flow than passenger vehicles, with the overall effect that one truck is equivalent to several passenger cars. AASHTO design guide noted that the number of equivalent passenger cars equalling the effect of one truck is dependent on the roadway gradient and, for two-lane highways, on the available passing sight distance. Thus, the larger the proportion of trucks in a traffic stream, the greater the equivalent traffic demand and the greater the highway capacity needed. For design purposes, the percentage of truck traffic during the peak hours has to be estimated. Data on the composition of traffic should be determined from traffic studies. Truck traffic is normally expressed as a percentage of total traffic during the design hour in the case of a two-lane road; and as a percentage of total traffic in the predominant direction of travel in the case of a multi-lane road. Additionally, it is not practical to design for a heterogeneous traffic stream and, for this reason; trucks and other types of vehicles are converted to equivalent Passenger Car Units (PCUs). The number of PCU's associated with a single vehicle type is a measure of the impedance that it offers to the passenger cars in the traffic stream.

A comparative assessment of the vehicle equivalent factors given in the Kenya, Tanzania and Uganda geometric design manuals revealed that these countries have recommended use of same vehicle equivalent factors shown in Table 15. However, Tanzania design manual specified a factor of 3.0 as the vehicle equivalent factor for Medium Goods Vehicles in rolling terrain, which we suggest should be changed to 5.0 as shown in Table 15.

Table 15: Passenger Car Equivalent Factors

Vehicle type	Terrain		
	Level	Rolling	Mountainous
	pcu		
Passenger cars	1.0	1.0	1.5
Light goods vehicles	1.0	1.5	3.0
Medium goods vehicles	2.5	5.0	10.0
Heavy goods vehicles	3.5	8.0	20.0
Buses	2.0	4.0	6.0
Motor cycles, scooters	1.0	1.0	1.5
Pedal cycles	0.5	0.5	NA

On the other hand, Lee et al. (2010) noted that there are currently a wide range of PCUs for motorcycles found in the literature (Table 16). Without a careful contextual qualifications on the applicability of each value, such a wide range of PCUs causes confusion. On the other hand, Innovative Transport Solutions (2009) report from IRC106:1990 PCU values for motorcycle and bicycle for India in plain areas as 0.75 and 0.5, respectively, when their percent composition in the traffic stream is 10% and above and 0.5 and 0.4, respectively when their percent composition in the traffic stream is less than 5%.

Table 16: PCU values of motorcycles (source: Lee et al., 2010)

Source	PCU	Note
The 'best available value in the literature'	0.5	• Generally derived from road occupancy, or as a plausible value in the absence of hard data
Quoted in Wigan (2000)	0.2	• Quoted from the Indonesian Highway Capacity Manual, for congested signalised intersections
	0.25	• Quoted from the Indonesian Highway Capacity Manual, for free running urban road sections
Chandra and Kumar (2003)	0.25 ~ 0.31	• Measured in road sections
Highway Department of Bangladesh (2006)	0.75	• Used in Bangladesh
Branston and Zuylen (1978)	-0.08	• Measured from saturation flows at intersections by using the headway ratio method
	0.04	• Measured from saturation flows at intersections by using the flow rate method
Quoted in Powell (2000)	0	• During the first 6 sec of a green period at a signalised intersection, quoted from May and Montgomery (1986).
	0.53 ~ 0.65	• After the 6th sec of a green period at a signalised intersection, quoted from May and Montgomery (1986).
Turner and Harahap (1993)	0.37	• Measured from saturation flows at intersections by using the headway ratio method
Hossain (2001)	0.15 ~ 0.30	• Measured from saturation flows at intersections by using the flow rate method
Quoted in Nguyen and Montgomery (2006)	0.25	• Used in Vietnam. Nguyen and Montgomery (2006) used it for calculating saturation flow rates in signalised intersections.
Kimber et al. (1982)	0.4	• Used in most studies since early 1980s in the UK
Lan and Chang (2005)	0.17 ~ 0.49	• Measured by using Cellular Automata Modelling under different speeds, proportions of motorcycles and lane widths
Road Research Laboratory (1965)	1.0	• For rural roads; used in most studies in 1960s and 1970s in the UK
	0.75	• For urban streets and roundabouts; used in most studies in 1960s and 1970s in the UK
	0.33	• For signalised intersections; used in most studies in 1960s and 1970s in the UK

Recommendation

EAC member states should adopt vehicle equivalent factors shown in Table 17.

Table 17: Recommended Passenger Car Equivalent Factors

Vehicle type	Terrain		
	Level	Rolling	Mountainous
	pcu		
Passenger cars	1.0	1.0	1.5
Light goods vehicles	1.0	1.5	3.0
Medium goods vehicles	2.5	5.0	10.0
Heavy goods vehicles	3.5	8.0	20.0
Buses	2.0	4.0	6.0
Motor cycles, scooters	0.5	1.0	1.5
Pedal cycles	0.5	0.5	NA

Projection of Future Demands

Geometric design of new roadway or improvements of existing roadways should consider future traffic volumes expected to use the facility. A roadway should be designed to accommodate the future volume that is likely to occur within the design life of the facility. It is difficult to define the life of a highway because major segments may have different lengths of physical life. Each segment is subject to variations in estimated life expectancy for reasons not readily subject to analysis, such as obsolescence or unexpected radical changes in land use, with the resulting changes in traffic volumes, patterns, and demands. As a general practice, the design volume should be a value that can be estimated with reasonable accuracy.

AASHTO (2011) observes that right-of-way and grading may be considered to have a physical life expectancy of 100 years; minor drainage structures and base courses, 50 years; bridges, 25 to 100 years; resurfacing, 10 years; and pavement structure, 20 to 30 years, assuming adequate maintenance and no allowance for obsolescence. Bridge life may vary depending on the cumulative frequency of heavy loads. Pavement life can vary widely, depending largely on initial expenditures and the repetition of heavy axle loads. AASHTO notes that the assumption of no allowance for functional obsolescence is open to serious debate. The principal causes of obsolescence are increases in the number of intersections and driveways and increases in traffic demand beyond the design capacity. On non-freeway highways, obsolescence due to addition of intersections and driveways is much more difficult to forestall; this occurs particularly in urban and suburban areas, but may occur in rural areas as well. It is a moot question whether the design capacity of a highway should be based on its life expectancy. The decision is greatly influenced by economics. For example, a highway might be designed for traffic volumes 50 years hence with the expectation that the pavement structure would be restored in 20 to 25 years. However, if the added cost of a 50-year design over a design with a 25-year life expectancy is appreciable, it may be imprudent to make a further investment providing capacity that will not be needed for at least 25 years. The construction cost savings could be used to construct another currently needed highway project. Furthermore, the cost of increased maintenance for the larger highway would be avoided for at least 25 years. Also, most highways are capable of handling higher traffic volumes than their design volume indicates, but this may cause more inconvenience, such as a reduction in speed and less maneuverability.

To be practical, AASHTO suggests that the maximum design period should be in the range of 15 to 24 years and a period of 20 years is widely used as a basis for design of new facilities and 5 to 10 years for reconstruction or rehabilitation projects because of uncertainties of predicting traffic and funding constraints. Traffic cannot usually be forecast accurately beyond this period on a specific roadway since there may be changes in the regional economy, population and land development along the roadway, which cannot be predicted with any degree of assurance. It should be noted that the challenges of uncertainty of traffic volume forecast and funding constraints are much more pronounced in developing countries like ours in East Africa.

The draft design manual for Tanzania does not specify a specific design period, however, the pavement and materials design manual suggests 20 years. Uganda design manual recommends use of 10 – 20 years as design year.

Recommendation

We propose that projection of future traffic demand should be based on a period of 20 years for new roadway design and 10 years for reconstruction and rehabilitation in EAC member states.

Design hourly volume (DHV)

The design hourly volume is the projected hourly volume that is used for design. This volume is usually taken as a percentage of the expected ADT on the highway. The 30th-highest hourly volume is usually selected as the DHV. Experience has shown that the 30th-highest hourly volume as a percentage of ADT varies only slightly from year to year, even when significant changes of ADT occur. It has also been shown that, excluding rural highways with unusually high or low fluctuation in traffic volume, the 30-highest hourly volume for rural highways is usually between 12 and 18 percent of the ADT, with the average being 15 percent. The 30th-highest hourly volume may also be used as the DHV for urban highways. It is usually determined by applying between 8 to 12 percent of the ADT. Other relationships may be used for highways with seasonal fluctuation in traffic flow much different from that on rural roads. One alternative is to use the average of the highest afternoon peak hour volume for each week in the year as the DHV (Garber and Hoel, 2010).

The 30th-highest hourly volume should not be indiscriminately used as the DHV, particularly on highways with unusually or high seasonal fluctuation in the traffic flow. Although the percent of average annual daily traffic (AADT) represented by the 30th-highest hourly volume on such highways may not be significantly different from those on most rural roads, this criterion may not be applicable, since the seasonal fluctuation results in a high percentage of high-volume hours and a low percentage of low-volume hours. For example, economic consideration may not permit the design to be carried out for the 30th-highest design volume, but at the same time, the design should not be such that severe congestion occurs during peak hours. A compromise is to select a DHV that will result in traffic operating at a somewhat slightly lower level of service than that which normally exists on rural roads with normal fluctuations. For this type of rural road, it is desirable to select 50 percent of the volume that occur for only a few peak hour during the design year as the DHV, even though this may not be equal to the 30th-highest hourly volume. This may result in some congestion during the peak hour, but the capacity of the highway normally will not be exceeded (Garber and Hoel, 2010).

Geometric design manual for Uganda also estimates DHV as = AADT x K or ADT x K where K is estimated from the ratio of the 30th HV to the AADT from a similar site. The 30th HV is the 30th highest hourly volume during the year. The manual indicates that the 30th HV which is expressed as a fraction of ADT can vary as indicated in Table 18. Other design manuals in the EAC region do not include the computation of DHV or show factors that are used to compute the DHV.

Table 18: Traffic Condition and 30th HV as a fraction of ADT

Traffic Condition	30th HV as a fraction of ADT
Rural Arterial (average value)	0.15
Rural Arterial (maximum value)	0.25
Heavily trafficked road under congested urban	0.08 – 0.12

conditions	
Normal urban conditions	0.10 – 0.15
Road catering for recreational or Other traffic of seasonal nature	0.20 – 0.30

Source: Uganda design manual (2005)

Higher percentages in the table refer to roads with relatively high concentration of traffic during rush-hours or large seasonal changes.

Recommendation

We recommend that field data should be used to estimate K factors for estimation of DHV and in the absence of such factors the 30th-highest DHV can be estimated by applying 0.15 and 0.10 to ADT for rural highways and urban roads, respectively.

2.3.3.6 Design Speed

The most important factor in geometric design is the design speed. This was previously defined as the highest continuous speed at which individual vehicles can travel with safety on the road when weather conditions are favourable, traffic volumes are low and the design features of the road are the governing condition for safety. The current definition is simply states that the design speed is the speed selected as the basis for establishing appropriate geometric elements for a section of road. AASHTO (2004) guide on geometric design of highway and streets defines design speed as a selected speed used to determine the various geometric design features of the roadway. The assumed design speed is usually expected to be a logical one with respect to the terrain, anticipated operating speed, the adjacent land use, and the functional classification of highway. Except for local streets where speed controls are frequently included intentionally, it is advised to use as high a design speed as practical to attain a desired degree of safety, mobility, and efficient within the constraints of environmentally quality, economics, aesthetics, and social or political impacts.

Above-minimum design values should be used, where practical. Some design elements, such as curvature, superelevation, and sight distance, are directly related to, and vary appreciably with, design speed. Other design features, such as widths of lanes and shoulders and clearances to walls and rails, are not directly related to design speed, but they do affect vehicle speeds. It is desirable that the running speed of a large proportion of drivers be lower than the design speed. However, curves with low design speeds (relative to driver expectation) are frequently overdriven and tend to have poor safety records. Therefore it is recommended that the design speed used for horizontal curve design be a conservative reflection of the expected speed on the constructed facility. Where a difficult condition is obvious, drivers are more apt to accept a lower speed than where there is no apparent reason for it. A highway of higher functional classification may justify a higher design speed than a less important facility in similar topography, particularly where the savings in vehicle operation and other operating costs are sufficient to offset the increased costs of right-of-way and construction.

Drivers, on the whole, are concerned with minimising their travel times, and speed is one of the most important factors governing the selection of alternate routes to gain time savings. The attractiveness of a specific road or route is generally judged by its convenience in travel time, which is directly related to travel speed. Various factors influence the speed of vehicles on a particular road. These include trip length, vehicle operating capabilities, the physical characteristics of the road and its surroundings, weather, presence of other vehicles, and speed limitations (posted speed limits).

Roads should, therefore, be designed to operate at a speed that satisfies most, but not necessarily all, drivers. Various studies have shown that the 85th percentile speed generally exceeds the posted speed limit by a margin of at least 10 km/hr when weather and traffic conditions are favourable. For this reason, design speed is typically equated to the 85th percentile speed. The relationship between road design and speed is interactive. While the designer shapes the elements of the road by the anticipated speed at which they will be used, taking into account the inherent economic trade-offs between construction and environmental costs of alternative alignments (vertical and horizontal) to match desired travel speed, the speed at which they will be used depends to a large extent on the chosen design features.

AASHTO design guide suggests that an overall range in design speeds of 20 to 120 km/h be used in geometric design and that it is desirable to select design speeds in increments of 10 km/h. While smaller increments would make little difference in the dimensions of design elements between one design speed and the next higher design speed, larger increments of 20 to 30 km/h would results in too large a difference in the dimensions of design elements between any two consecutive design speeds.

A low design speed, should not be assumed where the topography is such that drivers are likely to travel at high speeds. When carefully selected, these factors should result in a design speed which is acceptable to all but a very few drivers. Above minimum design values should be used where feasible though consistency is essential. When a substantial length of road is being designed, it is desirable to adopt a constant design speed to maintain consistency. Changes in terrain and other physical controls may, however, dictate a change in design speed on certain sections. Each section, however, should be relatively long, compatible with the general terrain or development through which the road passes. The justification for introducing a reduced design speed should be obvious to the driver. A case in point is where a road leaves relatively level terrain and starts traversing hilly or mountainous terrain. Moreover, the introduction of a lower or higher design speed should not be effected abruptly but over sufficient distance to encourage drivers to change speed gradually.

Where design speeds exceed 90 km/h the variation between successive speeds should be limited to 10 km/h and, below 80 km/h, this variation should be limited to 20 km/h. Where it is necessary to change the design speed, the new design speed should apply to an extended section of road. Even if properly signposted, isolated design speed variations are hazardous as they do not match driver expectations and it is always possible that the signpost may be obscured, illegible, removed or even simply not perceived by the driver. Isolated design speed changes are, therefore, to be avoided. The need for a multilane cross-section suggests that traffic volumes are high. A design speed of at least 120 km/h should be used if the topography permits. Major roads, even if two-lane two way roads, should also be designed to this speed if possible. Rolling terrain may, however, necessitate a reduction to 100 km/h in the design speed and, in the case of mountainous terrain, it may even be necessary to reduce the design speed to 60 km/h. Secondary and tertiary roads may have lower design speeds than those advocated for the primary road network. However, where traffic is likely to move at relatively high speeds on these roads, higher design speeds should be selected.

On the other hand, there is still debate as to whether speeds greater than 120 km/h should be used for design purposes on freeways. Higher design speeds not only safeguard against early obsolescence of the highway, but also provide an increased margin of safety for those driving at high speeds. That there is some validity in this statement is reflected by the fact that the design speed of high-type roads is now at least 120 km/h as compared with 56 km/h in 1927, a change brought about by the continuing increase in vehicle performance. The choice of a design speed for a dual carriageway is much less influenced by construction cost than that for other rural roads. In practice, lower design speeds are often accepted on single carriageway roads in order to keep construction costs within certain limits. There is danger in this philosophy since, although drivers will obviously accept lower speeds in what are clearly difficult locations, repeated studies have shown that they do not adjust their speeds to the importance of the facility. Instead they endeavour to operate at speeds consistent with the traffic on the facility and its physical limitations. Ideally, then, design speed should be chosen to reflect the 85th percentile desired speed that is likely to materialize. This is often achievable for roads for which the primary function is mobility and where severe physical constraints do not exist. Limited studies in South Africa have shown that the 85th percentile speed exceeds 120 km/h for unhindered vehicles on a four-lane divided roadway. It is therefore suggested that use of a design speed of 130 km/h could satisfy driver demands in most areas.

However, the French design guide observes that the currently used principle is to restrict the V85 speed to the speed limit. Roads are no longer designed to provide operating speeds that exceed speed limits. Therefore, except in the case where the actual V85 speed proves to be below the speed limit, it is the latter which should be applied. The only exception involves visibility at intersections where in all cases the V85 speed is used on safety grounds.

Tables 19 to 21 present a summary of design speeds specified in the design manuals used in EAC Partner States. It can be noted that the minimum design speed in Tanzania for flat terrain for Class A roads is higher compared to Kenya and Uganda. Apart from this observation, it can be observed that the design speeds for various classes of roads in various terrain types varies among the Partner States. This is not surprising as the there is some variances in the transverse slope values even within the same terrain classification. Additionally, it can be noted from the tables that the maximum design speed in flat terrain is 120 km/hr and 100 km/hr and 80 km/hr in rolling and mountainous terrains, respectively.

SATCC guide noted that the need for a multilane cross-section presumes that traffic volumes are high. To minimise road user costs, a design speed of 120 km/hr was recommended for flat terrain and this applies as well for the case of trunk roads which may be two-lane two-way. The guide also recommends a design speed of 100 km/hr and 80 km/hr in rolling and mountainous terrains, respectively. *We recommend use of design speed values shown in Table 22 for road classes that fall under the EAC road networks in the Partner States.*

Table 19: Comparison of design speed for various road class A and terrain

	Design speed		
	Kenya	Tanzania	Uganda
Road design class		1, 2 & 3	III, II, Ib & Ia
Flat terrain	100 – 120	110 – 120	80 – 120
Rolling terrain	70 – 100	80 – 90	70 – 100
Hilly terrain	na	-	na
Mountainous terrain	50 - 70	70	50 - 80
Escarpment terrain	na	na	-

Table 20: Comparison of design speed for various road class B and terrain

	Design speed		
	Kenya	Tanzania	Uganda
Road design class		1, 2, 3, 4 & 5	III, II, Ib & Ia
Flat terrain	100 – 120	80 – 120	80 – 110 (90*)
Rolling terrain	70 – 100	70 – 90	70 – 100 (80*)
Hilly terrain	na	-	na
Mountainous terrain	50 - 70	50 - 70	50 – 80 (70*)
Escarpment terrain	na	na	-

*for A Gravel Design Class

Table 21: Comparison of design speed for various road class C and terrain

	Design speed		
	Kenya	Tanzania	Uganda
Road design class		3, 4 & 5	III, II, Ib & Ia
Flat terrain	90 – 100	80 – 110	80 – 90
Rolling terrain	60 – 90	70 – 80	70 (80*)
Hilly terrain	na	-	na
Mountainous terrain	40 - 60	50 - 70	50 - 60 (70*)
Escarpment terrain	na	na	-

*for A Gravel Design Class

Table 22: Recommended Design Speeds for the EAC Region

Road class		Design Speed (km/h)			
		Flat terrain	Rolling terrain	Mountainous	Steep
Mobility roads	Class 1	120	100	60	60
	Class 2	110	80	50	50
	Class 3	100	80	50	40
Access roads	Class 4	80	60	40	40
	Class 5	60	40	30	30

2.3.3.7 Levels of Service

Levels of Service express the effectiveness of the road in terms of operating conditions. It is a qualitative measure of the effect of traffic flow factors, such as speed and travel time, interruptions, freedom of manoeuvre, driver comfort and convenience, and indirectly, safety and operation costs. The choice of level of service shall generally be based on economic considerations. The Highway Capacity Manual (HCM) defines six levels of service. The specific definitions of level of service differ by facility type. The levels of service vary from level A which is the free flow condition, where drivers can maintain their desired speed (low volume and high speed); to level of service F which is the forced flow condition (most congested) where the traffic density is the maximum with drivers subjected to frequent stop-go and queues. Volumes here vary from 0 to capacity, but usually are approaching zero. On the other hand, level of service E is where the traffic is approaching saturation with drivers travelling at low speed due to high volume of traffic. The traffic volume at level of service E is the capacity of the facility. The maximum volume that can be carried at any selected level of service is referred to as the service volume for that level. The traffic flow rates that can be served at each level of service are termed as service flow rate. Once a level of service has been identified as applicable for design, the accompanying service flow rate logically become the design service flow rate, implying that if the traffic flow rate using the facility exceeds that value, operating conditions will fall below the level of service for which the facility was designed.

Once a level of service has been selected, it is desirable that all elements of the roadway are consistently designed to this level. Design service flow rate is the maximum hourly flow rate of traffic that a certain road of designed dimensions would be able to serve without the degree of congestion falling below a prescribed level of service. Design service flow rate is the maximum hourly flow rate of traffic that a project road of designed dimensions would be able to serve without the degree of congestion falling below a pre-selected level of service. The objective in road design is to create a facility with dimensional values and alignment characteristics such that the resulting design service flow rate (design capacity) is at least as great as to the traffic flow rate during the peak 15-minute period of the design hour, but not enough greater as to represent extravagance or waste. Where this objective is accomplished, a well-balanced, economical road system will result. A volume-capacity ratio higher than 0.65 should not be adopted at the design stage.

Guidelines for selection of design levels of service for the different classes of rural roads are given in various roadway design guides. AASHTO provides guidelines for selection of design levels of services for specified combinations of road functional class and terrain type. Guidelines for selection of design levels of service for the different classes of rural roads are also given in the design manuals for EAC Partner States. However, the level of service given varies between the roadway geometric design guides. The design guide for Uganda suggests design levels of service for rural roads for a given combinations of road functional class, road design class and terrain (Table 23) while levels of service selection guidelines are missing in the design guides for Kenya and Tanzania. Additionally, there is variation in the maximum service volumes suggested by the design guides for EAC Partner States as shown in Table 24.

Table 23: Selection of levels of service for rural roads in Uganda

Road Functional Class		Level of Service			
		Level terrain	Rolling terrain	Mountainous terrain	Road Design Class
A	International Trunk Roads	B	B	C	Ia Paved Ib, II and III Paved
B	National Trunk Roads	B	C	D	Ib, II and III Paved, A Gravel
C	Primary Roads	C	D	D	II and III Paved, A Gravel
D	Secondary Roads	D	D	E	A and B Gravel
E	Minor Roads	E	E	E	B and C Gravel

Table 24: Comparison of maximum service volumes for various levels of service

Level of Service	Tanzania	Uganda	
	Two-lane road (pcu/h)	Two lane rural	Multi lane rural without access control
A	400	Average travel speed of ≥ 93km/hr. Most passing manoeuvres 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.	Average travel speed ≥ 96 km/h. Under ideal conditions, flow rate is limited to 720 pcu/lane/hr or 33% of capacity.
B	900	Average travel speed of ≥ 88km/hr. Flow rates may reach 27% of capacity with continuous passing sight distance. Flow rate of 750 pcu/hr total for both direction.	Reasonably free flow. Volume at which actions of preceding vehicle will have some influence on following vehicles. Flow rates will not exceed 55% of capacity or 1,200 pcu/lane/h at 96 km/hr average travel speed under ideal condition.
C	1400	Flow still stable. Average travel speed of ≥ 84km/hr. Flow rates under ideal condition equal to 43% of capacity with continuous passing sight distance or 1,200 pcu/hr total for both direction.	Stable flow to a flow rate not exceeding 75% of capacity or 1,650 pcu/lane/h, under ideal conditions maintaining at least a 95 km/hr average travel speed.
D	1700	Approaching unstable flow. Average travel speed of ≥ 80km/hr. Flow rates, two directions, at 64% of capacity with continuous passing opportunity, or a total of 1,800 pcu/hr for both direction.	Approaching unstable flow at flow rates up to 89% of capacity or 1,940 pcu/lane/h at an average travel speed of about 92 km/h under ideal condition.
E	2000	Average travel speeds in neighbourhood of 72 km/hr. Flow rates under ideal conditions, total two way, equal to 2,800 pcu/hr. Level E may never be attained. Operation may go directly from Level D directly to Level F.	Flow at 100% of capacity or 2,200 pcu/lane/hr under ideal conditions. Average travel speeds about 88 km/h.
F	Varies from 0 to 2000	Forced congested flow with unpredictable characteristics. Operating speeds less than 72 km/hr.	Forced flow congested condition with widely varying volume characteristics. Average travel speed of less than 50 km/h.

Recommendation

We recommend that EAC should adopt guidelines shown in Table 25 for the selection of design levels of service for the different road classes.

Table 25: Proposed guidelines for selection of design levels of service

Level of service for specified combinations of terrain and area type					
Function	**Class**	**Level terrain**	**Rolling terrain**	**Mountainous terrain**	**Steep terrain**
Mobility roads	Class 1	B	B	C	C
	Class 2	B	C	D	D
	Class 3	C	C	D	D
Access roads	Class 4	D	D	D	D
	Class 5	D	D	D	D

2.3.3.8 Access Control and Access Management

Regulation of public access rights to and from properties abutting the highway facilities is called access control. It is usually achieved though a number of controls which can be categorized as full control of access, partial control of access, access management, and driveway/entrance regulations. The principal advantages of controlling access are the preservation or improvement of service and the reduction of crash frequency and severity (AASHTO, 2011).

Full control of access means that preference is given to through traffic by providing access connections by means of ramps with only selected public roads and by prohibiting crossings at grade and direct private driveway connections. With partial control of access, some preference should be given to through traffic. Access connections, which may be at-grade or grade-separated, are provided with selected public roads and private driveways (AASHTO, 2011).

Access management involves providing (or managing) access to land development while simultaneously preserving the flow of traffic on the surrounding road system in terms of capacity, speed, and low crash frequency and severity. Access management applies to all types of roads and streets. It calls for setting access policies for various types of roadways, keying designs to these policies, having the access policies incorporated into legislation, and having the legislation upheld in the courts. Access management views the highway and its surrounding activities as part of a single system. Individual parts of the system include the activity centre and its circulation systems, access to and from the centre, the availability of public transportation, and the roads serving the center. All parts are important and interact with each other (AASHTO, 2011).

Driveway/entrance regulations may be applied even though no control of access is obtained. Each abutting property is permitted access to the street or highway; however, the location, number, and geometric design of the access points are governed by the regulations (AASHTO, 2011).

Access management addresses the basic questions of when, where, and how access should be provided or denied, and what legal or institutional changes are needed to enforce these decisions. In a broad context, access management is resource management, since it is a way to anticipate and prevent congestion and to improve traffic flow. Key elements of access management include defining the allowable access and access spacing for various classes of highways, providing a mechanism for granting variances when reasonable access cannot otherwise be provided, and establishing means of enforcing policies and decisions. These key elements, along with appropriate design policies, should be implemented through a legal code that provides a systematic and supportable basis for making access decisions. The code should provide a common basis for decisions for both the public and private sectors (AASHTO, 2011).

The following principles define access management techniques (AASHTO, 2011).

- Classify the road system by the primary function of each roadway. Freeways emphasize movement and provide complete control of access. Local streets emphasize property access rather than traffic movement. Arterial and collector roads serve a combination of both property access and traffic movement.
- Limit direct access to roads with higher functional classifications. Direct property access should be denied or limited along higher class roadways whenever reasonable access can be provided to a lower class roadway.
- Locate traffic signals to emphasize through traffic movements. Signalized access points should fit into the overall signal coordination plan for traffi c progression.
- Locate driveways and major entrances to minimize interference with traffic operations. Driveways and entrances should be located away from other intersections to minimize crashes, to reduce traffic interference, and to provide for adequate storage lengths for vehicles turning into entrances.
- Use curbed medians and locate median openings to manage access movements and minimize conflicts. The extent of access management depends upon the location, type, and density of development, and the nature of the highway system. Access management actions involve both the planning and design of new roads and the retrofitting of existing roads and driveways.

Access can be managed and controlled by means of statutes, land-use ordinances, geometric design policies, and driveway regulations (AASHTO, 2011).

- Control by the transportation agency—Transportation agency has the basic statutory authority to control all aspects of highway design to protect public safety, health, and welfare. The extent to which an agency can apply specific policies for driveways/entrances, traffic signal locations, land use controls, and denial of direct access is specifically addressed by legislation.
- Land-use ordinances—Land-use control is normally administered by local governments. Local zoning ordinances and subdivision requirements can specify site design, setback distances, type of access, parking restrictions, and other elements that influence the type, volume, and location of generated traffic.
- Geometric design—Geometric design features, such as the use of raised-curb medians, the spacing of median openings, use of frontage roads, closure of median openings, and raised-curb channelization at intersections, all assist in controlling access.
- Driveway regulations—Agencies may develop detailed access and driveway/entrance policies by guidelines, regulations, or ordinances, provided specific statutory authority exists. Guidelines usually need no specific authority, but are weak legally. Cities can pass ordinances implementing access management policies.

- Likewise, government agencies may develop regulations when authorized by legislation. Regulations can deny direct access to a road if reasonable, alternative access is provided, but they cannot “take away” access rights.

Recommendation

EAC member states should put in place statutes, land-use ordinances, geometric design policies, and driveway regulations for managing and controlling access.

2.3.4 Design Elements

2.3.4.1 Sight Distance

A critical feature of safe road geometry is adequate sight distance. As an irreducible minimum, drivers must be able to see objects in the road with sufficient time to allow them to manoeuvre around them or to stop. Sight distance is dependent on the height of the driver’s eye above the road surface, the specified object height above the road surface, and the height and lateral position of sight obstruction within the driver’s line of sight. These are discussed and presented in Section 2.3.3.

Stopping Sight Distance

This is the distance required by a vehicle driven by a below-average driver at or near the design speed to stop before reaching a stationary object in its path. It is basically a sum of two distances namely; a brake reaction distance and a breaking distance. A beak reaction distance is the distance from the time the driver sights an object necessitating a stop to the time the brakes are applied, while a breaking distance is the distance required to stop the vehicle from the instant brakes are applied.

The brake reaction time corresponds to the perception reaction time which has been harmonised at a value of 2.5 seconds. The breaking distance is the distance to bring the vehicle to zero speed and depends on the brake force coefficient. Therefore the stopping sight distance can be determined as: D = 0.278Vt + 0.039(V2/a) where; t = brake reaction time, 2.5 sec; V = design speed, km/h; and a = acceleration rate of 3.4 m/s2. Table 26 gives the minimum stopping sight distance for the various design speeds.

Table 26: Stopping sight distance on level road

Design Speed (km/h)	30	40	50	60	80	100	110	120
SSD (m)	35	50	65	85	130	185	220	250

Passing Sight Distance

Passing sight distance also is an important criterion that affects the capacity as well as the quality of service provided by the road. A heavily trafficked road requires a higher percentage of passing sight distance than a lightly trafficked road to provide the same level of service. Passing sight distance is calculated on the basis of a distance required for a successful overtaking manoeuvre and makes adequate provision for an aborted manoeuvre in the case of a truck attempting to pass another truck.

The minimum passing sight distance is the total of four components; $d_1 + d_2 + d_3 + d_4$,
where
d_1 = distance traversed during perception-reaction time and during initial acceleration to the point where the passing vehicle just enters the right lane
d_2 = = distance travelled during the time the passing vehicle is travelling in the right lane
d_3 = = distance between the passing vehicle and the opposing vehicle at the end of the passing manoeuvre
d_4 = = distance moved by the opposing vehicle during two thirds of the time the passing vehicle is in the left lane (usually taken to be 2/3 d_2)

AASHTO provide minimum passing sight distances rounded for design purposes (Table 27), which in designing a highway should be exceeded as much as practical.

Table 27: Passing sight distances for design of two-lane highways

Design speed (km/h)	**Assumed speeds**		**Passing sight distance (m)**	
	Passed vehicle	**Passing vehicle**	**Total passing sight distance**	**Rounded for design**
30	29	44	200	200
40	36	51	266	270
50	44	59	341	345
60	51	66	407	410
70	59	74	482	485
80	65	80	538	540
90	73	88	613	615
100	79	94	670	670
110	85	100	727	730
120	90	105	774	775
130	94	109	812	815

Source AASHTO (2004)

Recommendation

We recommend adoption of sight distances given in Table 26 and 27 as minimum values for stopping and passing sight distances, respectively.

2.3.4.2 Horizontal Alignment

The straight

Straight sections provide better visibility and more passing opportunities and hence enhance safety. However, long straight sections increase the danger from headlight glare and usually lead to excessive speeding. In hot climate areas, long tangents have been shown to increase driver fatigue and hence cause accidents. Thus, the design guides have attempted to limit the length of straight sections. Table 28 shows the approaches used by other countries to limit the length of straight sections.

Table 28: Minimum and Maximum Tangent Length in Other countries

Country	Min. tangent length	Max. tangent length
AASHTO	- (favours long sections for passing purpose)	
Germany	6 times design speed in km/h (m)	20 times design speed in km/h (m)
France		2 to 3 km
Swiss highway officials	Sections permitting one minute of driving not permitted	

Design guide for Kenya recommended two guidelines to be observed and applied for the lengths of straights:

- Straights should not have lengths greater than ($20xV_D$) metres (where V_D is design speed in km/h)
- Straights between circular curves following the same direction should have lengths greater than ($6xV_D$) metres (V_D in km/h)

Design guides for Tanzania and Uganda recommended that the length of straights on a road should not exceed 2 km and 4.0 km respectively. SATCC design guide observed that American studies have shown that when the distance between successive curves is such that superelevation development is represented by a continuous rollover from one side of the road to the other, the crash rate is high. When the distance from the end of one curve to the commencement of the next curve is of the order of 20 km the crash rate is at about the same value as for continuous curvature. In between these two extreme values the collision rate declines and then increases in a nearly symmetrical parabolic fashion with its minimum value being at a tangent length of about 12 km. It is recommended that this should be considered a desirable length of tangent in the case where vehicles tend to travel at unaltered speed along tangents and around curves, i.e. for a design speed of 120 km/h or more.

At lower design speeds, a tangent of this length would cause speeds to creep up to about 120 km/h or even higher and the driver would have to reduce speed to negotiate the following curve thereafter accelerating again. Ideally, drivers should be encouraged to maintain a speed which is close to that selected for design purposes to reduce the possibility of an error of judgment leading to a crash. It has been found that, under these circumstances, a maximum tangent length which, when measured in metres, is 20 times the design speed in km/h, achieves this effect. For example, a design speed of 80 km/h would suggest that tangents should not be longer than about 1.6 km.

Where the topography is flat, a long tangent is a significant problem in night driving. A driver is very aware of approaching lights for as much as two minutes before the vehicles actually pass each other and, in the second of the two minutes, reference can be made to dazzle which becomes increasingly severe. Two vehicles approaching each other, when they are both travelling at say 120 km/h, are as much as 8.0 km apart at the commencement of the two minute period referred to and are still 4.0 km apart at the onset of dazzle. During the last fifteen seconds, at the commencement of which the two vehicles are still a kilometre apart, the drivers can only really guide their vehicles by concentrating on their left road edge at a point which will not be much more than about 50 m in advance of their present position. Seeing that, at 120 km/h, a vehicle requires a stopping distance of 210 m, this is a potentially hazardous situation.

Where large volumes of nighttime traffic are expected, it may be necessary to consider tangent lengths shorter than the 12 km proposed above. In extreme cases, it may even be necessary to consider including a median in the cross-section and planting shrubs in it or providing some other means of reducing dazzle. Because light from headlights hit it at a very flat angle, a conventional fence is remarkably effective in reducing glare.

The minimum length of tangent must also allow for the run-off of the super-elevation of the preceding curve followed by the development of that for the following curve. This distance should actually be calculated during detailed design but, as a rough rule of thumb, a tangent length of less than 200 m is likely to prove inadequate.

Recommendation

Straights should not have length greater than (20xV_D) m (where V_D is design speed in km/h and a minimum length of (6xV_D) m should also be adopted between circular curves following the same direction.

Minimum radius of horizontal curve

AASHTO (2011) noted that accumulated research and experience have established limiting values for superelevation rate (e_{max}) and side friction demand (f_{max}) for horizontal curve design. Use of these limiting values in a horizontal curve formula permits determination of a minimum curve radius for various design speeds. Use of lower values than the superelevation and side friction limiting values results in curves with radii larger than the minimum. The amount by which each factor is below its respective limit is chosen to provide an equitable contribution of each factor towards sustaining the resultant centripetal acceleration. AASHTO elaborates five methods that are used to achieve this equity for different design situations.

AASHTO recommends maximum superelevation rates be limited to 10%, although 12% is used in some cases for rural roadways, and 6% or 4% for urban roads where traffic congestion or extensive marginal development acts to restrict top speeds. It also suggests that superelevation may be omitted on low-speed urban streets where severe constraints are present. It is recommended that several rates, rather than a single rate, of maximum superelevation should be recognised in establishing design controls for highway curves. Additionally, there is a tradeoff between the maximum rate of superelevation and the minimum curve radius permitted at any design speed. The maximum superelevation rates really apply only to fairly low design speeds. A lower value of maximum superelevation should be considered in a road where steep gradients occur with any frequency. Also, values of maximum side friction vary with design speed.

While the maximum superelevation rates are limited to 10% (although 12% is used in some cases for rural roadways) and 6% or 4% for urban roads and side friction factors of 0.08 to 0.18 are used in the AASHTO guide, the maximum rate of super-elevation recommended by SATCC guide for the design of rural roads is 10 per cent. The SATCC guide provides minimum radii at superelevation rates of 6%, 8% and 10% and it recommends that a lower value of superelevation rate should be applied to gravelled surfaces since the rate of 10% would make them scour.

The draft design manual for Tanzania adopts superelevation rates ranging from 6% to 8% while ranges of superelevation rates adopted by Uganda manual ranges from 4 to 8%. In urban areas where traffic congestion or extensive marginal development acts to curb top speeds, the Uganda manual recommends use of a low maximum rate of superelevation, usually 4 percent. Similarly, either a low maximum rate of superelevation or no superelevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals. Kenya design manual provides minimum horizontal curve radii with respect to design speeds without specifying limiting values of superelevation rates and side friction.

Because of congestion and the application of traffic control devices in the urban areas, the speeds achieved at any point along the road can fluctuate between zero and the posted speed - or even higher depending on the local level of law enforcement. Negotiating a curve with a superelevation of 10 per cent at a crawl speed can present a major problem to the driver. As a general practice, urban superelevations do not exceed 6 per cent although, in the case of an arterial, this could be taken as high as 8 per cent, provided that this value of superelevation is used only between intersections and that the superelevation is sufficiently remote from the intersections for full run-off to be achieved prior to reaching the intersection area.

In other cases, minimum radii or curve lengths for roadways may be established by the need to provide stopping sight distance or by appearance standards. Where deflection angles are small a short horizontal curve may give the appearance of a kink. To prevent this, minimum horizontal curve lengths may be specified for curves with small deflection angles. For a given design speed, minimum curve radius is limited by maximum allowable side friction, which is usually based on comfort standard, maximum superelevation rate for the curve, and the necessity to maintain stopping sight distance.

The other limiting factor is side friction. Side friction factors are dependent on speed, road surface condition or texture, weather conditions, and type and condition of tires. For the purposes of design, it is desirable to select a value lower than the limit at which skidding is likely to occur and the international general practice is to select values related to the onset of feelings of discomfort. AASHTO recommends further that horizontal curves should not be designed directly on the basis of the maximum available friction factors. Rather, the maximum side friction factor used in design should be that portion of the maximum available side friction that can be used with comfort and safety by the vast majority of drivers.

AASHTO noted that side friction factors developed between new tires and wet concrete pavements range from about 0.5 at 30 km/h to approximately 0.35 at 100 km/h. for wet concrete pavements and smooth tires the maximum side friction factor at impending skid is about 0.35 at 70 km/h. In all cases, a decrease in side friction values was noted as speed increased. It was further noted from other tests that speeds on curves that avoid driver discomfort 30 km/h, 40 km/h and 50 km/h through 80 km/h the side friction factors of 0.21, 0.18 and 0.15, respectively provided ample margin of safety against skidding. In view of other tests, AASHTO noted that a maximum side friction factor of 0.16 for speeds up to 100 km/h was recommended. Speed studies documented by AASHTO concluded that the side friction factor should not exceed 0.10 for design speeds of 110 km/h and higher. Generally, AASHTO noted that the maximum side friction values vary directly with design speed from 0.08 for high speed (130 km/h) roads to 0.4 on lower speed (15 km/h) roads. The results of empirical studies have indicated 0.22 as a side friction factor above which passengers experience some discomfort.

Canadian practice suggests that the side friction factor be taken as $f = 0.21 - 0.001 \times V$ where V is vehicle speed (km/h). In the SATCC guide, the maximum side friction factor accepted for design purposes is expressed as f_{max} = 0.19 - V/1600 where V = Speed (km/h) representing a safety factor of approximately three.

Side friction factors ranging from 0.09 to 0.17 are recommended in the Tanzania and Uganda design manuals as shown in Table 29 below.

Table 29: Side Friction Factors for Different Design Speeds

Design speed (km/h)	Limiting value of *f*
30	0.17
40	0.17
50	0.16
60	0.15
70	0.14
80	0.14
90	0.13
100	0.12
110	0.11
120	0.09

Source: Uganda Design Manual (2005)

Recommendations

1. *Adopt maximum superelevation rate of 4% on roads in urban areas and 10% on roads in the rural areas.*
2. *Adopt side friction factors shown in Table 29 with corresponding design speed.*

Minimum and maximum lengths of curve

For small deflection angles, it is required that curves should be long enough to avoid the appearance of a kink. SATCC design guide recommends a minimum length of curve of 300 m and if space is limited this length may be reduced to 150 m. For deflection angles of less than 5^o, the minimum length of the curve should be increased from 150 m by 30 m for each 1^o decrease in the deflection angle. On the other hand, a long curve, particularly if it is of near-minimum radius, may causes tracking problems. These are experienced principally by vehicles travelling at speeds markedly different from the design speed of the road, and the main complication introduced by a long curve is its possible effect on passing opportunities. SATCC design guide recommends that the length of a curve should not exceed 1 000 m and the preferred maximum length is 800 m. This length applies also to multilane cross-section because, while passing opportunities do not pose a problem, tracking still remains an issue.

For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. A widely adopted guideline is that, on minor roads, curves should have a minimum length of 150 metres for a deflection angle of 5o and that this length should be increased by 30 metres for every 1o decrease in deflection angle. On major roads and freeways, the minimum curve length in metres should be three times the design speed in km/h. The increase in length for decreasing deflection angle also applies to these roads. In the case of a circular curve without transitions, the length in question is the total length of the arc and, where transitions are applied; the length is that of the circular curve plus half the total length of the transitions. South African practice recommends an upper limit to the length of horizontal curves. Curves to the left generally restrict passing opportunities and, furthermore, dependant on their radius, operation on long curves tends to be erratic. For this reason, it is desirable to restrict the length of superelevated curves to a maximum of 1 000 metres. If a curve radius is such that the curve either has a normal camber or a crossfall of 2 per cent, the limitation on curve length falls away and the curve can be dealt with as though it were a tangent.

Recommendation

The length of curve should not exceed 1000 m while the minimum length should be 300 m.

Transition curve

A transition curve give the driver a natural path that is easy to follow as a vehicle enters or leaves a circular horizontal curve and at the same time provides a suitable arrangement for superelevation runoff. The appearance of the road is also enhanced by the use of transition curves, since this avoids noticeable breaks at the beginning of circular curves, these breaks often being made more pronounced by the superelevation runoff. They are most appropriate for roadways with relatively high design standards, where large-radius curves are used. In horizontal curve design lateral acceleration is equivalent “centripetal acceleration”, the term lateral acceleration is used.

The design of the transition includes consideration of the changes of the alignment from tangent to circular as well as the cross section from normal to superelevated. An Euler spiral (clothoid) is the most common to be used in the design of a spiral transition from tangent to curve and curve to tangent. A spiral transition curve give the driver a natural path that is easy to follow as a vehicle enters or leaves a circular horizontal curve and at the same time provides a suitable arrangement for affecting the superelevation runoff. The appearance of the road is also enhanced by the use of spiral transition curves, since this avoids noticeable breaks at the beginning of circular curves, these breaks often being made more pronounced by the superelevation runoff. The maximum relative gradient between pavement edges (Table 30) will determine the minimum length for the runout and runoff and hence the minimum length of the transition curves.

Table 30: Maximum relative gradient

Design Speed (km/h)	30	40	50	60	80	100	110	120
Maximum relative gradient	1:133	1:143	1:154	1:167	1:200	1:227	1:244	1:263

AASHTO (2011) observe that a minimum length of transition curve is defined based on consideration of driver comfort and shifts in the lateral position of vehicles. Criteria based on driver comfort are intended to provide spiral length that allows for a comfortable increase in lateral acceleration as a vehicle enters a curve. The criteria based on lateral shift are intended to provide a spiral curve that is sufficiently long to result in a shift in a vehicle's lateral position within its lane that is consistent with that produced by the vehicles natural spiral path. It is recommended that these two criteria can be used together to determine the minimum length of spiral which can be computed as

$$L_{s,\min} = \sqrt{24(p_{\min})R} \text{ or } L_{s,\min} = 0.0214\frac{V^3}{RC}$$

Where:

- $L_{s,\min}$ = minimum length of spiral, m
- $p_{\min}$ = minimum lateral offset between the tangent and circular curve (0.20 m)
- R = radius of circular curve, m
- V = design speed, km/h
- C = maximum rate of change in lateral acceleration (1.2 m/s^3)

AASHTO (2011) also reports that international experience indicates that there is a need to limit the length of spiral transition curve. Spirals should not be so long (relative to the length of the circular curve) that drivers are misled about the sharpness of the approaching curve. A conservative maximum length of spiral that can minimize the likelihood of such concerns can be computed as:

$$L_{s,\max} = \sqrt{24(p_{\max})R}$$

Where:

- $L_{s,\max}$ = maximum length of spiral, m
- $p_{\max}$ = minimum lateral offset between the tangent and circular curve (1.0 m)
- R = radius of circular curve, m

Further, AASHTO (2011) provides desirable lengths of spiral transition curves as shown in Table 32. These lengths correspond to 2.0 s of travel time at the design speed of the roadway. The travel time has been found to be representative of the natural spiral path for most drivers and thus the spiral lengths listed in Table 31 are recommended as desirable values for street and highway design. The use of longer spiral curve lengths that are less than $L_{s,\max}$ than is acceptable. However, where such longer spiral curve lengths are used, consideration should be given to increasing the width of the travelled way on the curve to minimize the potential for encroachments into adjacent lanes. Further, if the desirable spiral curve length shown in Table 31 is less than the minimum spiral curve length $L_{s,\min}$, the minimum spiral curve length should be used in design.

Table 31: Desirable length of spiral curve transition

Design speed (km/h	20	30	40	50	60	70	80	90	100	110	120	130
Spiral length (m)	11	17	22	28	33	39	44	50	56	61	67	72

In Kenya transition curves are not required for horizontal curves with radii greater than 2000 m. Design guides for Tanzania and Uganda recommended the application of transition curves to circular curves under the following condition: $R < \frac{V^3}{432}$

where:
R = Radius of curve in metres,
V = Design speed in km/hr

Recommendation

We propose the maximum relative gradient given in table above for various speeds for determination of the minimum length for the runout and runoff. Additionally, the minimum length of transition curve should be based on consideration of driver comfort and shifts in the lateral position of vehicles and these two criteria can be used together to determine the minimum length of transition curve.

2.3.4.3 Vertical Alignment

Gradients

Maximum grades vary, depending on the type of facility, and usually do not constitute an absolute standard. The effect of a steep grade is to slow down the heavier vehicles and increasing operating costs and the extent to which any heavier vehicle is slowed depends on both the steepness and length of the grade. The effect the slowing of the heavier vehicle therefore depends on the situation, and is therefore more a matter of traffic analysis than simple geometric design. As a result, the maximum grade for a given facility is a matter of judgement, with the tradeoffs usually being cost of construction versus speed. Steep descending gradients may cause braking problems to HGVs and thus impair road safety. In particular, it is necessary to avoid locating a moderate descending gradient between two steep descending gradients (difficulties in reducing speeds to a suitable level) but also positioning critical points in or immediately after zones with steep gradients. Likewise, in ascending gradients, they cause traffic flow problems (which can make it necessary to provide crawler lanes) and excess fuel consumption (beyond an ascending gradient of 2.5 % each additional percentage point of gradient leads to a 12 % increase in fuel consumption compared with a flat road).

In theory, vertical curves in opposite directions do not require grades between them. In practice, however, the outcome is visually not successful. The junction between the two curves creates the impression of a sharp step in the alignment, downwards where a sag curve follows crest and upwards where the crest curve follows the sag. A short length of grade between the two curves will create the impression of continuous, smoothly flowing vertical curvature. The length of the intervening grade in metres need not be more than the design speed in km/h to achieve this effect. Where the total change of gradient across a vertical curve is very small, e.g. less than 0,5 per cent, the K-value necessary to achieve the minimum length of curve would be high. Under these circumstances, the vertical curve could be omitted altogether without there being an adverse visual impact.

Maximum gradient guidelines range in complexity, with various countries considering some or all of the following factors: road classification, design speed and terrain. For example in the UK, desirable maximum gradient values are specified for road types: motorway (3 %), dual carriageway (4 %), and single carriageway (6 %). In Switzerland, maximum gradient is a function of design speed (from 10 % for a 60 km/h to 4 % for 120 km/h design speed). In Germany, maximum gradient is a function of road type and design speed.

In the USA maximum gradients are based upon road type, topography and design speed. AASHTO guide observed that passenger cars can readily negotiate grades as steep as 4% to 5% without an appreciable loss in speed below that normally maintained on level roadways, except for cars with high weight/power ratios, including some compact and subcompact cars. Studies also show that, under uncongested conditions, operation on a 3% upgrade, has only a slight effect on passenger car speeds compared to operations on the level. However, the effect of truck speeds is much more pronounced than on speeds of passenger cars. Thus, a maximum grade of 5% for design speeds greater than 100 km/h and 5-8% for 60 – 100 km/h design speeds are recommended by the AASHTO guide. Generally, for main rural roads values range from 8 % for 60 km/h to 4 % for 120 km/h design speed. On the other hand, the maximum value recommended by the French Road Act range in between 8 to 10% and should be complied with reference to sensitivity in particular in winter conditions.

In Tanzania and Uganda, maximum grades are based on design speed and topography as shown in Tables 32 and 33. As it can be noted from the tables, there is no much difference in the grades specified by the two design manuals.

Table 32: Maximum gradients specified in Tanzania design manual

Topography	**Maximum gradient [%] for design speed [km/h]**						
	40	**60**	**70**	**80**	**100**	**110**	**120**
Flat	5	5	5	4	3	3	3
Rolling	8	6	6	5	-	-	-
Mountainous	10	8	8	8	-	-	-

Table 33: Maximum gradients specified in Uganda design manual

Topography	**Maximum gradient [%] for design speed [km/h]**								
	40	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
Flat	-	-	5	4	4	3.5	3	3	3
Rolling	-	7	6	5.5	5	4.5	4.5	-	-
Mountainous	10	9	8	7	-	-	-	-	-

Minimum grades are sometimes specified for roadways and are normally intended to provide for drainage on curbed facilities. AASHTO recommends that a typical minimum grade is 0.5% but a 0.3% grade may also be used where there is a high type pavement accurately sloped and supported on firm subgrade. On the other hand, the French Road Act recommends a minimum of 0.5 to 1% for zones with no superelevation in order to allow surface water to drain away and 0.2% in long sections in cuts in order to avoid the need for extra depth for the longitudinal storm water drainage system.

It should be noted minimum grade in itself is not a complete design control. AASHTO suggest consideration of the length of a particular grade in relation to desirable vehicle operation. The term 'critical length of grade' is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given grade, lengths less than critical lengths result in acceptable operation in the desired range of speeds. The guide also notes suggests that if the desired freedom of operation is to be maintained on grades longer than critical, design adjustments such as changes in location to reduce grades or addition of extra lanes should bee considered. The data for critical lengths of grade should be used with other pertinent factors such as traffic volume in relation to capacity to determine where added lanes are warranted.

To avoid standing water in side ditches, the minimum gradient for roads in cutting is recommended to be 0.5% for Kenya. It is further recommended to check for length of grades or otherwise climbing lane may be justifiable in some cases. For Tanzania, the draft guide proposed minimum grades on cut sections to be 0.5% unless special drainage treatments are provided while the recommended minimum gradient in cuttings in order to avoid standing water in the road side ditches is 0.3% - 0.5% in Uganda.

Recommendation

Adopt 0.5% as minimum grade and maximum grades shown in Table 34.

Table 34: Recommended maximum grades

Terrain	Maximum gradient (%)
Flat	4
Rolling	4 – 8
Mountainous	7 – 12
Steep	12 - 18

Climbing lanes

A climbing lane is an auxiliary lane added outside the continuous lanes and has the effect of reducing congestion in the through lanes by removing slower-moving vehicles from the traffic stream. As such, it is used to match the Level of Service on the rising grade to that prevailing on the level sections of the route. The climbing lane is also referred to as a crawler lane, truck lane and, confusingly, even as a passing lane. The passing lane is also an auxiliary lane but is typically provided on level sections of the route.

The SATCC guide noted that the maintenance of an acceptable level of service over a section of the route is one of the reasons for the provision of climbing lanes. Another reason is the enhancement of road safety by the reduction of the speed differential in the through lane. The warrants for climbing lanes are therefore based on both the speed and volume of the traffic. A further warrant that could be considered is based on the intention to match levels of service along the route. A Level of Service (LOS) analysis would then be undertaken for the grade in question. Various warrants have been proposed from time to time. Typically, a drop of two levels of service, e.g. from LOS B to LOS D is considered to be adequate grounds for the provision of a climbing lane. The only weakness of the capacity analysis approach is that each level of service implies a range of operating conditions. A road could be operating at just above the boundary between LOS B and

LOS C while, on the upgrade of interest, the level of service is just below the boundary between LOS C and LOS D. In practice, there has not, in this example, really been a drop of two levels of service. An alternative to these warrants is to consider some form of economic analysis. For example, software has been developed that relates the cost of construction of the climbing lane to the value of time saved by its provision. The analysis is based on calculation of delay that would ensue over the design life of the road if the climbing lane was not provided.

AASHTO 2004 edition is recommending the following criteria to be fulfilled for climbing lane to be introduced.

- i) Upgrade traffic flow rate in excess of 200 vehicles per hour
- ii) Upgrade truck flow rate in excess of 20 vehicles per hour
- iii) One of the following conditions exists:
  - a) A 15 km/h or greater speed reduction is expected for a typical heavy truck
  - b) Level of service E or F exists on the grade
  - c) A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

In addition safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes.

AASHTO requirements for climbing lanes are simple and require data which is easily obtainable for site assessment, and above all they include a safety criterion.

Design guide for Kenya recommended the following guidelines to warrant the design and provision of climbing lanes:

- i) Climbing lanes will not be required on (i) roads with AADT < 2000 pcu in design year 10, and (ii) on all D and E class roads even if the AADT exceeds 2000 pcu in design year 10.
- ii) Where passing opportunities are limited on the gradients, then climbing lanes must be considered on design class on A, B, and C class roads with traffic flows in design year 10 is in the range of 2000 pcu < AADT < 6000 pcu.
- iii) Climbing lanes will normally required on roads with AADT ≥ 6000 pcu in design year 10.
- iv) Climbing lanes should be considered on A class roads when the speed of a typical heavy vehicle falls by 15 km/h and the corresponding fall in speed applicable to B and C class roads shall be 20 km/h.

It further recommended that, where climbing lanes are required the widths of the through traffic lanes be reduced to 3.25 m in cross-section type II, 3.00 m in cross-section type III and IV. Over the sections where climbing lanes are required, the shoulder widths shall be reduced on both sides of the road to the values tabulated on page 5.24. The introduction and termination of a climbing lane shall be effected by tapers of length 60 m.

Design guide for Tanzania recommended the following guidelines to warrant the design and provision of climbing lanes:

- i) A traffic volume of ADT ≥ 1500 shall warrant introduction of climbing lane when the critical length of gradients is exceeded.
- ii) When the speed of a typical heavy vehicle falls by 20 km/h.
- iii) It is recommended in the Tanzanian manual that, the width of the climbing lanes be the same as the width of the adjacent lane. The minimum entry and exit tapers length is 100 metres.

While design guide for Uganda recommended that:

i) Climbing lanes will not be required on (i) roads with AADT < 2000 pcu in design year 10, and (ii) on all design class III, A, B and C roads even if the AADT exceeds 2000 pcu in design year 10.

ii) Where passing opportunities are limited on the gradients, then climbing lanes must be considered on design class I, & II roads with traffic flows in design year 10 is in the range of 2000 pcu < AADT < 6000 pcu.

iii) Climbing lanes will normally be required on roads with AADT ≥ 6000 pcu in design year 10.

iv) Climbing lanes should be considered if the design truck speed decreases more than 20 km/h under the truck speed limit, normally 80 km/h in rural conditions.

The existing design guides for EAC Partner states show that each of the EAC Partner state is following different requirements for climbing lanes. Comparing these requirements with those prescribed in the AASHTO guide, it is found that AASHTO requirements for climbing lanes are simple to use because they require data (i.e. traffic volumes and speed only) which can easily be obtained for site assessment, and above all they include a safety criterion. We therefore recommend use of AASHTO requirements for climbing lanes.

Recommendation

Adopt the following warrants for climbing lanes.

i) *Upgrade traffic flow rate in excess of 200 vehicles per hour*

ii) *Upgrade truck flow rate in excess of 20 vehicles per hour*

iii) *One of the following conditions exists:*

   a) A 15 km/h or greater speed reduction is expected for a typical heavy truck

   b) Level of service E or F exists on the grade

   c) A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

In addition safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes

Vertical curves

Vertical curves should be simple in application and should result in a design that is safe and comfortable in operation, pleasing in appearance, and adequate for drainage. The major control for safe operation on crest curves is the provision of ample sight distances for the design speed; while research has shown that vertical curves with limited sight distance do not necessarily experience frequent crashes, it is recommended that all vertical curves should be designed to provide at least the stopping sight distances and additional sight distance should be provided at decision points. For driver comfort, the rate of change of grade should be kept within tolerable limits. This consideration is most important in sag vertical curves where gravitational and vertical centripetal forces act in opposite directions. Appearance also should be considered in designing vertical curves. A long curve has a more pleasing appearance than a short one; short vertical curves may give the appearance of a sudden break in the profile due to the effect of foreshortening (AASHTO, 2011).

AASHTO (2011) recommends that minimum lengths of crest vertical curves should be based on sight distance criteria generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as ramp exit gores, where longer sight distances and, therefore, longer vertical curves should be provided. The basic formulas for length of a crest vertical curve in terms of algebraic difference in grade when stopping sight distance is taken as a control are:

When S is less than L, $$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

When S is greater than L, $$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}$$

Where: L = length of vertical curve, m

S = Stopping sight distance, m

A = algebraic difference in grades, %

h_1 = height of eye above roadway surface, m

h_2 = height of object above roadway surface, m

When the height of eye and height of object are 1.08 m and 0.15 m respectively, as it is used in the stopping sight distance the above formulae become:

When S is less than L, $$L = \frac{AS^2}{658}$$

When S is greater than L, $$L = 2S - \frac{658}{A}$$

On curbed roadways on sag curves grades of not less than 0.5%, or in some cases, 0.3% for the outer edge of the roadway should be provided for drainage purposes. Furthermore, minimum lengths of crest vertical curves based on sight distance criteria (at eye height of 1.08 m and object height of 0.6 m) generally are satisfactory from the standpoint of safety, comfort, and appearance except at decision areas such as sight distance to ramp exit gores, where longer lengths are needed (AASHTO, 2011).

The horizontal circular curve provides a constant rate of change of bearing. Analogous to this is the vertical parabola which provides a constant rate of change of gradient. The reciprocal of the rate of change of grade, K, is thus the distance required to effect a unit change of grade. Vertical curves are specified in terms of this factor, K, and their horizontal length calculated by multiplying K by the algebraic difference, A, in percentage between the gradients on either side of the curves so that $L = A \times K$.

AASHTO guide (2004) gives the minimum lengths of vertical curves for different values of A to provide the minimum stopping sight distances for each design speed. It also gives the computed K values for lengths of vertical curves corresponding to the stopping sight distances for each design speed. The design control in terms of K covers all combinations of A and L for any one design speed and therefore A and L need not be indicated separately in a tabulation of design value. The selection of design curves is facilitated because the minimum length of curve in metre is equal to K times the algebraic difference in grades in percent, L=KA. Conversely, the checking of plans is simplified by comparing all curves with the design value for K.

For crest vertical curves, where S=L, the value of K or length of vertical curve per percent change in A is a positive whole number that is indicative of the rate of vertical curvature while where S is greater than L, and for small values of A, the vertical curve lengths are zero because the sight line passes over the high point. The latter relationship does not represent desirable practice and a good practice is to use a minimum length of vertical curve, expressed as a single value, a range for different design speeds, or a function of A. The most common values in use range from 30 to 100 m. However, to recognise the distinction in design speed and to approximate the range of current practice AASHTO observes that minimum length (in metres) of vertical curves are expressed as about 0.6 times the design speed in km/h. Furthermore, AASHTO notes that it is impractical to design crest curves to provide for passing sight distance because of high cost where crest cuts are involved and the difficulty of fitting the resulting long vertical curves to the terrain, particularly for high-speed roads except where there is unusual combinations of low design speeds and gentle grades or higher design speeds with very small algebraic differences in grades.

For sag curves, four different criteria for establishing lengths of sag curves are known; headlight sight distance, passenger comfort, drainage control and general appearance. Headlight sight distance is recommended by AASHTO for determining the length of sag curves. For overall safety on highways, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. As in the case of crest curves, it is convenient to express the design control in terms of the K rate for all values of A. Minimum lengths of vertical curves (i.e. 0.6 times the design speed in km/h) for the flat gradients also are recognised for sag conditions.

There is no general agreement as to the maximum value of radial acceleration. AASHTO suggests 0.3 m/s^2. Further appearance standards, which aim at avoiding short vertical curves that look like kinks when viewed from a distance, vary from agency to agency.

Design guide for Kenya recommended that the length of both sag and crest curves should generally be not less than (2 x V_D) metres where V_D is the design speed in km/h. Design guides for Tanzania and Uganda suggest the vertical curves lengths in terms of rate of curvature K as given by:

$L = KA$

Where:

L =	Minimum length of vertical curve
K =	Rate of vertical curvature per change in grade given as meters per percent grade change
A =	Algebraic difference between the gradient (%)

In order to compare values for vertical curves, the AASHTO ‘K-values’ have been converted to the approximate corresponding radii values and minimum radii for various design speeds are usually given and therefore can be compared.

Sag vertical curves are generally considered as less critical from the safety point of view than crest curves. Several countries base their values for minimum sag curves on headlight illumination distances to satisfy stopping sight distance requirements on unlit roadways at night. Other countries base their design values on driver comfort. In Germany, the minimum radius of sag curves is one half the minimum radius of crest curves art a given design speed.

Recommendation

Minimum lengths of vertical curves should be based on stopping sight distance criteria and the length of a curve (in metres) should not be shorter than the design speed. In the case of freeways, the minimum length should not be less than twice the design speed in km/h.

For crest vertical curves, the minimum length standard depends on the stopping sight distance, the height of the driver's eye measured from an eye height of 1.05 m, and the height of the object of 0.6 m to be seen over the crest of the curve. For sag curves, stopping sight distance is based on the distance illuminated by the headlight height of 0.6 m at night.

2.3.5 Cross Section Elements

2.3.5.1 Lane Width

A fundamental feature of roadway cross section is the width of a travel lane, which must be sufficient to accommodate the design vehicle, allow for imprecise steering manoeuvres, and provide clearance for opposing flow in adjacent lanes. In cases of paved road, lane width refers to the net width excluding width of edge strip, edge line and centreline markings. Wide lanes supposedly promote the safety of the occupants of vehicles although current evidence suggests that there is an upper limit beyond which safety is reduced by further increases in lane width. Wide lanes have a negative impact on the safety of pedestrians attempting to cross the road or street. Research has indicated that the crash rate starts to show a marginal increase above a lane width of 3.6 m. It is accordingly recommended that widths significantly greater than 3.7 m should not be employed. Some local authorities use lane widths of 5.4 m on major routes. The intention is apparently to make provision for parking or cycle traffic without demarcating the lanes as such.

AASHTO (2004) reported that lane widths of 2.7 to 3.6 m are generally used with a 3.6 m lane predominant on most high-type highways. The extra cost of providing a 3.6 m lane width, over the cost of providing a 3.0 m lane width is offset to some extent by a reduction in cost of shoulder maintenance and a reduction in surface maintenance due to lessened wheel concentrations at the pavement edges. The wider 3.6 m lane provides desirable clearances between large commercial vehicles travelling in opposite directions on two-lane rural highways when high traffic volumes and particularly high percentages of commercial vehicles are expected. Lanes less than 3.6 m, i.e. 3.3 m, can be used in urban areas where pedestrian crossings, right-of-way, or existing development become stringent controls. Lanes 3.0 m wide are acceptable on low-speed facilities, and lanes 2.7 m wide are appropriate on low volume roads in rural and residential areas. Lane width also affects highway level of service. Narrower lanes force drivers to operate their vehicles closer to each other than they would normally desire. Restricted clearances have much the same effect. Auxiliary lanes at intersections and interchanges often help to facilitate traffic movements and therefore they should be as wide as the trough-traffic lanes but not less than 3.0 m. Furthermore, the guide observes that it is not cost-effective to design the lane and shoulder with of local and collector roads and streets that carry less than 400 vehicles per day using the same criteria applicable to higher volume roads.

SATCC guide recommended that the greatest lane width should be 3.7 m for higher volume roads and speeds. Although there is no operational or safety benefit accrued from lane widths wider than 3.7 m, for different reasons urban lane widths can be as great as 5.5 m. The narrowest lane width is recommended to be 3.1 m wide giving a clear space of 0.3 m on either side of a vehicle that is 2.5 m wide. This lane width can only be used in places where speeds or traffic volumes are expected to be low while intermediate conditions of speeds and traffic volume can be adequately catered for by a lane width of 3.4 m. Where traffic volumes are such that a multi-lane cross-section or a divided cross-section is required, 3.7 m is a logical lane width to adopt. Lesser lane widths may however be warranted by abnormal circumstances.

Schramm and Rakotonirainy (2009) provide a summary of lane widths according to roadway classifications for various countries as shown in Table 35. Hall et al. (1995) noted that South Africa values are 3.7 m for freeways, 3.1 to 3.7 m for rural arterials, 3.0 to 3.7 m for urban arterials, and 2.25 to 3.0 m for minor local roads. It can be said that lane widths vary from country to country, but within reasonably narrow ranges: typically 3.5 to 3.75 m for freeways, 3.0 to 3.7 for arterials, and 2.7 to 3.65 for local roads.

Table 35: Comparison of lane widths for various countries

Country	**Roadway classification**		
	Freeway	**Arterial**	**Minor or local**
Brazil	3.75	3.75 m	3.0 m
Canada		3.0 to 3.7 m (rural)	3.0 to 3.3 m
China	3.5 to 3.75 m	3.75 m	3.5 m
Denmark	3.5 m	3.0 m	3.0 m to 3.25 m
France	3.5 m	3.5 m	3.5 m
Germany	3.5 to 3.75 m	3.25 to 3.5 m	2.75 to 3.25 m
Japan	3.5 to 3.75 m	3.25 to 3.5 m	3.0 to 3.25 m
Poland	3.5 to 3.75	3.0 to 3.5 m	2.5 to 3.0 m
United Kingdom	3.65 m	3.65 m	3.0 m to 3.65 m
USA	3.6 m	3.3 to 3.6 m	2.7 to 3.6 m

Source: Schramm and Rakotonirainy (2009)

Saakatoon and Saskatchewan (2000) adopted a performance-based approach to estimate lane width requirements for heavy vehicles travelling along straight paths using state-of-the art computer modelling techniques. The study covered nine types of heavy vehicles, a wide range of speed and road unevenness conditions. The findings from their study suggests most of the heavy vehicles (length ranging from 16.8 to 36 m) studied could comfortably travel along roads that have a usable lane width of 3.5 m, except the 53.5 m long vehicles (A-Triple and Rigid-plus-Three) which would require about 3.7 m of lane width.

Furthermore, review of geometric design practices for European roads by Brewer et al. (2001) found that high speeds on rural roads are also a safety issue for European countries, which experience a high number of runoff crashes. In Sweden for example, runoff crashes on two-lane roads comprise a third of all crashes on these roads (115 of a total of 339). Moreover, almost all of the runoff crashes occur on two-lane roads (100 of a total 115). Similar percentages were also noted for other countries as well. Head on crashes were the second most common type of crash in most of these countries. To address capacity issues, wider cross sections are used in England, with 5-m travel lanes, which also allow better overtaking for improved flow and is also safer than the standard width. However, Brewer et al. noted that the Swedes have indicated that these wider roads have not had a good safety record. The major contributing factor to the high number of runoff crashes is high speeds, so those countries are focusing on attempts to control and reduce speeds. To achieve this objective, higher speeds are eliminated to preserve safety. This is manifested in the lower design speeds and speed limits for rural roads as well as in efforts to implement the self-explaining, self-enforcing concept on these roadways. With the exception of the Netherlands, all countries visited use a common treatment on high-volume rural roadways, namely the conversion of a two-lane roadway to a 2+1 facility in lieu of four-lane facilities. On such roads the third (middle) lane serves as a passing lane in which the right of way alternates periodically. Each country has customized this design to conform to its design guidelines and safety goals, including use of varied roadway widths, lengths of passing lanes, median cable guardrail, and end treatment of passing lanes. Table 36 shows typical lane width design values for various EAC countries. In order to compare the values of different countries, the road design classes are given as well. As shown in the table, lane widths vary between EAC countries but with reasonable narrow ranges.

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Table 36: Comparison of road and lane width design values

Country	Road design class																			
	II[2], 1[3], Ia[4]				III[2], 2[3], Ib[4]				IV[2], 3[3], II[4]				V[2], 4[3], III[4]				VI[2], 5[3], A Gravel[4]			
	Road way width (m)	Lane width (m)	No. of lanes	Road reserve	Road way width (m)	Lane width (m)	No. of lanes	Road reserve (m)	Road way width (m)	Lane width (m)	No. of lanes	Road reserve (m)	Road way width (m)	Lane width (m)	No. of lanes	Road reserve (m)	Road way width (m)	Lane width (m)	No. of lanes	Road reserve (m)
Burundi[1]	Follows French design standards																			
Kenya[2]	7	3.5	2		6.5	3.25	2		6.0	3.0	2		4.0	4.0	1		4.0	4.0	1	
Tanzania[3]	14	3.5	4	60	7.5	3.75	2	60	7.0	3.5	2	60	6.5	3.25	2	60	6.5	3.25	2	60
Uganda[4]	14.6	3.65	4	60	7.0	3.5	2	60	6.0	3.0	2	50	5.6	2.8	2	50	6.0	3.0	2	40
Rwanda[5]	Follows French design standards																			

In Kenya, depending on the cross section type, the width of the lane is 2.0 m up to 3.5 m while in Tanzania the widest lane width recommended by the design guide is 3.75 m and 3.25 m as the narrowest width for national roads. For low traffic volumes roads the lane width is recommended to be 2.5 m. The selection of lane width is based on traffic volume and vehicle type and speed. In Uganda, lane widths of 2.80 m for Design Class Paved III and Gravel B to 3.65 m for Design Class Paved Ia, are used. When continuous two-way right-turn lanes are provided a lane width of 3.0 to 3.6 m could be provided. For road class Paved Ia a 3.6 m right turn lane is considered from the 5.0 m median width. For access roads with low volumes of traffic design class E roads, single lane operation is adequate with a basic width of 4.0 m.

Burundi and Rwanda follows French guide which observes that there is no legal minimum width for a carriageway. The width must be selected on the basis of the type of vehicles that travel or are expected to travel on the route and their expected speeds. With regard to vehicles, the French Highway Code fixes the maximum width of motor vehicles at 2.60 m excluding wing mirrors: beyond a width of 1.90 m, these may project 20 cm. It further noted that, in practice, the majority of light vehicles do not exceed the following widths:

- Small vehicles: 1.70 m
- People movers: 1.90 m
- 4X4 vehicles: 2.20 m
- Camping-cars: 2.30 m

The French guide observed further that the lateral safety margins must take account of the operating speeds on the route, for which reason widths of between 3.00 and 3.50 m are usually selected for main roads. On the basis of topographical constraints and the level of HGV traffic, lower widths can be used. However, it advises against the systematic marking of the centre line of carriageways with a width of less than 5.20 m (lanes which are compatible with the maximum width of a motor vehicle). For existing roads with a carriageway of between 4 and 6 m in width, it is important to note that operating speeds are highly sensitive to road widths, so any road widening action should take account of the impact with regard to speed increases.

Recommendation

We recommend the following lane widths for EAC:
1) For mobility roads classes 1 to 3, lanes should be 3.5 to 3.75 m wide
2) For access roads classes 4 to 5, lanes should be 2.5 to 3.5 m wide

2.3.5.2 Shoulders

Shoulders are used for emergency stopping, for parking of disabled vehicles, and for lateral support of the subbase, base, and surface courses of the travel roadway. In cases of paved road, lane width covers net width excluding width of edge strip, edge line and centreline markings. Shoulders should be wide enough to adequately fulfil their purpose, but excessive width encourages drivers to use them as an additional travel lane. The lateral clearance outside the paved carriageway provides a safety margin which has to be kept clear of solid obstacles. But the lateral clearance is not only a safety reserve; it is also needed due to the human senses of perception. For the approaching driver, obstacles situated aside the roadway occur under a certain angle. Due to the vehicle speed, this lateral angle increases by a certain rate. Researches have shown that that this rate must be above a certain threshold in order to be perceived by the driver. A stress-free driving is only given when all lateral obstacles are perceived above this threshold. There is no international consensus on appropriate shoulder width.

SATCC recommends that for speeds higher than 60 km/hr a shoulder width of 1.5 m should be regarded as the minimum. It suggests that intermediate traffic volumes and higher operating speeds require a shoulder width greater than 1.0 m and thus it recommends three alternative shoulder widths; 1.5 m, 2.0 m, and 2.5 m. It further recommends a shoulder width of 3.0 m for roads with the highest operating speeds and heavy traffic volumes. Where the traffic situation dictates a dual-carriageway cross-section, the highest standard of shoulder width is called for, namely 3.0 m in the case of the outer shoulder. Only 1.0 m is required for the inner shoulder where it would be possible to move a broken-down vehicle onto the median and thus clear the lane, or where the vehicle would have to be moved across one lane only to reach the safety of the outer shoulder, as would occur on a four-lane divided road. As it is generally conceded that crossing two lanes with a defective vehicle could be very difficult, a six-lane divided road should have inner shoulders 3.0 m wide. The intermediate shoulder widths suggested above would not normally be used for the inner shoulders of a divided road.

AASHTO observed that shoulders width varies from 0.6 m on minor rural roads where there is no surfacing to approximately 3.6 m on major roads where the entire shoulder may be stabilised or paved. Desirably, a vehicle stopped on the shoulder should clear the edge of the roadway by at least 0.3 m, and preferably by 0.6 m. This preference has led to the adoption of 3.0 m as the normal shoulder width that should be provided along high-type facilities. In difficult terrain and on low volume roads, shoulders of this width are being impractical. A minimum shoulder width of 0.6 m should be considered for low-type highway, and a 1.8 to 2.4 m shoulder width is preferable. Heavily travelled, high-speed highways and highways carrying large number of trucks should have usable shoulders of at least 3.0 m width and preferably 3.6 m wide, however, widths greater than 3.0 may encourage unauthorised use of the shoulder as a travel lane. Furthermore, where roadside barriers, walls or other vertical elements are present, it is recommended to provide a graded shoulder wide enough that the vertical elements will be offset a minimum of 0.6 m from the outer edge of the usable shoulder. To provide lateral support for guardrail posts and/or clear space for lateral dynamic deflection of the particular barrier in use, it may be appropriate to provide a graded shoulder that is wider than the shoulder where no vertical elements are present.

Review of geometric design practices for European roads by Brewer et al. (2001) found that a typical cross section of rural roads in some countries is 13 m, with 5.5 m travel lanes and 1.0 m shoulders (Sweden and Denmark) while in England a 9.3 m cross section is used, with 3.65 m travel lanes and 1.0 m shoulders (i.e., 7.3 m carriageway plus 2 x 1.0 m hardstrips). In Germany a 10.5 m cross section is used, with 3.75 m travel lanes and 1.5 m shoulders.

The lateral clearance outside the paved carriageway provides a safety margin which has to be kept clear of solid obstacles. But the lateral clearance is not only a safety reserve; it is also needed due to the human senses of perception. For the approaching driver, obstacles situated aside the roadway occur under a certain angle. Due to the vehicle speed, this lateral angle increases by a certain rate. KOCKS Consult GMBH (2008) reported that research studies have shown that that this rate must be above a certain threshold in order to be perceived by the driver. A stress-free driving is only given when all lateral obstacles are perceived above this threshold. They also noted that the relations between allowed maximum speed and minimum lateral clearance according to the German ‘Guidelines for Cross Sections’ are as shown in Table 37.

Table 37: Relations between maximum speed and minimum lateral clearance

Maximum allowed speed	Minimum lateral clearance
≤ 50 km/h	0.75 m
≤ 70 km/h	1.00 m
> 70 km/h	1.25 m

Source: German 'Guidelines for Cross Sections'

Hall et al. (1995) documented typical shoulder width design values for selected countries as shown in Table 38. They found that almost all countries use narrower shoulders for classifications of roadways with lower design speeds and that several studies have found that wider shoulders produce significant safety benefits. From their work, it can be said that there is no international consensus on appropriate shoulder width.

Table 38: Typical shoulder width design values

Country	Roadway classification		
	Freeway	Arterial	Minor/local
Brazil	3.0 m left 1.0 m right	2.5 m	1.5 to 2.5 m
Canada		1.5 to 3.0 m, rural collector	1.0 m, rural local
China	2.0 to 3.25 m	0.75 to 2.5 m	0.5 to 1.5 m
Czech Republic	1.5 to 2.5 m	0.25 to 1.5 m	
Denmark	3.5 m	2.5 m	1.0 m
France	3.0 m + 0.75 m earth	2.5 m +0.75 m earth	2.5 m +0.75 m earth
Germany	1.5 m (+ up to 2.5 m gravel)	1.5 m (+ up to 2.5 m gravel)	1.0 to 1.5 m
Greece	1.5 m	1.5 to 2.0 m, rural suburban	1.5 m
Hungary	4.0 m	2.0 to 2.5 m	0.75 to 1.5 m
Indonesia	1.5 to 4.0 m	1.0 to 3.0 m	0.25 to 2.5 m
Israel	3.0 m	3.0 m	2.0 to 2.5 m
Japan	> 2.5 m	> 1.75 m	> 0.5 m
Netherlands	1.25 m	0.20 to 0.45 m	0.15 to 0.45 m
Poland	2.5 to 3.0 m	2.0 to 2.75 m	1.0 to 1.5 m
Portugal	3.0 m left 1.0 m right	2.5 m	1.5 to 2.5 m
South Africa	> 2.0 m	Rural 1.0 to 3.0 m; urban not essential	
Spain	0.5 to 1.0 m left 2.5 to 3.0 right	1.5 to 2.5 m	0.5 to 2.0 m
Sweden		0.75 m, rural undivided	
Switzerland	1.0 to 2.5 m	0.5 to 1.5 m	No shoulders
United Kingdom	3.3 m left 1.0 m right D2 only	1.0 m left 1.0 m right	No shoulders
USA	3.0 to 3.6 m right 1.2 to 3.6 median	1.2 to 2.4 m	0.6 to 2.4 m
Venezuela	2.4 to 3.0 m right 0.9 to 1.2 m left	1.8 to 2.4 m	Not specified

Source: Hall et al. (1995)

Table 39 shows typical shoulder widths for various EAC member countries. In order to compare the values of different countries, the road design classes are also given in the table. There is a slight variation in widths between the countries, but almost all widths are within the ranges of widths specified by SATCC guide as well as AASHTO.

Table 39: Comparison of shoulder width design values (m)

Country	Road design class				
	II[2], 1[3], Ia[4]	**III[2], 2[3], Ib[4]**	**IV[2], 3[3], II[4]**	**V[2], 4[3], III[4]**	**VI[2], 5[3], A Gravel[4]**
Burundi[1]	Follows French standards				
Kenya[2]	1.50	1.00	0.50	1.50	0.50
Tanzania[3]	2.50	2.00	2.00	1.50	1.00
Uganda[4]	2.50	2.00	2.00	1.50	2.00
Rwanda[5]	Follows French standards				

Recommendation

We recommend the following shoulder widths for EAC:

d) *For mobility road classes 1 to 3 in flat terrain, minimum width for shoulders should be 2.0 m.*
e) *For roads in rolling and mountainous terrain shoulders should be 1.0 - 1.5 m wide.*
f) *For access roads classes, shoulders should be 0.6 to 1.0 m wide*

2.3.5.3 Normal Cross Fall

A cross slope is used on traffic lanes to promote drainage of surface water. Divided roadways can be treated as two separate roadways, each having a centre crown bordered by slopes to the outside of the pavement, or each direction of travel can be sloped unidirectionally.

AASHTO (2004) provides a range of cross slopes applicable to different surface type. The value of 1.5 to 2% is suggested for high surface type and 2 to 6% for low surface type. Table 40 provides the standard lane cross slopes cited in the international survey done by Hall et al. (1995). As shown in the table, the survey found that there is a relatively high degree of consistency in the most widely used design values for cross slopes. It appears that 2.0 to 2.5% cross slope is the most widely accepted value for design, and that there is little or no difference by roadway class. Hall et al., also noted that in some countries shoulders are sloped at the same rate as the adjacent roadway lane, while in others the shoulder slope can be as much as 2% greater than the adjoining lane. They also noted that when superelevation is employed, AASHTO recommends that all or part of the outside shoulder be sloped upward at the same rate or slightly lower rate than the adjacent lane.

Table 40: Typical lane slope design values

Country	Roadway classification		
	Freeway	Arterial	Minor or local
Australia		2%, 2.5%, 3%	
Brazil	2% concrete 2.5% asphalt	2% concrete 2.5 asphalt	2% concrete 2.5 asphalt
China	1.0 to 2.0%	1.0 to 2.5%	1.0 to 4.0%
France	2.5%	2.5%	2.5%
Germany	2.5 to 7.0%	2.5 to 7.0%	2.5 to 7.0%
Greece	2.5 to 8.0%	2.5 to 8.0%	2.5 to 8.0%
Hungary	2.5%	2.5%	2.5%
Israel	2.0%	2.0%	2.0%
Japan	2.0%	1.5 to 2.0%	1.5 to 2.0%
Poland	2.0%	2.0%	2.0%
Portugal	2.0% concrete 2.5% asphalt	2.0% concrete 2.5% asphalt	2.0% concrete 2.5% asphalt
South Africa	2.0 to 3.0%	2.0 to 3.0% rural 2.0 to 2.5% urban	2.0 to 2.5%
Spain	2.0%	2.0%	2.0 to 3.0%
Sweden		2.5 to 3.0%	
United Kingdom	2.5% (max superelevation 7%)	2.5% (max superelevation 7%)	2.5% (max superelevation 7%)
USA	1.5 to 2.0%	1.5 to 3.0%	1.5 to 6.0%
Venezuela	2%	2%	2% paved, 4% gravel

Source: Hall et al. (1995)

SATCC guide suggests slopes of 2% and a maximum of 3% in areas where heavy rainfall is common or where longitudinal gradient is zero. Unsurfaced shoulders should have a cross fall of 4%, and where shoulders are surfaced the camber should be taken to the edge of the outer shoulder. Kenya manual recommended that for rural roads with bituminous pavements the minimum cross fall should be 2.5% and for rural roads with gravel pavements 4.0%. The shoulder should have the same slope as the carriageway. The Tanzania manual proposed the following normal cross fall for paved carriageway on tangent sections and on very flat curves with larger radii:

- Asphalt concrete surfaces 2.5%
- Surface dressing surfaces 2.5%
- Stone paved surfaces 3.0%
- Gravel and earth surfaces 4.0%

Shoulder slopes should normally be of the same slope as the carriageway.

Uganda manual recommends that the normal cross fall for paved carriageway on tangent sections and on very flat curves with larger radius, should be 2.5% maximum. For areas of intense rainfall the cross fall of 3% may be accepted. On high type two-lane carriageway the crown slope of as low as 2% is accepted for all other conditions. The normal cross fall for unpaved roads should be 4%.

Recommendation

We recommend the following cross fall for EAC roads network:

1. *2.5% for asphalt concrete surfaces (or 3% in areas where heavy rainfall is common)*
2. *Surface dressing surfaces 2.5%*
3. *Stone paved surfaces 3.0%*
4. *Gravel and earth surfaces 4.0%*

2.3.5.4 Side Slope and Back Slope

Side slopes should be designed to ensure roadway stability and provide a reasonable opportunity for recovery for an out-of-control vehicle. Three regions of the side slopes are important to safety: the top of the slope (hinge point), the fore slope, and the toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch). AASHTO (2004) notes that fore slope steeper than 1V:4H are not desirable because their use severely limits the choice of back slopes. Slope 1V:3H or steeper are recommended only where site conditions do not permit use of flatter slopes. When slopes steeper than 1V:3H are used, consideration should be given to the use of a roadside barrier. Back slopes should be 1V:3H or flatter, to accommodate maintenance equipment. Back slopes steeper than 1V:3H should be evaluated with regard to soil stability and traffic safety. Retaining walls should be considered where space restrictions would otherwise result in slopes steeper than 1V:2H. On freeways and other arterials, AASHTO recommends a rate of slope of 1V:6H or flatter be provided on embankments since they can be negotiated by a vehicle with a good chance of recovery. For moderate heights with good roundings, steeper slopes up to about 1V:3H can also be traversable, though not recoverable. Furthermore, AASHTO observes that grasses usually can be readily established on side slopes as steep as 1V:2H in favourable climates and 1V:3H in semiarid climates. With slopes of 2V:3H and steeper, it is difficult to establish turf, even in areas of abundant rainfall.

The recommended maximum rates of side slopes for embankment (fills) and for back slope are the same in the Tanzania and Uganda. In Kenya, the values vary between 1:0.10 to 1:4 depending on the height of cut and the type of materials.

Recommendation

We recommend that

1. *For side slopes steeper than 1V:3H, roadside barrier should be used.*
2. *Back slopes should be flatter than 1V:3H to accommodate maintenance equipment*
3. *Slopes steeper than 1V:3H should be evaluated for slope stability and traffic safety*

2.3.5.5 Clear Zone

The term clear zone is used to designate the unobstructed, relatively flat area provided beyond the edge of the roadway for the recovery of errant vehicles. It includes any shoulders or auxiliary lanes. Clear zone widths are related to speed, volume, embankment slope, and horizontal geometry. The need for clear zones increases with speed and curvature. Clear zone widths vary throughout the world depending on land availability and design policy. For a typical high-speed road the clear zone width varies between 4.0 m (France and South Africa) to 10.0 m (Canada, USA). Recent studies have found that the first 4.0 – 5.0 m provides most of the potential benefits from clear zones.

Jurewicz and Pyta (2010) noted that the existing Austroads design guidance on the selection of clear zones draws heavily on the AASHTO design guide and Transportation Research Board. Generally, selection of clear zones has been based on the probability of a vehicle leaving the road and encroaching into the roadside to ascertain lateral distance from the traffic lane (depth), and on the likely severity outcome in case of a collision with a roadside hazard. The research carried out to estimate this probability was carried out between the mid 1960s and late 1970s in the US and

Canada. The probability of encroachment depth was based on measurements of roadside tire tracks, both in medians and in verges. These studies suffered from the fact that there was no way to differentiate between voluntary and errant roadside encroachments. Presence of hard shoulders confounded the collected data in the first 4 m and the regression relationships had to be extrapolated.

Jurewicz and Pyta (2010) noted that there have been various models developed from these and other similar studies over the years. Figure 7 presents a collection of these models as presented in Jurewicz and Pyta (2010) who cited Austroads as the their reference source for the figure. According to some of these relationships, between 80% and 90% of the encroaching vehicles should stop or recover control within a 9 m clear zone in 100 km/h speed environments. This infers, that after adjusting for exposure (traffic volume, road length and time), the likelihood of a run-off-road casualty crash should be very low if a clear zone wider than 9 m was provided, given relatively straight roads with flat batters.

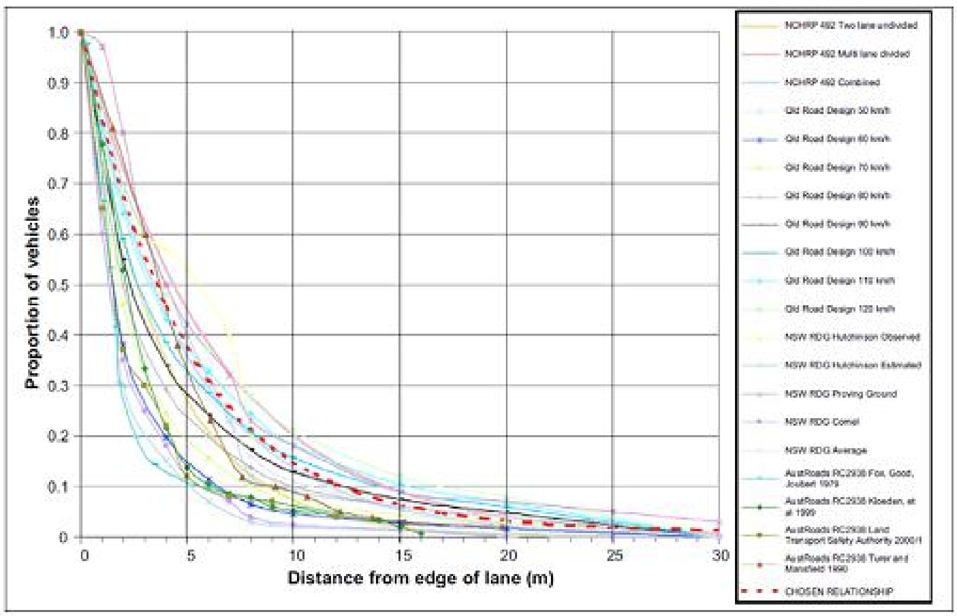


Figure 7: Probability models for lateral depth of encroachment by an errant vehicle (Source: Austroads cited in Jurewicz and Pyta (2010)).

Design guides for Tanzania and Uganda suggested clear zone widths design values in relation to speed limits as shown in the Table 41. The design guide for Kenya reported a clear zone of 3 m or more from the edge of the carriageway. At existing pipe culverts, box culverts and bridges, both manuals proposed clearance not to be less than the roadway width.

Recommendation

We recommend that clear zone width design values that are currently used in relation to speed limits be adopted for EAC. The values are shown in Table 37 and they should be increased at sharp bends on high speed roads by a correction factor.

Table 41: Clear zone widths

Speed limit	Standard	
	Desired (m)	Minimum (m)
70	5	3
80	6	4
100	9	6

2.3.5.6 Median

A median is the portion of a roadway separating opposing directions of the travelled way. Medians are highly desirable on multilane divided arterials carrying four or more lanes. Median width is expressed as the dimension between the edges of travelled way and includes the left/right shoulder (depending on whether driving is on the right/left, respectively), if any. The main functions of a median are to separate opposing traffic, provide recovery area for out-of-control vehicles, provide a stopping are in case of emergencies, allow space for speed changes and storage of left/right-turning (depending on whether driving is on the right/left, respectively) and U-turning vehicles, minimise headlight glare, provide width for future lanes, and in urban areas to offer an open green space, refuge area for pedestrians crossing the street, and control the location of intersection traffic conflicts. It may be depressed, raised, or flush with the travelled way surface.

In determining median with, AASHTO recommends that consideration should be given to the potential need for median barrier and where practical, median widths should be such that a median barrier is not needed. The recommended general range of median widths is from 1.2 to 24 m or more. It is noted that economic factors often limit the median width that can be provided. Cost of construction and maintenance increases as median width increases, but the additional cost may not be significant compared with the total cost of the highway and may be justified in view of the benefits gained. Furthermore, AASHTO suggest that where median widths are about 12 m or wider, drivers have a sense of separation from opposing traffic, thus, a desirable ease and freedom of operation is obtained, the noise and air pressure of opposing traffic is not noticeable, and glare of headlights at nights is greatly reduced. Width of 18 m or more are desirable at rural unsignalised intersections but not in urban and suburban intersections or at intersections that are signalised or may need signalisation in the foreseeable future. A survey conducted by Hall et al. (1995) showed that consensus median width values have not been reached for any category of roadway as shown in Table 42. It can be seen that minimum freeway median width ranges from 1.5 m to over 4.5 m, while for arterials ranges from 1.2 to over 20.0 m.

Table 42: Typical median width values

Country	Roadway classification		
	Freeway	**Arterial**	**Minor/local**
Brazil	2.0 to 6.0 m	2.0 to 6.0 m	2.0 to 6.0 m
China	1.5 to 3.0 m	1.5 to 3.0 m	
Denmark	3.0 m	2.0 m	
France	12 m; 3 m curbed	12 m; 3 m curbed	12 m; 3 m curbed
Germany	3.0 m to 3.5 m	3.0 m to 3.5 m	No median
Greece	3.0 to 3.5 m	3.0 to 3.5 m	
Hungary	3.0 m	1.5 to 3.0 m; 2.5 m curbed	
Indonesia	2.0 to 2.5 m	1.5 to 2.0 m	1.0 m
Israel	3.0 m	2.5 m for 4-lane roadway	
Japan	>4.5 m	>1.75 m	>1.0 m
Netherlands	12.0 m	3.0 to 4.5 m	
Poland	3.5 to 5.0 m	3.5 to 5.0 m	
Portugal	2.0 to 6.0 m, curbed	2.0 to 6.0 m, curbed	2.0 to 6.0 m, curbed
South Africa		9.2 m rural; 1.5 m urban, curbed	None
Spain	10 to 12 m; 3.0 m curbed		
Switzerland	3.5 m; 2.0 m curbed		
United Kingdom	4.0 m	4.0 m rural; 1.8 to 3.0 m urban	
USA	Min 3.0 m	1.2 to over 20 m	
Venezuela	Not defined	Not defined	Not defined

Source: Hall et al. (1995)

In Tanzania, installation of a median barrier is required where the median is less than 9.0 m wide and the speed is high. The minimum width of median is as narrow as 1.2 to 1.8 m. In Uganda a minimum median width is required to allow the provision of right-turning lanes outside of the adjacent carriageway. Four lanes and divided roads are required when the design traffic volume is sufficient to justify their use. The minimum width of median is as narrow as 1.2 to 1.8 m. Where provision for right turn lane is required the minimum width of median will be 4.8 – 5.0 m. In some cases for future upgrading of the road a central reserve of minimum 12.0 m could be introduced to serve as median in Uganda. The recommended median design widths on high-speed rural dual carriageway roads in Tanzania and Uganda are as shown in the Table 43. The manual for Kenya does not cover minimum median width design values. The above values are within the recommended median widths by AASHTO.

Table 43: Median widths

Speed limit	Median Widths			
	Desirable (m)		**Minimum (m)**	
	Tanzania	**Uganda**	**Tanzania**	**Uganda**
100	12	9	6	6
80	9	6	4	4

Recommendation

We recommend median widths shown in Table 44 for speed limits of 80 km/h and 100 km/h.

Table 44: Recommended median widths

Speed limit	Median Widths	
	Desirable (m)	Minimum (m)
100	12	6
80	9	4

2.3.5.7 Service Roads

A service road (also known as access road or frontage roads) serves numerous functions, depending on the type of arterial they serve and the character of the surrounding area. They may be used to control access to the arterial, function as a street facility serving adjoining properties, and maintain circulation of traffic on each side of the arterial. Service roads segregate local traffic from the higher speed through-traffic and intercept driveways of residences and commercial establishments along the highway. Cross connections provide access between the roadway and service roads and are usually located in the vicinity of the crossroads. Thus, the through character of the highway is preserved and unaffected by subsequent development of the roadsides. Service roads are used on all types of highways. Despite the advantages of using service roads on arterials roads, the use of continuous service roads on relatively high-speed arterial roads with intersections may be undesirable.

AASHTO (2004) recommends a minimum spacing of about 50 m between the arterial and the service roads in urban areas. The design of service road is influenced by the type of service it is intended to provide.

Design manuals for Tanzania and Uganda proposed that for the larger trading centres and towns service roads should be provided and typically be of 6.0 m wide while this design aspect is not covered in Kenya design guide.

Recommendation

We propose service roads of 6.0 m wide for larger trading centres and towns, however, lower widths may be adopted depending on the intended function of the service road.

2.3.5.11 Headroom and Lateral Clearance

Headroom is the required height to allow traffic to pass safely under objects restricting the height. AASHTO recommends minimum vertical clearance of 4.4 m and the desirable vertical clearance is 5.0 m. It also recommends, for local and collector roads, minimum clear roadway width of 0.6 m on each side of a bridge for traffic volume of less than 400 veh/day, 1.0 to 1.2 m on each side of a bridge for traffic volume of 400 to 2000 veh/day, clearance of approach roadway width (for bridges over 30 m span, the width is increased by 1 m) for traffic volume of over 2000 veh/day.

The design manual for Kenya recommended headroom under bridges or tunnels structures to be 5.0 m for class A and B roads and 4.5 m for class C, D and E roads and unclassified roads. If in future the road may be raised as the result of maintenance, an additional clearance of 0.1 m may be provided. The minimum headroom over footways and cycle ways is 2.5 m. Design manual for Tanzania suggested a value of 5.5 m as headroom under bridge structures and the minimum requirement to be 5.2 m. The headroom under high-power cables is 7 m and 6 m under low-power cables. The minimum headroom requirement for footways and cycle ways is set to be 2.5 m In Uganda, the recommended headroom under bridge structures is 5.0 m on class A, B and C roads and 4.5 m on lower road classes. The headroom should be 6 m under high-power cables and 5 m under low-power cables. The minimum headroom over footways and cycle ways should be 2.5 m. The minimum lateral clearance recommended by design manuals for Tanzania and Uganda is the same. Since the SADC harmonised guidelines for vehicle dimensions specify the maximum height allowed for vehicles is a double-deck bus not exceeding 4.65 m, the vertical clearance should be above this figure, with some allowance for roadway resurfacing.

Recommendation

We recommend

1) *5.5 m as headroom under bridge structures, 7.0 m as headroom under high-power cables and 6.0 m under low-power cables.*

2) *minimum lateral clearance (i) width of 0.6 m on each side of a bridge for traffic volume of less than 400 veh/day, (ii) width of 1.0 to 1.2 m on each side of a bridge for traffic volume of 400 to 2000 veh/day, and (iii) width of approach roadway width for traffic volume of over 2000 veh/day (and for bridges over 30 m span, the width should be increased by 1 m).*

2.3.5.12 Typical Cross Section Dimensions

Typical cross sections of roadway can be developed using the proposed standards discussed in the previous sections. Table 45 presents typical dimensions of basic cross sectional elements of a roadway, whereas Figure 8 presents typical cross sectional elements of a dual carriageway.

Table 45: Typical cross section dimensions for a road design

Road class	Dimension (m)						Slope (%)	
	Right of way	Roadway width	Carriageway width	Number of lanes	Lane width	Shoulder width	Cross slope	Shoulder slope
Mobility roads class 1	60.0	11.4	7.4	2	3.7	2.0	2.5	4.0
Mobility roads class 3	40.0	10.0	7.0	2	3.5	1.5	2.5	4.0
Access roads class 4	30.0	8.6	6.6	2	3.3	1.0	4.0	4.0

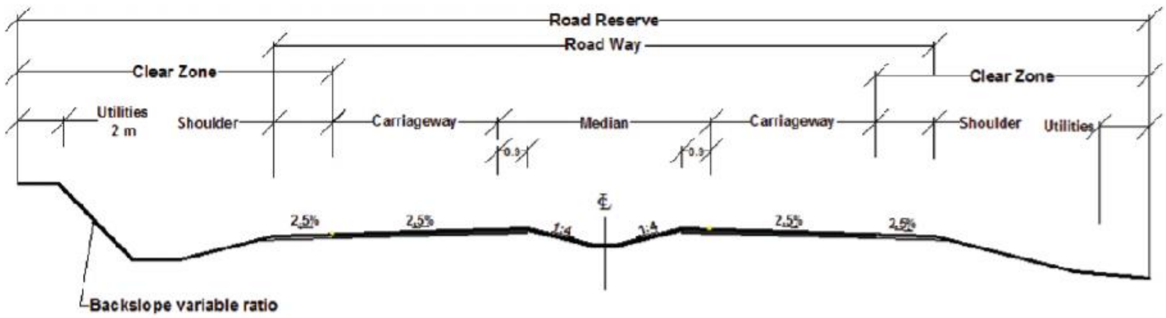


Figure 8: Typical cross sectional elements of a dual carriageway

2.3.5.12 Motorcycle Lanes

There are two types of motorcycle track: restricted track and exclusive track. Restricted motorcycle track can be developed within the carriageway of an existing road. It is usually sited on the left side of the road, preferably between the kerbs and the parking lane. Some form of physical barrier or pavement markings defines the corridor set aside for the cyclists and route markings are necessary to define the route and reduce potential conflicts. However, at crossings and intersections, this kind of cycle track ceases as a separate mode and conflicts may occur with other forms of movement. An exclusive motorcycle track is a complete separate right-of-way established for the sole use of cyclists. This kind of cycle track separates the cyclists from other motorists. This type of exclusive cycle track differs from the restricted cycle track in that it normally has a wide right-of-way and is not developed from existing carriageway of a wide road. It helps to separate conflicts at crossings and intersections with the provision of underpasses and other related facilities (Arahan, 1986).

Warrants for exclusive lanes

In areas, where there is usually a high proportion of motorcyclist, the volume may be so substantial as to affect the smooth flow of traffic. In such instances, the provision of separate and exclusive cycle lanes should be considered. In Malaysia, the general warrants for determining the need for an exclusive cycle lane are (Arahan, 1986):

- the total volume of traffic exceeds the provided lane capacity, and
- the volume of motorcycles exceeds 20% of the total volume of traffic.

Recommendation

Thus, the following warrants are recommended for determining the need for an exclusive cycle lane: the total volume of traffic exceeds the provided lane capacity, and the volume of motorcycles exceeds 20% of the total volume of traffic.

2.3.5.13 Emergency Escape Ramps

The design and construction of an emergency escape ramp at an appropriate location is desirable to provide a location for out-of-control vehicles, particularly trucks, to slow and stop away from the main traffic stream. Out-of-control vehicles are generally the result of a driver losing braking ability either through overheating of the brakes due to mechanical failure or failure to downshift at the appropriate time. Each grade has its own unique characteristics. Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. For existing highways, operational concerns on a downgrade will often be reported by law enforcement officials, truck drivers, or the general public. A field review of a specific grade may reveal damaged guardrail, gouged pavement surfaces, or spilled oil indicating locations where drivers of heavy vehicles had difficulty negotiating a downgrade. For existing facilities, an escape ramp should be provided as soon as a need is established. Crash experience (or, for new facilities, crash experience on similar facilities) and truck operations on the grade combined with engineering judgment are frequently used to determine the need for a truck escape ramp. Often the impact of a potential runaway truck on adjacent activities or population centres will provide sufficient reason to construct an escape ramp.

Unnecessary escape ramps should be avoided. For example, a second escape ramp should not be needed just beyond the curve that created the need for the initial ramp. While there are no universal guidelines available for new and existing facilities, a variety of factors should be considered in selecting the specific site for an escape ramp. Escape ramps generally may be built at any practical location where the main road alignment is tangent. They should be built in advance of horizontal curves that cannot be negotiated safely by an out-of-control vehicle without rolling over and in advance of populated areas. Escape ramps should exit to the right of the roadway. Although crashes involving runaway trucks can occur at various sites along a grade, locations having multiple crashes should be analyzed in detail. Analysis of crash data pertinent to a prospective escape ramp site should include evaluation of the section of highway immediately uphill, including the amount of curvature traversed and distance to and radius of the adjacent curve. An integral part of the evaluation should be the determination of the maximum speed that an out-of-control vehicle could attain at the proposed site. This highest obtainable speed can then be used as the minimum design speed for the ramp.

The principal factor in determining the need for an emergency escape ramp should be the safety of the other traffic on the roadway, the driver of the out-of-control vehicle, and the residents along and at the bottom of the grade. An escape ramp, or ramps if the conditions indicate the need for more than one, should be located wherever grades are of a steepness and length that present a substantial risk of runaway trucks and topographic conditions will permit construction.

Design Considerations

The design and construction of effective escape ramps involve a number of considerations as follows:

- To safely stop an out-of-control vehicle, the length of the ramp should be sufficient to dissipate the kinetic energy of the moving vehicle.
- The alignment of the escape ramp should be tangent or on very flat curvature to minimize the driver’s difficulty in controlling the vehicle.

- The width of the ramp should be adequate to accommodate more than one vehicle because it is not uncommon for two or more vehicles to have need of the escape ramp within a short time. A minimum width of 8 m may be all that is practical in some areas, though greater widths are preferred. Desirably, a width of 9 to 12 m would more adequately accommodate two or more out-of control vehicles. Ramp widths less than indicated above have been used successfully in some locations where it was determined that a wider width was unreasonably costly or not needed. Widths of ramps in use range from 3.6 to 12 m.

To assist in decelerating the vehicle smoothly, the depth of the arrester bed should be tapered from a minimum of 75 mm at the entry point to the full depth of aggregate in the initial 30 to 60 m of the bed.

As a vehicle rolls upgrade, it loses momentum and will eventually stop because of the effect of gravity. To determine the distance needed to bring the vehicle to a stop with consideration of the rolling resistance and gradient resistance, the following simplified equation may be used (61):

$$L = \frac{V^2}{254(R - G)}$$

where:

L = length of arrester bed, m

V = entering velocity, km/h

R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see Table 3-33)

G = percent grade divided by 100

The final speed for one section of the ramp is subtracted from the entering speed to determine a new entering speed for the next section of the ramp and the calculation repeated at each change in grade on the ramp until sufficient length is provided to reduce the speed of the out-of-control vehicle to zero. Figure 9 shows a plan and profile of an emergency escape ramp with typical appurtenances.

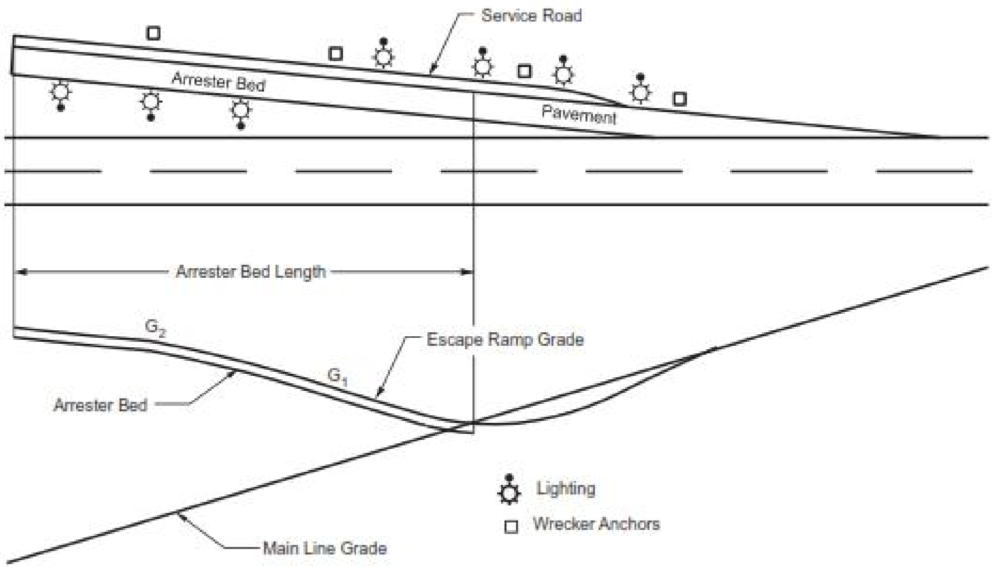


Figure 9: Typical emergency escape ramp

Brake-Check Areas

Turnouts or pull-off areas at the summit of a grade can be used for brake-check areas or mandatory-stop areas to provide an opportunity for a driver to inspect equipment on the vehicle and check that the brakes are not overheated at the beginning of the descent. AASHTO (2011) notes that an elaborate design is not needed for these areas. A brake-check area can be a paved lane behind and separated from the shoulder or a widened shoulder where a truck can stop. Appropriate signing should be used to discourage casual stopping by the public.

Recommendation

*We recommend that a*n escape ramp or ramps, if the conditions indicate the need for more than one, should be located wherever grades are of steepness and length that present a substantial risk of runaway trucks and topographic conditions permit construction

2.3.6 Intersections

An intersection is an important part of a road network because the safety, speed, efficiency, and cost of operation of vehicles on and capacity of the network are greatly influenced by the effectiveness of its intersections. It is defined as an area, shared by two or more roads, whose main function is to provide for the change of route directions. Intersections vary in complexity from a simple intersection, which has only two roads crossing at a right angle to each other, to a more complex intersection, at which three or more roads cross within the same area. There are three categories of intersections at-grade intersections, grade separated without ramps, and grade separated with ramps (interchanges). At-grade intersections do not provide for the flow of traffic at different levels of elevation. Grade separations provide spatial separation between the crossing movements but do not make provision for turning movements. The principal difference between interchanges and other forms of intersection is that, in interchanges, crossing movements are separated in space whereas, in the other forms of intersection, they are separated in time. At-grade intersections accommodate turning movements either within the limitations of the crossing roadway widths or through the application of turning roadways whereas the turning movements at interchanges are accommodated on ramps. The ramps replace the slow turn through an angle of skew by high-speed merging and diverging manoeuvres.

2.3.6.1 The Choice of an Intersection Type

The choice between different types and designs of intersections is normally made with regard to the capacity required, safety, environment and uniformity. Besides, spatial limitations, topographical conditions and construction costs have to be considered. The type of intersection constructed on a road help to make it clear to drivers what type of road they are on. When intersections are too varied or inconsistent they are a source of harmful ambiguity. For example, the construction of interchanges on an ordinary major road generates behaviours downstream which are incompatible with the operating conditions of the road in question (because of facilities such as frontage access and at-grade intersections). Furthermore, drivers on a certain type of road expect certain types of intersections. For example, drivers on an expressway, where there is normally no frontage access and where intersections are grade-separated, would be unprepared for frontage access or an at-grade intersection and fail to react appropriately and rapidly if a non-priority vehicle were to cross the road.

The manner an intersection operates must be compatible with the operating conditions of the type of road on which it is installed. For example, the difficulty of non-priority movements (crossing or left turns) on a standard at-grade intersection is incompatible with the width and speeds that exist on some roads (for example divided highways). It is therefore necessary to ascertain that the selected type of intersection is compatible with the type of road on which it is to be installed. Furthermore, the type of intersection must be compatible with the specific conditions of the site and its operation. Such conditions include traffic volume, closeness to a built-up area, transition between two types of road, and safety problems.

French design guide for intersections provides an example of the choice between two types of at-grade intersections which are compatible with the type of road. Such example shows that the choice of intersection type (from among the possible options) is usually between the two types of at-grade intersections (standard or roundabout). However, the choice of intersection type only takes this form when the other possible solutions (removal of the junction, grade separation with exchange, for example) have been rejected. The choice is based on local conditions at the site. It may be assisted by multi-criteria analysis, taking not only safety into account and other aspects such as user costs, financial analysis and cost benefit appraisal. In the case of a new construction, the decision will frequently be based on general knowledge about how a given type of intersection affects, for instance, the number of accidents and delays. At a site with an existing junction, a safety diagnosis is an essential basis for decision-making. In particular, there is no need to transform a junction into a roundabout when no accidents have occurred there. The safety and journey time benefits depend mainly on the traffic on the major and minor roads, in particular the nature of through and exchange traffic. This must be known in order to make an informed choice. The French guide enumerates a number of main decision criteria for choice of an intersection type including:

- Safety
- Cost
- Delay (long vs medium distance traffic, as well as local traffic)
- Overall consistency between the facilities long the road
- Site characteristics

Generally, the guidelines for junction selection mainly deal with traffic safety. Other important impacts such as capacity or road user costs, environmental issues, investment and maintenance costs are also recommended for consideration. Capacity, delays, queue lengths, road user costs and also exhaust emissions could be estimated using standard software.

Most countries have different ways of appraising the various elements and factors and of setting and applying monetary values to the various costs and benefits, including the costs of fatalities and injuries. This results in different ways of choosing the appropriate intersection type. In many countries, a sample diagram has been developed to indicate the appropriate type of intersection with regard to capacity and traffic safety.

Figure 10 shows an example of the UK approach to selection of appropriate intersection type. Similar approach is also used in the Czech Republic Sanca (2002).

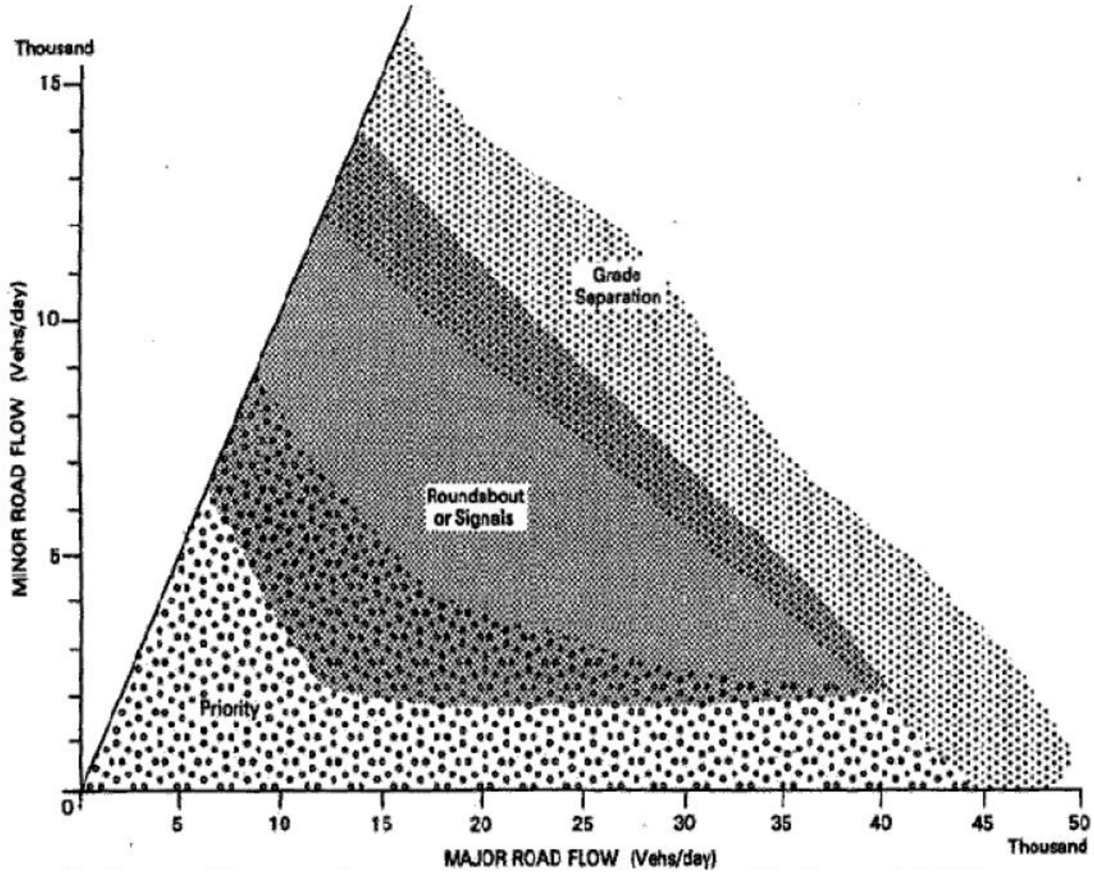


Figure 10: Types of intersections appropriate to different traffic flows, AADT in two directions, an example from the UK (Source: Brüde et al 1998 cited in Sanca (2002)).

In Sweden, there are different diagrams for the safety and the capacity. Figure 11 (B) shows a diagram taking into account the capacity at a three-leg intersection with a speed limit of 50 km/h while and Figure 11 (C) shows a diagram taking into account traffic safety. Figures 11 (D) and 11 (E) show capacity and safety criteria for four-leg intersection with a speed limit of 50 km/h. intersections of category I cover signalised crossing or roundabout, while category II represent priority intersections.

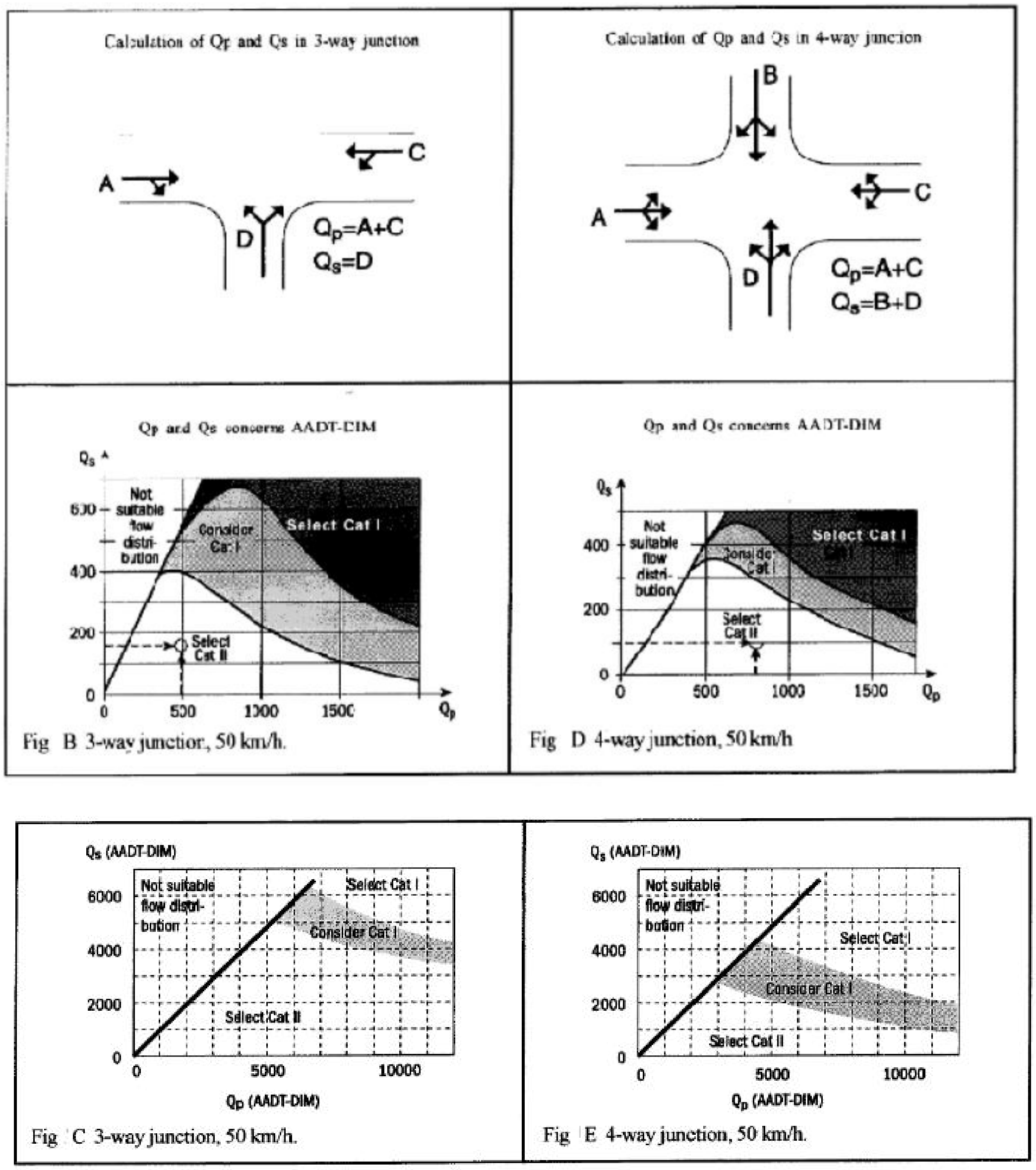


Fig B 3-way junction, 50 km/h.

Fig D 4-way junction, 50 km/h

Fig C 3-way junction, 50 km/h.

Fig E 4-way junction, 50 km/h.

Figure 11 (B, C, D & E): Typical diagrams used to assist intersection selection process, an example from Sweden (Source: Brüde et al 1998 cited in Sanca (2002)).

In France, the choice of intersection type depends on classification of the roads involved as shown in Table 46.

Table 46: Typical matrix of road type and intersection type, an example from France (Source: Brüde et al 1998 cited in Sanca (2002)).

	Motorway A	Motorway B	Urban artery	Distribution road	Local road
Motorway A	motorway interchange	– motorway interchange – road interchange	– road interchange	– partial road interchange – on side roads with give-way or stop sign	
Motorway B		– road interchange – signalised junction – roundabout	– road interchange – signalised junction – roundabout	– signalised junction – give-way or stop sign	– on side roads with give-way or stop sign
Urban artery			– road interchange – signalised junction – roundabout	– signalised junction – roundabout – give-way or stop sign	– on side lane with give-way or stop sign
Distribution road				– signalised junction – roundabout – give-way or stop sign	– signalised junction – give-way or stop sign
Local road					– give-way to traffic from right – give-way or stop sign

In Denmark, the choice of intersection type is based on the desired speed on the major road as shown in Table 47.

Table 47: Relationships between intersection type and desired speed on the major/primary road, an example from Denmark (Source: Brüde et al 1998 cited in Sanca (2002)).

Type of junction	Speed class			
	10–20 km/h	30–40 km/h	50 km/h	60–70 km/h
Signalised		x	x	x
Priority, X-type		(x)	(x)	(x)
Priority, T-type		x	x	x
Raised side-road junction	x	x	x	(x)
Roundabout		x	x	x
Uncontrolled	x	(x)		
Relations marked by (x) are not advisable and should therefore not be used for new construction projects.				

AASHTO (2004) observes that many factors enter into the choice of type of intersection and the extent of design of a given type, but the principal controls are the design-hour traffic volume, the character or composition of traffic, and the design speed. The character of traffic and design speed affects many details of design, but in choosing the type of intersection they are not as significant as the traffic volume. Of particular significance are the actual and relative volumes of traffic involved in various turning and through movements. Local conditions and the cost of right-of-way often influence the type of intersection selected as well as many of the design details. Limited sight distance, for example, may make it desirable to control traffic by yield signs, stop signs, or traffic signals when the traffic densities are less than those ordinarily considered appropriate for such control. The alignment and grade of the intersecting roads and the angle of intersection may make it advisable to channelize or use auxiliary pavement areas, regardless of the traffic densities. In general, traffic service, highway design designation, physical conditions, and cost of right-of-way are considered jointly in choosing the type of intersection.

A review of the existing intersection selection practices in the EAC region showed that Tanzania and Uganda follows a two-steps procedure for selection of at-grade intersection; these are selection of intersection category (priority or control) and selection of intersection type. The selection procedure follows the European traffic safety research results on the relationship between safety, speed and incoming traffic flows on the major and minor road. The selection is based on the following assumptions:

- Priority intersections can be safe and give sufficient capacity for certain traffic volumes and speed limits
- If a priority intersection is not sufficient for safety and capacity, the major road traffic must also be controlled
- Depending on location, traffic conditions and speed limits, different types of priority or control intersection should be selected

Recommendation

All intersections on the EAC road network must be compatible with the operating conditions of the type of road on which it is installed with a view to apply grade separation, if warranted.

2.3.6.2 Basic at-grade intersection elements

The main objectives of intersection design are to ensure effective utilization of the road network and to reduce the severity of potential conflicts between vehicles or between vehicles and pedestrians, while facilitating the necessary manoeuvres. The basic types of at grade intersection are three-leg intersection (T-junction), which generates six vehicle conflict points and ten vehicle-pedestrian conflict points; a four-leg or cross intersection which has twenty-four vehicle conflicts and twenty-four vehicle-pedestrian conflict points; and multi-leg intersection. The purpose of section is to review current intersection design elements and to make recommendations for the same.

The Distance between Intersections

Designers seldom have influence on the spacing of roadways in a network as it is largely predicated by the original or developed land use. Nevertheless, the spacing of intersections impacts significantly on the operation, level of service and capacity of a roadway. For safe and efficient traffic flow, intersections should not be placed too close together. Drivers accelerating away from one intersection are not expecting to encounter traffic slowing for another intersection, for example. Also, a queue of vehicles from one intersection that blocks another intersection, called spill-back, will cause congestion to propagate and cause extra delay. Further, excessively close spacing of intersections frequently has an adverse effect on (i) visibility, (ii) legibility - it becomes difficult for drivers to match their behaviour to the situation and anticipate events on the road, the overabundance of signals lengthens the time taken to perceive and understand messages -, (iii) the placement of traffic signs (regulatory, prohibitory, priority and directional), and (iv) the conditions for overtaking. In general, such changes reduce safety. On the other hand, pedestrians and bicyclists enjoy shorter paths and greater mobility when intersections are more closely spaced.

It follows that intersection spacing should be based on road function and traffic volume. The principles described in the National Guidelines for Road Access Management in South Africa require intersection spacing along signalised arterials to be consistent with the running speed and signal cycle lengths, which are variables in themselves. If the spacing of the intersections is based on acceptable running speeds and cycle lengths, signal progression and an efficient use of the roadway can be achieved. In Figure 12, these three variables are combined in a chart allowing the selection of suitable intersection spacing. From this figure it can be noted that the minimum spacing on arterial roads should be at least 400 m. Where spacing closer than the minimum exist, a number of alternative actions can be considered, and for example two-way flows can be converted to one-way operation or minor connecting roads can be closed or diverted, and channelisation can be used to restrict turning movements. Where the crossing road of an interchange is an arterial, the suggested minimum distance along the arterial from the ramp terminal to the next intersection is 200 m in the case of a collector road. If the next intersection is with an arterial the spacing between the ramp and the intersection should be increased to 600 m for a Class 3 arterial and 800 m for a Class 2 arterial.

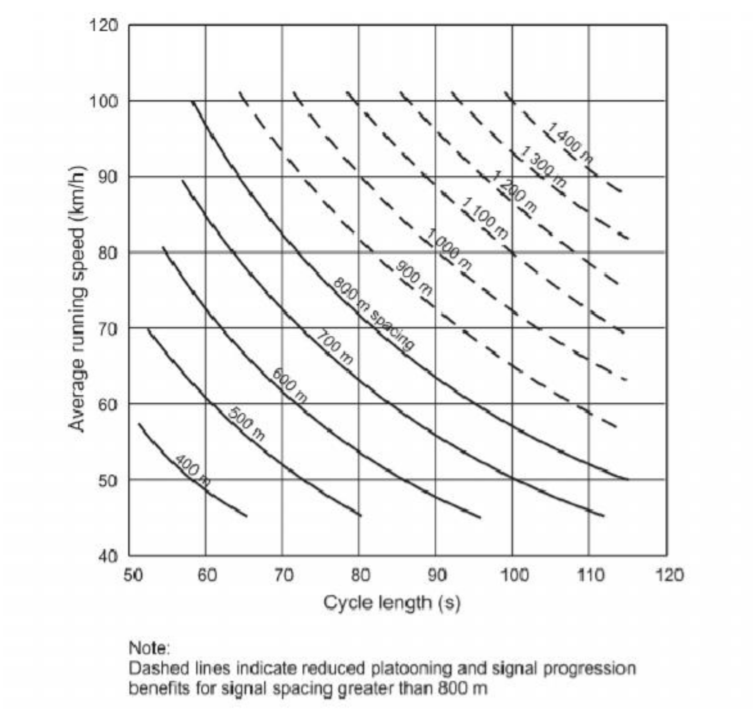


Figure 12: Desirable signal spacing (Source: Tanzania Draft Design Manual, 2010)

A French guide for at-grade intersections observes that there is therefore no universally valid formal rule for the minimum distance between two successive intersections. However, a minimum distance of 250 m can generally be considered to be satisfactory, but specific site characteristics can make this very inadequate. The presence of a centre lane can also lead to a minimum distance being recommended between two successive intersections, in order to allow sufficient opportunity for safe overtaking.

Hummer (2004) observed that intersection spacing of at least 150 m is typically desirable for vehicles and intersection spacing as low as 90 m works well for pedestrians and bicyclists. Spacing between signalized intersections is critical. To ensure optimum progression in both directions on an arterial, signals should be spaced far enough apart that vehicles travel from one signal to the next in one-half the signal cycle length. Hummer pointed out that for typical suburban speeds and cycle lengths, signal spacing around 0.8 km provides for optimum two-way progression and that good two-way progression is often impossible with signal spacing from 150 to 610 m.

The design manuals for Tanzania and Uganda recommends that the minimum distance between consecutive junctions shall preferably be equal to (10 x VD) meters; where VD is the major road design speed in km/h. Where it is impossible to provide this minimum spacing, then the design shall incorporate either, or both, of the following:

- A distance between minor road centrelines equal to the passing sight distance appropriate for the Junction Design Speed plus half the length of the widened major road sections at each junction, or
- A grouping of minor road junctions into pairs to form staggered T-junctions and a distance between pairs as in (i) above.

The manual also recommends that where staggered three-leg intersections are used to replace a crossroads, the right-left stagger as indicated in Figure 13 is preferred to the left-right stagger and the minimum stagger should be 50 m. On traffic grounds this is because in the latter case opposing queues of right turning vehicles from the major road will have to wait side by side with the consequent possibility of the whole junction locking.

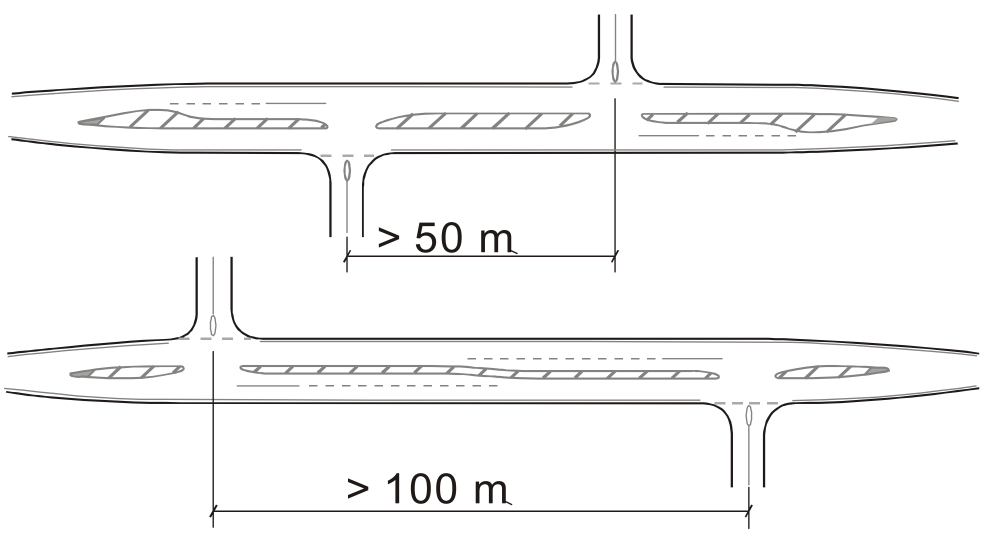


Figure 13: Right/Left and Left/Right staggered intersections and their respective minimum distances (Source: Geometric design manual for Uganda)

In a left/right staggered intersection the minimum distance should be at least 100 meters and should be longer in order to allow the provision of right turn lanes in the major road. The length of right turn lane depends on the junction design speed and traffic turning right in pcu/hr. Where there is a lot of cross traffic (from one minor road to the other) the right/left stagger is preferable. This is because, once the driver has turned into the main road, the driver can proceed to the exit without impeding other traffic. The left/right stagger involves vehicles turning right out of the main road across the path of oncoming traffic, and this is a particularly hazardous manoeuvre.

Furthermore, the design manuals for Tanzania and Uganda indicate that the minimum spacing for signalised intersections is 400 m.

Recommendation

Since proper intersection spacing, particularly for signalized intersections, is critical for providing coordinated signal timing, optimal timing progression for two-way movements should allow travel time between intersections to be about half of the cycle length. We therefore recommend that signal spacing should be at least 400 m in urban areas and 800 m in suburban areas for optimum two-way progression, and for unsignalised intersections the minimum distance should be at least $10*V_D$ *meters.*

Location

Intersections are safer in some locations than others. One important guideline highlighted by Hummer (2004) is that intersections should not be on a horizontal curve if possible. Horizontal curves could restrict sight distances to the intersection and to traffic signals or signs near the intersection. A horizontal curve requires some drivers turning at the intersection to make complex reverse turn manoeuvres. Also, horizontal curves are often superelevated, which makes the profile of the intersecting roads very difficult to negotiate for turning or crossing motorists. Likewise, in relation to the vertical alignment of a road, intersections should not be near a crest vertical curve, again due to restrictions on sight distance. This aspect of location of intersection is not covered by the existing design manuals for EAC Partner States.

Recommendation

We suggest that intersections should neither be on a horizontal curve if possible nor near a crest vertical curve, due to restrictions on sight distance.

Angle

The angle of intersection is important to operations. A right angle intersection provides the most favourable conditions for intersecting and turning traffic movements. Specifically, a right angle (90°) provides (a) the shortest crossing distance for motor vehicles, bicycles, and pedestrians, and (b) sight lines which optimize corner sight distance and the ability of drivers to judge the relative position and speed of approach vehicles. Minor deviations from right angles are generally acceptable provided that the potentially detrimental impact on visibility and turning movements for large trucks can be mitigated. However, large deviations from right angles may decrease visibility, hamper certain turning operations, and will increase the size of the intersection and therefore crossing distances for bicyclists and pedestrians. This makes vehicle and pedestrian time in the area conflicting with other traffic streams increase so delays and collisions increase. In addition, the paved area—and therefore construction and maintenance cost—will increase. Tradition is that these effects increase dramatically with angles less than 60 or greater than 120. Hummer (2004) noted several retrofit improvement strategies exist for treating existing or proposed intersections with unfavorable angles. One (minor leg) or both roads could be realigned with horizontal curves to create a more favorable angle. In some cases, one intersection can be made into two, creating an offset intersection. Another option is to use islands to guide drivers and pedestrians safe manoeuvre through the intersection and reduce the paved area. This element of intersection design is not covered by the existing design manuals for EAC Partner States.

Recommendation

We suggest that angle of existing intersections on EAC road network should be checked for possible implementation of retrofitting improvement strategies, if necessary.

Warrant for Turning Lanes

Turning lanes provide room for left-turning or right-turning vehicles to decelerate before their turns and/ or to queue while waiting to turn. They are particularly effective at reducing delay and collisions by getting those vehicles out of the way of through vehicles. At busy signalized intersections, dual and triple turning lanes are used effectively to reduce the time that those vehicles need the right-of-way. Dual left-turn (left side driving) lanes are also used at some intersections. The drawbacks to using turning lanes include higher right-of-way costs and longer crossing distances for pedestrians.

AASHTO (2004) reports that when designing an intersection, left-turning traffic should be removed from the through lanes, whenever practical. Therefore provision for left turn lanes have widespread application. Ideally, left-turn lanes should be provided at driveways and street intersections along major arterial and collector roads wherever left turns are permitted. In some cases or at certain locations, providing for indirect left turns (e.g., U-turn lanes) may be appropriate to improve safety and preserve capacity. The provision of left-turn lanes has been found to reduce crash rates anywhere from 20 to 65%. Left turn facilities should be established on roadways where traffic volumes are high enough or safety considerations are sufficient to warrant them. They are often needed to ensure adequate service levels for the intersections and the various turning movements. Available guidelines key the need for left turn lanes to

- The number of arterial lanes
- Design and operating speeds
- Left turn volumes
- Opposing traffic volumes

Hummer (2004) noted that over the years many criteria have been published for left-turn and right-turn lanes. The criteria are typically different for unsignalized and signalized intersections. For signalized intersections, one well-known set of turning lane criteria is provided in the Highway Capacity Manual (HCM). The HCM recommends (considering right side driving):

- A single left-turn bay for peak hour left-turn volumes of 100 veh/h or more
- A dual left-turn bay for peak hour left-turn volumes of 300 veh/h or more

- A single right-turn bay for peak hour left-turn volumes of 300 veh/h or more.

The Manual also recommends additional through lanes for each 450 veh/ hr of through volume. Hummer (2004) observed that many agencies use a triple left-turn bay for peak hour left turn volumes of 500 veh/ hr or more and a dual right-turn bay for peak hour right-turn volumes of 600 veh/ hr or more. Table 48 presents proposed guide by AASHTO (2004) to traffic volumes where left turn lanes should be considered on two-lane highways. For the volumes shown, left turns and right turns from the minor street can be equal to, but not greater than, the left turns from the major street. In the case of the double left turn lanes, a capacity analysis of the intersection should be permitted to determine what traffic controls are needed in order for it to function properly.

Table 48: Guide for left-turn lanes on two-lane highways

Opposing volume (veh/h)	Advancing volume (veh/h)			
	5% left turns	10% left turns	20% left turns	30% left turns
60 km/h operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
80 km/h operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
100 km/h operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: AASHTO (2011)

On the other hand, Harwood et al. (2002) undertook economic evaluation for installing left turn lanes at specific intersection types under specific traffic assumptions and crash costs. The assumptions were:

- Major road ADT from 1,000 to 10, 000 veh/day for unsignalised intersections
- Major road ADT from 10,000 to 40,000 veh/day for signalised intersections
- Major road ADT equal to either 10 to 50% of major road ADT for unsignalised intersections
- Major road ADT equal to either 10 to 50% of major road ADT for signalised intersections
- Fatal and injury crashes – US$ 103,000
- Property damage only crashes – US$ 2,300

On major road left turn lane at rural three-leg unsignalised intersection, Harwood et al. found that left turn lane installation would become cost effective for a major road ADT of 4,000 veh/day with 10 percent of the major road volume on the minor road and at 2,000 veh/day with 50% of the major road volume on the minor road. On rural four-leg unsignalised intersection, it was found that left turn lane installation would become cost-effective for a major road ADT of 3,000 veh/day with 10% of the major road volume on the minor road. With a minor road volume equal to 50% of the major road volume, left turn lane installation would be cost effective at all of the major-road volume levels considered. For urban four-leg intersection, left turn lane installation would become cost effective for major road volume ADT of 2,000 veh/day with both 10 and 50% percent of the major road volume on the minor road. On urban four-leg signalised intersection, left turn lane installation was found to be cost effective for all combinations of major and minor road ADT's considered.

The guidelines by Harwood et al. (2002) require higher volumes than those of the HCM or AASHTO in justifying the installation of left-turn lanes based on economic justification of vehicle delay savings and safety benefits.

The design manuals for Tanzania and Uganda suggests that left turn lanes, comprising diverging sections and deceleration sections, should be provided under any of the following conditions:

- On dual carriageway roads.
- When the intersection design speed is 100 km/h or greater and the A.A.D.T. on the major road in design year 10 is greater than 2000 p.c.u.
- When the A.A.D.T. of the left turning traffic in design year 10 is greater than 800 p.c.u.
- Where junctions are sited on left-hand bends and perception of the junction for major road traffic would be greatly improved by its inclusion.
- On four or more lane undivided highways.

A comparison of the above criteria reveals that AASHTO criteria for installing turning lanes are simple and require data which is easily obtainable at site and can be directly applied for site assessment, whereas criteria that are currently used in the Tanzania and Uganda require conversion of daily volumes into annual average daily volumes and information concerning passenger car equivalent units.

Recommendation

We recommend use of AASHTO criteria for provision of turning lanes.

Grades

Steep grades hamper traffic operations at intersections. Steep downgrades on an intersection approach increase stopping distances and make turning more difficult. Steep upgrades on an intersection approach make idling difficult for vehicles with manual transmissions and make acceleration slower for all vehicles, which in turn increases necessary gap sizes and sight distances for crossing and turning movements. Hummer (2004) suggested that grades under 2 percent do not cause many operational problems, grades from 2 to 4 percent begin to introduce noticeable problems, and grades over 4 percent should be avoided where practical. This element of intersection design is not covered by the existing design manuals for EAC Partner States.

Recommendation

We recommend that intersections should be at locations that have grades of less than 4%.

2.3.6.3 Grade Separations and Interchanges

The ability to accommodate high volumes of intersecting traffic safely and efficiently through the arrangement of one or more interconnecting roadways can be achieved by utilizing a grade separation or an interchange system to provide for the movement of traffic between the roadways. By definition, a grade separation represents a crossing of two highways (or a highway and a railroad) at different levels while an interchange represents a system of interconnecting roadways, in conjunction with one or more grade separations, to provide for the movement of traffic between two or more roadways on different levels.

For Interstate highways, interchanges shall be provided between all intersecting Interstate routes, between other selected access controlled highways and at other selected public highways to facilitate distribution of traffic. Each interchange shall provide for all traffic movements. The type and design of grade separations and interchanges are influenced by many factors such as highway classification, character and composition of traffic, design speed and degree of access control. These controls plus signing needs, economics, terrain and right-of-way are of great importance in designing facilities with adequate capacity to safely accommodate traffic demands. Although each interchange presents an individual problem, its design shall be considered in conjunction with adjacent interchanges or grade separations on the project as a whole to provide uniformity and route continuity to avoid confusion in driver expectancy.

Interchanges vary in type from single ramps connecting local streets to complex and comprehensive design layouts involving the intersection of multiple highways. The basic interchange configurations are indicated in the 2004 AASHTO Green Book, Chapter 10, Exhibit 10-1. Their application at a particular location is reflected by surrounding topography and culture, the degree of flexibility in the traffic operations desired and the practical aspects of costs. Any one configuration can vary extensively in shape and scope since numerous combinations of interchange types can evolve through the assembly of one or more of the basic types. An important element of interchange design which influences the efficiency, safety and capacity attained is the size and arrangement of ramps that connect two or more legs at an interchange.

2.3.6.1 Warrants for grade separations and interchanges

The justification of an interchange at a given location is difficult due to the wide variety of site conditions, traffic volume, highway types and interchange layouts. AASHTO (2011) provides six warrants that should be considered when determining if an interchange is justified at a particular site:

- Design designation: The design designation of a roadway is one method used to help decision-makers decide whether or not grade-separated interchanges are justified. Once a controlled access facility has been designated on the major road, an interchange may be warranted for a crossing road based on the anticipated demand for access to the minor road. If a given route has been designated as a freeway corridor, then all conflicting approaches to the freeway must be evaluated individually. In other words, it must be determined whether a given crossroad should be terminated, rerouted, or provided with grade separation. This determination is based on the importance of the crossroad. A minor or local road would typically be terminated, or rerouted, due to their nature of having low traffic volumes. If an arterial or a collector crosses the freeway with significant traffic volumes, a grade separation may be provided. Since this determination is based, in part, on traffic volumes, a method for estimating the amount of traffic that justifies grade separation needs to be evaluated in more detail. The justification for implementing grade separations based solely on the design designation of a roadway is generally applicable in the case of freeway corridors only. If the arterial road has not been designated as a freeway corridor and does not conflict with any existing freeways, this warrant would not be applicable. Of course this does not imply that the designation of the arterial roadway cannot be changed in the future. In fact the alteration of an arterial road to a highflow arterial may merely be a stepping stone in the design evolution from a simple arterial street to a freeway. Thus, an intersection that might warrant only traffic signal, if considered as an isolated case, will warrant a grade separation or interchange when considered as part of a freeway.

- Reduction of bottlenecks or spot congestion: An interchange may be warranted if the congestion at an at-grade intersection is intolerable, and the intersection cannot be redesigned to accommodate the traffic volumes. Intersections that cannot provide sufficient capacity for roadways with significantly large volumes will inevitably experience excessive levels of congestion on one or more of its approaches. Such intersections are typically classified as bottlenecks. A single congested intersection of an arterial network can easily affect nearby intersections or driveways if long queues of vehicles are allowed to spillback. Therefore, it is essential to minimize the delay that results from heavy congestion through the use of at-grade treatments. Surface treatments may include signal optimization, channelization, and pavement re-striping. If a bottleneck cannot be eliminated through simple means, then grade separation may be justified.
- Safety improvement: For intersections where the crash rate is significantly high, grade separation may be justified. This is because more crossing or turning conflicts are encountered at surface intersections, and grade separations remove a significant portion of these vehicle conflicts. A cost-effective analysis may justify the expense of an interchange solely on the basis of safety benefits so that the likelihood of traffic crashes can be significantly decreased.
- Site topography: The topography at some locations may be such that an interchange can be constructed at less than or comparable to the cost of an at-grade intersection. This is due primarily by the design constraints associated with vertical alignment.
- Road-user benefits: An interchange may be warranted where the road user costs such as fuel and oil usage, wear on tires, repairs, delays, crashes that results from speed changes, stops, and waiting are significantly reduced when compared to at-grade intersections. The relation of road-user benefits to the cost of improvement indicates an economic warrant for that improvement. Comparison of benefit cost ratios for design alternatives is an important factor in determining the type and extent of improvement to be made.
- Traffic volumes: This may be the most tangible of any interchange warrant. Although a specific volume of traffic at an intersection cannot be completely rationalized as the warrant for an interchange, it is an important guide, particularly when combined with the traffic distribution pattern and the effect of traffic behavior. However, volumes in excess of the capacity of an at-grade intersection would certainly be a warrant.

Not all warrants for grade separations are included in the warrants for interchanges. Additional warrants for grade separation include grade separation that would be installed at:

- Locations where the termination of local roads and streets is not feasible because of limitations in freeway right-of-way

- Areas that are not accessible by means of frontage roads or other sources of access.
- Eliminate a railroad-highway grade crossing
- Locations with unusual concentrations of pedestrian and/or bicycle traffic for instance, a city park developed on both sides of a major arterial
- Other areas with routine pedestrian and/or bike traffic, especially in school zones
- Mass transit stations that require access because of their location within the confines of a major arterial
- Places with free-flow characteristics of certain ramp configurations and completing the geometry of an interchange.

NCHRP 15-30 reports that a volume warrant for converting a two-way stop controlled rural expressway intersection to a full diamond interchange based on a benefit-cost analysis. This analysis showed that a diamond interchange is generally warranted when expressway volumes exceed 4000 veh/day and minor road volumes exceed 4000 veh/day. Likewise, Lang and Machemehl (1995) carried out an analysis for determining whether or not grade separation is warranted for intersections along urban arterial streets. Their study involved the evaluation of user benefits attributable to operational and design improvements made to arterial intersections. Comparisons were made between grade-separated interchanges (GSI) and at-grade intersections (AGI) in terms of the delay, user travel-time costs, and vehicle operating costs. Overall, the justification for grade separation was found to depend on the user benefits offsetting the interchange construction cost over an assumed design life.

Recommendation

We recommend use of the AASHTO warrants and undertaking of benefit cost analysis for justification for grade separation.

2.3.6.2 Spacing

If an interchange is warranted for any of the listed reasons, interchange spacing is an additional consideration in the decision-making process. Interchange spacing has a pronounced effect on freeway operations. In areas of concentrated urban development, proper spacing usually is difficult to attain because of traffic demand for frequent access. Minimum spacing of arterial interchanges (distance between intersecting roads with ramps) is determined by weaving volumes, ability to sign, signal progression, and lengths of speed-change lanes. AASHTO (2004) gives a general rule of thumb for minimum interchange spacing of 1.5 km in urban areas, and 3.0 km in rural areas (between freeway-to-freeway interchanges and local roads interchanges). In urban areas spacing of less than 1.5 km may be developed by grade-separated ramps or by adding collector-distributor roads.

Recommendation

We recommend a rule of thumb for minimum interchange spacing of 1.5 km in urban areas and 3.0 km in rural areas.

2.3.6.3 Railroad – Highway Crossing

A railroad-highway crossing, like any highway-highway intersection, involves either a separation of grades or a crossing at-grade. The geometrics of a highway and structure that involves the overcrossing or undercrossing of a railroad are substantially the same as those for a highway grade separation without ramps. The horizontal and vertical geometrics of a highway approaching a railroad grade crossing should be constructed in a manner that facilitates drivers' attention to roadway conditions. Some of the safety, operational and emergency issues associated with an at-grade crossing include traffic and emergency response may periodically be disrupted by train traffic. Operational issues with this type of crossing may be worsened as traffic volumes increase over time. Intersections handling a high volume of traffic and pedestrians (and possibly railroads) limit the capacity of the approaching roads. Grade-separating these conflict points allow an uninterrupted flow of traffic while also eliminating the safety threat posed by trains, pedestrians, or other vehicles. Three primary roadway improvement objectives are accomplished using grade separated intersections:

- Increased capacity and uninterrupted flow.
- Increased safety.
- Reduced vehicle-train conflict and delay.

Overpasses increase the capacity of a roadway by allowing uninterrupted flow in all directions. The intersection approaches do not come in direct contact but rather bypass each other. Fewer signals are required to direct traffic, eliminating the queues caused by signals. Grade separation increases roadway safety by reducing the vehicle-vehicle and vehicle-pedestrian conflicts. The crossing traffic is removed from the intersection, thus eliminating the possibility of collisions between those streams of vehicles. Pedestrians are given greater protection from cars, as there will be only one line of traffic to cross and more refuge points can be provided at multiple locations. Removing at-grade intersections with railroads substantially increases speed for both trains and cars. Street traffic moves freely over or under railroad tracks, reducing wait times for a passing train and increasing travel speed and capacity of the roadway. Most importantly, the incidence rate of train-vehicle collisions is eliminated as the intersection no longer puts traffic in front of trains.

We recommend that the decision to grade separate a highway-rail crossing is primarily a matter of economics. Investment in a grade separation structure is long-term and impacts many users. Such decisions should be based on long-term, fully allocated life-cycle costs, including both highway and railroad user costs, rather than on initial construction costs. Such analysis should consider the following:

1) Eliminating train/vehicle collisions (including the resultant property damage and medical costs and liability).
2) Savings in highway-rail grade crossing surface and crossing signal installation and maintenance costs.
3) Driver delay cost savings.
4) Costs associated with providing increased highway storage capacity (to accommodate traffic backed up by a train).
5) Fuel and pollution mitigation cost savings (from idling queued vehicles).
6) Effects of any "spillover" congestion on the rest of the roadway system.
7) Benefits of improved emergency access.
8) Potential for closing one or more additional adjacent crossings.

9)	Possible train derailment costs.

2.3.7 Speed Management

Speed management encompasses a range of measures aimed at balancing safety and efficiency of vehicle speeds on a road network. It aims to reduce the incidence of driving too fast for the prevailing conditions, and to maximize compliance with speed limits. Speed management aims to reduce the number of road traffic crashes and the serious injury and death that can result from them. There are many tools available for effective speed management. They include appropriate speed limits, engineering treatments, effective enforcement of speed limits by police and the use of extensive public information and education programmes to encourage compliance with speed limits. In most cases a mix of tools is required to create solutions that are appropriate to the needs and capacities of the individual country. The main focus of this review is on engineering treatments which cover a range of measures is available to reduce speed in high-risk locations, and to give recommendations for the same.

2.3.7.1 Speed control measures

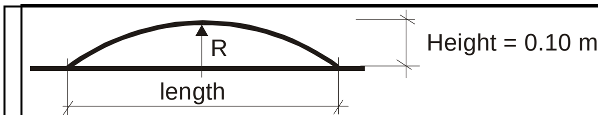
Numerous practices have been implemented in urban areas to reduce speeds:

- Pre-warnings: typically lines on the pavement with (rumble strips) or without punishment (lines and traffic signs)
- Gates: typically different pavement color or structures that indicate transition between traffic environments, often augmented with signs and landscaping
- Narrowings: typically the available roadway width is reduced to narrower lane widths with the addition of islands, by eliminating one lane in two-lane roads or by using wider edge markings
- Humps and tables: with varied profiles including circular, sinusoidal, dome-shaped, or trapezoidal cross-sections and varied lengths depending upon the desired speed reduction
- Raised areas: typically a trapezoidal hump with extended length to allow for longer vehicles to have all wheels on them
- Staggering: typically a lane is shifted over
- Chicanes: typically extensions of the curb at intersections to reduce approach lane widths
- Islands: typically raised elements along the centerline of the roadway to shelter pedestrians and ease street crossing
- Cushions: typically square humps in each travel lane

Speed Humps

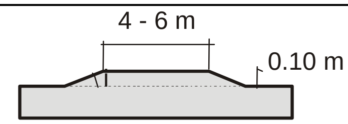
The design manuals for Tanzania and Uganda recommend the use of gates, humps, and rumble strips for speed control. Both manuals give the same design standards for gates, humps (circular and plateau/flat-topped), and rumble strips. The manuals acknowledged the use Danish standards for the design of speed humps. The standards do no differ from the round shaped speed humps design standards (see de Langen and Tembele, 2001) that were tested on Nile road, Nairobi, and in Temeke, Dar es Salaam. The other measure documented by de Langen and Tembele (2001), which proved to be very effective and positive for pedestrian and cyclists, was raised zebra crossings. They eliminated road crashes on the road sections, as safe crossing became possible and bicycle safety improved significantly.

Road bumps are 75 – 100 mm high and 300 – 900 mm long and are typically used in parking lots and on private roads. To pass over road bumps without doing damage to the vehicle or causing discomfort, the driver must slow down almost to a complete stop. They are suitable for residential areas but are not acceptable and should not be used on national roads i.e. Trunk and Regional roads. Road humps are 75 – 100 mm high and 4.0 – 9.5 m long. They may be used on National Trunk Roads and Regional Roads where proven to be absolutely necessary. Road humps should not be used on International Trunk Roads. There are two main types of road humps i.e. circular, which are intended for traffic speed reduction only and flat-topped humps, which are intended for speed reduction and for use as a pedestrian crossing. Figures 14 and 15 illustrate the recommended standards for speed humps.



Vehicle speed(km/h)	Radius (m)	Length (m)
20	11	3.0
25	15	3.5
30	20	4.0
35	31	5.0
40	53	6.5
45	80	8.0
50	113	9.5
55	180	12.0

Figure **14**: Detailed design of circular speed hump



Vehicle speed(km/h)	Grade i (%)
	14
25	12.5
30	10
35	7.5
40	6
45	5
50	4
55	3
60	2.5

Figure 15: Detailed design of flat-topped speed hump

Rumble Strips

Rumble strips are transverse strips across the road used to alert and warn drivers with a vibratory and audible effect before a hazard such as a sharp bend, an intersection or a lower speed limit at the entry to a trading centre. Warning signs are not normally needed when the strips are built to the specifications given below.

However, rumble strips create disturbing noises and can cause vibration problems on soft ground. Thus, it is recommended to avoid installing them near places such as houses, schools, and hospitals. Research in other countries indicates that speed reduction effects tend to be minor and also erode over time. Reliance should therefore not be placed on using rumble strips alone to reduce speeds.

Rumble strips can be used for example in the following situations:

- Before a local speed limit
- At an approach to a dangerous intersection
- Before a sharp bend
- Before a hump.

Tanzania and Uganda design manuals recommend similar standards for the design of rumble strips. The manuals recommend the following principles (Figure 16) to be observed when using rumble strips:

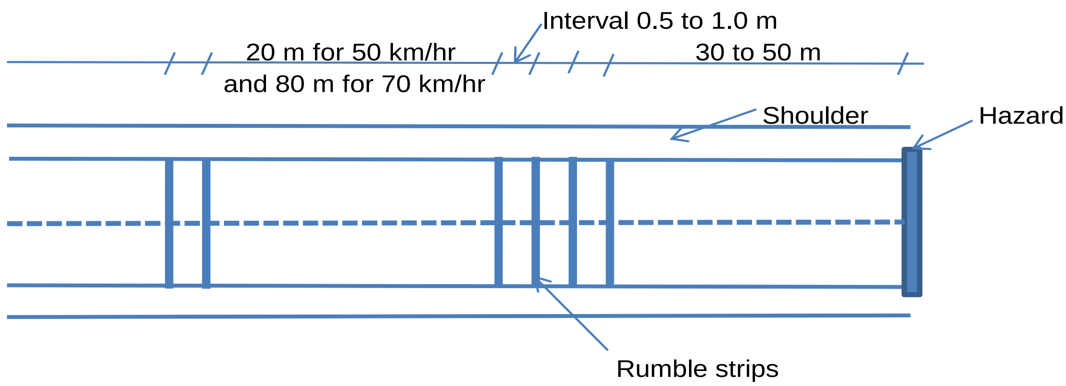


Figure 16: Design of rumble strips

- Rumble strips should normally be in groups of 4 strips
- The height of the strips shall be no more than 10 – 15 mm
- The strip width should be 0.5 m
- One set of rumble strips is usually enough within 50km/h sections
- The last or only strip should be located 30 to 50 m before the hazard
- Pre-warning sets can, if used, be located 20 to 80 m before the hazard depending on speeds
- Rumble strips should preferably have yellow thermoplastic lines across the top for better visibility
- Strips should continue across the full width of the carriageway, including the shoulders but be terminated so that they do not interfere with drainage.

Recommendation

We recommend that other speed control measures other than the speed humps and rumble strips should be tried on major roads to reduce vehicle speeds. Such measures may include narrowings, raised zebra crossings, and use of combination of measures.

Also, we recommend speed humps dimensions shown in Figures 14 and 15 and rumble strips design shown in Figure 16.

2.3.8 Pedestrian and Cyclists Facilities/ NMT Facilities

2.3.8.1 Pedestrians facilities

Pedestrian facilities include sidewalks, crosswalks, traffic control features, bus stops or other loading areas, sidewalk on grade separations, and the stairs, escalators, or elevators related to these facilities, and curb cuts (depressed curbs and ramped sidewalks) and ramps for older walkers and persons with mobility impairments.

In the past the needs of pedestrians were largely ignored, and this may be one reason why so many pedestrians are killed and injured on our roads. Pedestrians have as much right to use the road as motorists, and roads must be designed with their needs in mind. The first step is to identify major pedestrian generators (markets, shops, schools, etc.) and determine which the most important pedestrian routes are. The aim should be to develop a network of pedestrian routes and crossing facilities that is convenient to use and avoids conflicts with vehicular traffic.

Shoulders and footways

Both manuals for Tanzania and Uganda recommended a shoulder of 1.5 m wide for pedestrian in rural areas as pedestrian can walk on the road shoulders. The conventional view is that pedestrians in rural areas can walk on the road shoulders. The shoulder should be at least 1.5 m wide, though 1 m is just acceptable if there are constraints. The surface must be well drained and be as smooth as the traffic lanes – if not, pedestrians may prefer to walk in the traffic lane. The implication of this is that low-cost chip seal shoulders may not be a good investment. Letting pedestrians use the shoulders is not entirely satisfactory, as there is nothing to protect the pedestrian from speeding traffic. This is of particular concern on high-speed and / or high volume roads. In these situations it is preferable to provide a separate footway several metres beyond the edge of the shoulder – and separated from it by a grass strip. Some criteria for the provision of footways are given in Table 49 but these should be used with caution – in some circumstances footways can be justified at lower pedestrian flows.

Table 49: Criteria for provision of footways

Location of footway	Average daily vehicle traffic	Pedestrian flow per day	
		Speed limit of 60 – 80 km/h	Speed limit of 80 – 100km/h
One side only	400 to 1,400	300	200
	> 1,400	200	120
Both sides	700 to 1,400	1,000	600
	> 1,400	600	400

Standard footway widths are:
Absolute minimum: 1 m (two persons cannot pass each other)
Desirable minimum: 1.8 m (two persons can pass each other closely)
Light volume: 2.25 m (two persons can pass each other comfortably)
Heavy volume: 3.5 m+ (space for three persons)

In urban areas, however, the manual for Tanzania recommends a 150-200 mm high barrier kerbs and that higher kerbs of 250 mm can also be used in order to deter vehicles from parking on the footway (but are not recommended for general use) while the manual for Uganda recommends a barrier kerbs of 100-150 mm high with higher kerbs of 200 mm. AASHTO suggests use of kerbs of 100 – 200 mm high.

Pedestrian bridges and underpasses

The manual for Tanzania recommends dimensions for underpasses which are different from those documented by the manual for Uganda. Tanzania recommends a minimum height of 2.5 m and 3.0 m for short (< 15 m) and long (> 15 m) underpasses, respectively, while Uganda recommends heights of 2.3 m and 2.6 m for the respective underpasses. Moreover, Tanzania recommends a vertical clearance of 5.5 m (and absolute minimum of 5.2 m) for pedestrian bridge above the carriageway surface while Uganda recommends a clearance of 5.0 m. The recommended flights of stairs (between landings) is limited to 12 steps for the case of Tanzania while for Uganda the number of steps for flights of stairs (between landings) are limited to 20 steps (9 steps where there are significant numbers of disabled persons).

Recommendation

We propose adoption of

1) 100 – 200 mm as height of barrier kerbs to deter vehicle from encroaching sidewalks
3) 5.5 as headroom under pedestrian bridges

2.3.8.2 Cycle facilities

The conventional view is that cyclists in rural areas can use the shoulders, and this is acceptable provided that the combined volume of pedestrians and cyclists is low (<400 per day) and the shoulder is at least 1.5 m wide. But, with heavier flows, and especially if there is high-speed traffic and / or a high proportion of heavy goods vehicles, it will be better to provide a separate cycleway or a combined cycleway and footway. Figure 17 shows the basic dimensions.

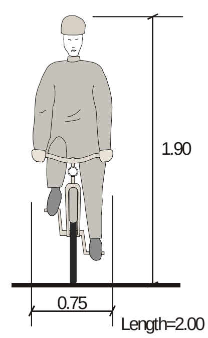


Figure 17: Cyclist dimensions

The design manuals for Tanzania and Uganda recommend similar dimensions for widths for cycle facilities (Table 50) and clearances (Table 51) to wall, fence, barrier or other fixed object. They specify minimum clearance to edge of traffic lane for various speed limits. It is also recommended to provide separate cycleway or combined cycleway and footway if there is high-traffic and the combined flow of pedestrians and cyclists is more than 400 per day. Cycleways need to have a smooth surface with good skid resistance.

Recommendation

We propose recommend widths shown in Table 50 and clearance shown in Table 51 for cycle facilities

Table 50: Recommended widths for cycle facilities

Type	Minimum width (m)	Standard width (m)	Width for heavy usage (m)
Cycleway (separate from carriageway)	2.0	2.5	3.5
Combined cycleway and footway	2.0	3.0	4.5
Cycle lane (one way)	1.5	2.0	2.5

Table 51: Recommended clearances for cycle facilities

Type	Recommended clearance [m]
Minimum overhead clearance	0.50
Clearance to wall, fence, barrier or other fixed object	0.50
Clearance to unfenced drop-off, e.g. embankment, river, wall	1.0
Minimum clearance to edge of traffic lane for speed limit of:	
50 km/h	0.50
80 km/h	1.00
100 km/h	1.50

2.3.9 Traffic Control Devices and Other Facilities

Road furniture and safety represents a collection of marginal elements intended to improve the driver's perception and comprehension of the continually changing appearance of the road. Such elements include traffic signs, road markings, traffic signals (these are covered in Annex A5), marker posts, kilometre posts and road reserve marker posts.

Marker Posts

These are intended to make drivers aware of potential hazards. The draft design manual for Tanzania recommends the marker posts to be sited at 0.25 m outside the edge of the shoulder.

Kilometre Posts

These are blocks or pillars of concrete set up beside major roads to show distances from that point to town centres or major settlements along the road. They show the distances to the destination and that of the origin placed in such a way that the road user will only immediately see the distance to the destination. The design manual for Tanzania recommend kilometre posts is installed along the whole road at an interval of distance of 5 kilometres from each other. Also the posts should be placed in stagger thus forming a 10 km interval on each side of the road.

Road Reserve Marker Posts

Numerous unauthorised accesses tend to develop within the road reserve area. This unwanted development can be limited by providing proper demarcation of the boarder of the road reserve. Road reserve marker posts are recommended by the draft design manual for Tanzania. The posts are recommended to be erected on both sides of the road at intervals of 100 m from each other when traversing inhabited areas and at 300 m on other areas. Whenever new villages are formed along the roads, additional road reserve marker posts should be erected to meet the 100 m interval.

Recommendation

We recommend that

1) *Marker posts should be sited at 0.25 m outside the edge of the shoulder*
2) *Kilometre posts should be installed along the whole road at an interval of distance of 5 kilometres from each other and they be placed in stagger thus forming a 10 km interval on each side of the road*
3) *Road reserve marker posts should be erected on both sides of the road at intervals of 100 m from each other when traversing inhabited areas and at 300 m on other areas. Whenever new villages are formed along the roads, additional road reserve marker posts should be erected to meet the 100 m interval.*

Rest Areas

The draft manual for Tanzania covers requirements for provision of rest areas. It categories three types of rest areas: major, minor and truck parking bays. Having taken these requirements into account, we recommend that the following requirements should be taken into account when planning for rest areas.

m) Planning for Rest Areas

A detailed Rest Area Strategy Plan should be developed for all mobility roads and significant freight routes. Three categories of rest areas should be reflected in the Rest Area Strategy Plans developed by road agencies for all major highways and significant freight routes:

- Major Rest Areas: These areas are designed for long rest breaks, offering a range of facilities and separate parking areas for heavy and light vehicles. These are designed to allow drivers to take rest and sleep breaks required under current driving hours regulations.
- Minor Rest Areas: These areas are designed for shorter rest breaks, and at a minimum should provide sufficient parking space for both heavy and light vehicles. While it is not anticipated that these stops will be used for long rest breaks/sleep opportunities, separate parking areas for heavy and light vehicles may be required at some locations.
- Truck Parking Bays: These areas are primarily designed to allow drivers of heavy vehicles to conduct short, purpose-based stops including load checks, completing logbooks and addressing associated operational needs.

On a given highway or freight route, a mix of the three rest area categories should be provided. In the development of new rest areas, the category and size of a selected rest area should flow from the Rest Area Strategy Plan developed for the route. Selection should be based on the mix and volume of traffic, the perceived demand for the rest area and the availability of existing rest opportunities, commercial service centres or nearby towns.

n) Spacing intervals

Intervals between rest areas should depend on the category of rest area selected, the volume and mix of traffic and the demand for parking and rest opportunities identified in the Rest Area Strategy Plan for a given highway or route. However, as a general rule: Major Rest Areas (Figure 18) should be located at maximum intervals of 100 – 120 km, Minor Rest Areas (Figure 19) should be located at maximum intervals of 50 – 60 km, and truck Parking Bays should be located at maximum intervals of 30 – 40 km.

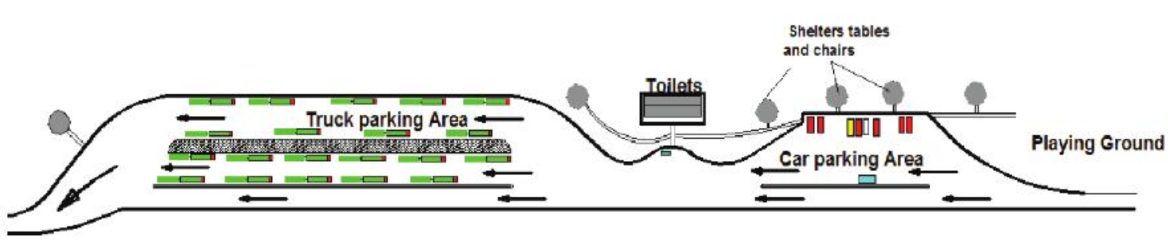


Figure 18: Typical layout design with truck and car parking of major rest areas –front to rear parking design (Source: Tanzania Draft Design Manual, 2010)

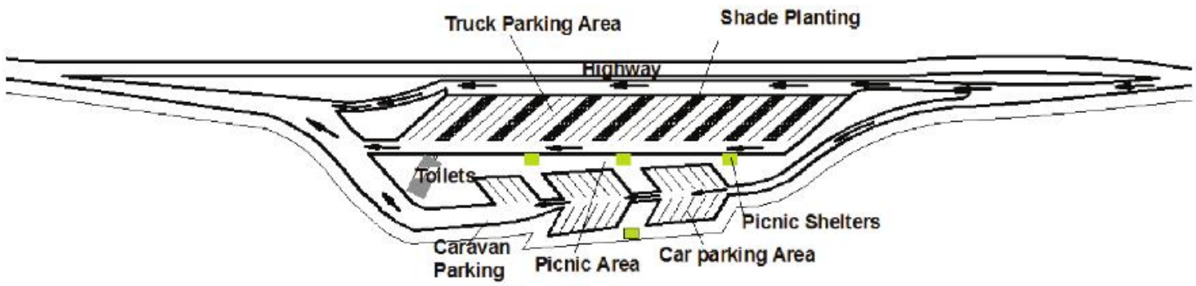


Figure 19: Typical layout design with truck and car parking of minor rest areas –angle parking design (Source: Tanzania Draft Design Manual, 2010)

Differences in traffic volume and rest area demand will impact on the spacing of rest areas, and the number and size of parking spaces provided. Spacing may need to be reduced on highly trafficked routes where demand is evident.

On mobility roads it is recommended that Rest Areas, and especially commercial service centres, are located at mid-block locations on both carriageways, offering equal stopping opportunities in advance of sites servicing opposing traffic. Where duplicate rest areas are not available (due to insufficient funding or demand), facilities should be staggered in the direction of approaching traffic to discourage cross median vehicular movements and to deter drivers from parking on shoulders and walking across the carriageway to access facilities. On major, undivided highways it is recommended that Rest Areas and Truck Parking Bays are provided on both sides of the road. This is particularly important in order to eliminate the need for heavy vehicles to turn across oncoming traffic to enter a rest area on the opposite side of the road.

o) Proximity to a town

To the extent possible, rest areas should be located to promote the use of town facilities (including toilet and shower amenities and the purchase of food and fuel), where they are provided and accessible on a given route. Where the traffic volume and demand warrants, consideration should be given to providing a Truck Parking Bay within 20km to 30km of a township to allow drivers the opportunity to take a rest break and check vehicle loads.

p) Location

In planning the location of new rest areas and the upgrading of existing rest areas, the status and physical characteristics of the environment of a potential or existing site must be examined. Issues associated with topography, landmarks or scenic viewpoints and environmental qualities should be considered. The location of watercourses and utilities, the proximity to major road interchanges and the need for additional land should also be considered. Whilst it is most desirable to comply with the siting requirements summarised above, there are many other features that influence the location of rest areas. It must be practicable to provide a rest area at a particular site in relation to factors such as topography, availability of utilities, environmental considerations, other road infrastructure, and traffic operations and safety.

q) Proposed Layout

The primary goals of rest area layout design is to provide suitable facilities in an environment that promotes effective and safe rest and/or sleep opportunities, and to ensure that there is adequate provision for vehicles and pedestrians to move safely within the site. The number of spaces provided at a given rest area site should be based on traffic volume and expected demand. As a general rule major rest areas should provide sufficient parking space for at least 20 vehicles, minor rest areas should provide parking for up to 10 vehicles, and truck parking bays should provide sufficient area to accommodate at least four to five heavy vehicles at any one time.

r) Pedestrian Access and Visibility

It is recommended that rest areas and service centres be designed to ensure that potential conflict between vehicles and pedestrians is minimised, and that any necessary interaction occurs at a very low speed.

s) Speed

The design speed for vehicle travel within a rest area should be considered in conjunction with the type of rest area, the location within the area and environmental conditions.

t) Access

When designing rest area layouts, consideration should be given to the provision of safe and effective access to the facility required for different standards of roads. Features including acceleration and deceleration lanes, entrance and exit ramps and slip lanes need to be designed.

u) Minimum Signage Requirements

As a minimum, signage for Rest Areas and Truck Parking Bays should be provided in accordance with the traffic signs proposed in Annex A 5.

v) Minimum Facilities

The minimum facilities that should be provided at major and minor rest areas are:

- All weather pavements, with sealed pavements for access and egress roads/ramps.
- Shade.
- Rubbish bins.
- Separate parking for heavy and light vehicles.
- Sheltered areas.
- Tables and/or benches.

Truck parking bays should include, at a minimum, rubbish bins, shade and all weather pavements. If possible, sheltered areas and tables and benches should be provided. Many heavy vehicle drivers report that rest areas are primarily used for sleeping and rest opportunities. Rest area selection is based on the provision of natural shade, rubbish bins, all weather surfaces and sealed pavements, and the quality and cleanliness of toilets and running water provided.

Provision for People with Disabilities should also be taken into account when design rest Area. This also includes access to and the use of rest area facilities, provision of suitable toilets, parking spaces, and appropriately graded and treated footways and ramps.

w) Additional Facilities

Additional facilities such as toilets and drinking water should be provided in rest areas where they can be adequately justified based on estimated use and the cost of maintenance and cleaning. The provision of additional facilities, such as toilets, showers and drinking water, should be considered on a site by site basis, taking into consideration the availability of required utilities, the installation requirements and the costs associated with ongoing facility maintenance. Drivers consistently report that the quality and cleanliness of rest area facilities, particularly toilets, play a large role in rest area selection. Therefore a plan for the ongoing maintenance and upkeep of these facilities should be documented and implemented.

x) Cross-Border Compatibility

It is recommended that State and Territory road authorities adopt common guidelines for the provision of rest areas and service centres, and coordinate their provision across state and territory borders, so that a consistent level of service is provided along important routes.

In order to maximise the benefits of rest areas and service centres it is imperative that the provision of these facilities be closely coordinated across state and territory borders. Duplication of such facilities may lead to under-utilisation of one or both facilities and adversely affect their viability.

2.4 Impact of Harmonisation

Implementation of the above recommendations will ensure uniform roadway geometric standards and hence the optimum balance between road infrastructure construction cost and road user cost is obtained, considering road safety issues and natural and human environmental aspects in the EAC Region. Other impacts of the above harmonised regimes include improved efficiency of the road transport system by minimisation of road crashes and energy consumption. It should be noted that this study also revealed that the width, height, wheelbase and minimum turning radius dimensions of vehicles such as the 22 m Interlinks, which have recently been permitted to traverse EAC road network, conform to the respective dimensions of the design vehicles documented in the existing design manuals for EAC member countries and therefore will be accommodated by the existing intersections on the EAC road network. On the flip side, all countries will need to check whether headroom under bridge structures complies with the recommended value of 5.5 m and make improvements, if necessary. Additionally, it is proposed that the recommended standards be implemented in all new road designs, and road rehabilitation, reconstruction and widening projects.

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FINAL REPORT

ANNEX A 3

HARMONISATION OF PAVEMENT AND BRIDGE DESIGN STANDARDS

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GLOSSARY AND ACRONYMS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transport Officials
AC	Asphalt Concrete
AFNOR	Association Française de Normalisation
ALD	Average Least Dimension
BEMIX	Bitumen emulsion mix
BICO	Bureau for Industrial Cooperation
CBR	California Bearing Ratio
CC	Cement Concrete
COV	Coefficient of Variation
CRCP	Continuously Reinforced Concrete Pavements
CRR	Crushed Rock
CRS	Crushed Stone
DBM	Hot Mix of Bitumen and Aggregate
DR	Damp rock
EAC	East African Community
EICM	Enhanced Integrated Climatic Model
ESAL	Equivalent Standard Axle Load
FBMI	Foamed Bitumen Mix
FHWA	Federal Highway Administration
G	Soil/ Natural Gravel Material
GW	Gravel Wearing
HDM	Highway Design and Management
HGV	Heavy Goods Vehicle
HMA	Hot Mix Asphalt
JPCP	Jointed Plain Concrete Pavements
JRCP	Jointed Reinforced Concrete Pavements
LAMPS	Large Aggregate Mix for Base Course
LRFD	Load and Resistance Factor Design
MC	Medium Curing
MGV	Medium Goods Vehicle
NCHRP	National Cooperative Highway Research Program
OMC	Optimum Moisture Content
PCA	Portland Cement Association
PCP	Prestressed concrete pavements
PM	Penetration Macadam
RC	Rapid Curing
S	Seal

SADC	Southern African Development Community
SAMDM	South African Mechanistic Design Method
SATCC	Southern Africa Transport and Communications Commission
STD	Standard Deviation
TLC	Traffic Load Classes
UK	United Kingdom
VHGV	Very Heavy Goods Vehicle
WIM	Weigh-in-Motion

EXECUTIVE SUMMARY

The main focus of pavement design is to recommend a suitable pavement structure that will meet functional and structural performance criteria through the service life of the pavement. The design philosophy involves not only a pavement design strategy that seeks to identify the best initial structural section, but also the best combination of materials, construction technologies, and maintenance and rehabilitation strategies. Therefore the basic objective of pavement design is to provide structural alternatives that are feasible both technically and economically. This is achieved by specifying appropriate pavement and surfacing materials to ensure that the pavement performs adequately and requires minimal maintenance under the anticipated traffic loading for the design period adopted. This selection process involves adoption of material types, thicknesses and configurations of the pavement layers to meet the design objectives. When designing or selecting pavements, there are four fundamental external design parameters to consider: the environment, design traffic, the characteristic of the subgrade upon which the pavement is placed, and pavement and surfacing Materials.

Pavement Design Standards Adopted by EAC Partner States

Burundi and Rwanda have been following French standards for pavement design while Kenya, Tanzania and Uganda are using their own design standards which are largely empirical-based methods. A review of the design standards that are used by EAC Partner States revealed a number of areas which are common and unique to particular EAC Partner State as well as areas which need further improvement. French design standards, which are followed by Burundi and Rwanda, combine the useful aspects of mechanics of materials and lessons gained from experimentation. They employ mechanistic pavement models to calculate stresses and strains, and utilises laboratory-based fatigue damage test results to define pavement failure. Knowledge derived from observation of the behaviour of real pavement is used to specify criteria to check subgrade strains values of the unbound layer and to tie in with the results of the mechanistic computation model. They therefore specify materials strengths in terms of resilient modulus values (MPa).

The design manuals have particular reference to the prevailing conditions in the respective countries and reflect EAC member countries' experience gained in road pavement design during the last 40 years. For Tanzania and Uganda, the manuals supersede the previous design guides of 1989 and 1994, respectively. Generally, revision of a design manual is only possible when new technical information and performance data become available.

Potential Areas for Harmonisation and Improvement

Flexible Pavement Design Standards

Environment: The environmental factors that significantly affect pavement performance are moisture and temperature. Both of these factors must be considered at the design stage of the pavement and need to be harmonised across the region. For the purpose of pavement design some of the Partner States divide their countries into climatic zones. The study, however, found variations in the way the climatic zones are defined and sub-divided.

We recommend a harmonised definition and division of climatic zones into three; wet, moderate and dry zones. A dry zone is as an area in which the number of months per year with higher rainfall than evaporation is less than 1 month. A moderate zone is as an area in which the number of months per year with higher rainfall than evaporation is 1 to 3 months, whereas a wet zone is as an area in which there are more than 3 months per year with higher rainfall than evaporation. In addition we recommend the use of performance graded asphalt specification which corresponds to the maximum and minimum pavement temperatures of the climatic zone similar to what is found in Super-pave mix design.

Drainage: A review of the design manuals in the EAC region indicates that manuals for Tanzania and Uganda provide guidance on the cover the issue of drainage. Drainage systems can be categorised as surface drainage or subsurface drainage system. Surface drainage encompasses all means by which surface water is removed from the pavement and right of way of the road and it includes adequate transverse and longitudinal slopes on both the pavement and shoulder to ensure positive run-off, longitudinal channels (ditches), culverts, and bridges to provide for the discharge of the surface water to the natural waterways. Subsurface drainage system aims to keep the variation of moisture in the subgrade and pavement layers to a minimum at the same time reducing the amount of the same in the subgrade soil and pavement.

Traffic: One of the most important factors that affect Pavement design is traffic loading through the weight of the vehicle that is imposed on the pavement. Current practices in the partner states vary and therefore there is need for harmonisation as indicated in the table below.

Factor	Practice in EAC Partner States and Recommendations
Design period	A review of the design periods for new pavements in the EAC Partner States revealed that design period varies from 10 to over 20 years. Kenya and Uganda allow stage construction while Tanzania does not and Burundi and Rwanda uses a minimum design period of 20 years. We are recommending a design period of 20 years for the design of new pavement or reconstruction of those that have reached the end of their design life. The main reason for this recommendation is that traffic forecasting beyond 20 years increases uncertainty in the forecast. With economic justification, design periods other than 20 years can be adopted on a case by case basis.
Heavy Vehicle Classification	A review of the design manuals found that there are variations in the definition of a Heavy Goods vehicle to be used for pavement design as well as in design their categories. We are recommending that a heavy goods vehicle be defined as vehicle having a registered un-laden weight of 3 tonnes. We are further recommending that EAC should consider adopting heavy vehicle classifications of medium goods, heavy goods, very heavy goods, and buses having seating capacity of more than 24 including the driver. Where a Medium Goods Vehicle (MGV) covers small and medium sized trucks including tankers up to 7 tons load, Heavy Goods Vehicle (HGV) covers Trucks above 7 tons load, and Very Heavy Goods Vehicles (VHGV) are articulated trucks with trailer or semi-trailer and tanker trailers.
Determination of Design Traffic Loading	In determining the relative damage of the various axle loads there are variations in the exponent used across the Partner States. We recommend that a value of n = 4.5 be used as it represents the worst case scenario, for pavement design classes of up to 30 million ESA above which analytical methods of design should be adopted.

Subgrade: The subgrade is the foundation of the pavement structure whose strength has a marked effect on the pavement thickness and performance. A number of factors influencing subgrade strength were reviewed, compared and contrasted among EAC Partner States. The following table summarises our findings.

Factor	Practice in the EAC Partner States and Recommended Guidelines
Design depth	Design manuals for Tanzania and Uganda consider it as a design factor; nevertheless, there is variation in the general depth requirements. We recommend that depths of 0.8 m and 1.2 m should be adopted as subgrade design depths for normal traffic load categories and heavy traffic loading, respectively.
Centreline soil survey	This design concept is addressed in Tanzania and Uganda design manuals. We recommend a minimum of 2 CBR strength tests per km for paved trunk roads, a minimum of 1 CBR strength test per km for other roads and a minimum of 1 CBR strength tests per 2 km for gravel roads should be adopted as a minimum materials CBR testing frequency for EAC Partner States.
Characterisation of subgrade strength design value	Kenya, Tanzania and Uganda use CBR as subgrade strength and there is variation in the way CBR design value is established, while French standards which is followed by Burundi and Rwanda characterises subgrade strength in term of modulus value. We recommend the adoption of the statistical approach to estimating subgrade CBR design value for a section, which takes the 90%-ile CBR test value for a homogeneous section as the CBR design value.
Subgrade class and improved layers	There is variation in the grouping of subgrade classes and warrants for improved subgrade layers in the manuals used in Kenya, Tanzania and Uganda. We therefore recommend that Subgrade design classes should be categorized into three classes: S3, S7 and S15 where G15 is natural gravel/soils with nominal CBR value of minimum 15, G7 is natural gravel/soils with nominal CBR value of minimum 7, and G3 is natural gravel/soils with nominal CBR value of minimum 3. Improved subgrade layers should only be applied where subgrade strength values are S3 and S7.

Pavement Materials: Under this design parameter, a number of factors were reviewed, compared and contrasted among the EAC Partner States. Following the detailed review, the study recommends codes for unbound and cemented and surfacing materials for the EAC Partner States, as indicated in the next chapter. 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 roadbase layer of Hot Rolled Apshalt (HRA), with DBM or Dense Tarmacadam (DTM) as the reminder of the roadbase material.

Pavement Rehabilitation Design: Pavement rehabilitation design methods were also reviewed and we have found some variation in the methods recommended by existing design methods. We recommended maximum deflection method, structural number method and mechanistic methods for EAC as discussed in this Annex A 3.

Application of Analytical Pavement Design Method in the Longer Term

Following the review of analytical pavement design methods, we recommend that EAC Partner States should consider and start adopting an analytical pavement design method in the region. The method should be based on experience, theory of pavement structural and material behaviour. It should take into account of local conditions of climate, traffic, available local materials, and other factors. With such a method it will be possible to carry out structural analysis of pavements and the prediction of their performance from the calculated parameters. However, in order for such a method to be successful attention should be paid to the methods of characterisation of the pavement layers and subgrade to meet the requirements of a theoretical model, calculation of parameters considered to have a primary influence on selected aspects of pavement performance and utilisation of these parameters in performance models to evaluate the structural adequacy of the pavement under consideration. This Annex provides details of the review and our recommendation.

Rigid Pavements

The study reviewed a number of issues which deserve special attention for successful and sustainable application of cement concrete pavement construction. A wide range of technical aspects of concrete pavement design have been covered. It has been pointed out that the design of concrete pavements is not just a matter of considering key design inputs which influence thickness design but also the selection, specification and constructability of features of concrete pavements. Also, it has been indicated that concrete roads have been used successful in some developing countries and there is great potential for their use in EAC.

Taking into account the special advantages of concrete pavement, we propose that concrete roads should be used on all heavily trafficked interurban and urban roads in the EAC region.

Super Singles

We found that the first generation of wide base tires (385/65R22.5, 425/65R22.5) slightly increases the induced pavement damages. It can be concluded that first generation of wide base tires (385/65R22.5, 425/65R22.5) slightly increases the induced pavement damages. However, with the exception of 445/50R22.5, results on the performance of second generation of wide base tires are still mixed in the sense that some studies have found 455/55R22.5 tires to induce approximately the same pavement response or damage as the standard dual assembly while others have found that these tires still cause greater fatigue damage, subgrade rutting, and HMA rutting (densification) compared to conventional dual-tire assembly when carrying the same load. Additionally, the 455/55R22.5 is noted to cause less HMA rutting (shear) and base shear failure potential than the conventional dual-tire assembly. Our review, and more particularly studies by COST 334 and CSIR (2010), showed that replacement of single tyres on steering axles by wide base singles results in a reduction of pavement damage. Accordingly, CSIR (2010) give recommendations which need to be considered by the EAC. In line with the conclusion and recommendations drawn by CSIR (2010) and the studies reviewed in the previous sections, we recommend that the use of super single tyres of 385 mm and conventional tyres of 315 mm as single tyres on legal axles and axle units should be banned forthwith.

Additionally, we recommend that:

a) Because of the continued development of new and improved tires, it should not be concluded that all new generation of wide-base tires will increase pavement damage. Rather, EAC should initiate and sustain continued research and investigation into the immediate and prolonged effects of tires as they are developed. This includes analysis through mechanical models, measured pavement models, and field performance.

b) Since the investigation of effects of wide base tires on pavements involve assessment of pavement responses in terms of stress, strain, and deflections, it is high time for the EAC Partner States to use analytical methods for the design of new or rehabilitation of pavements.

Bridge Design Standards

The EAC Partner States are basically using developed countries bridge design standards. We found that Burundi and Rwanda are following French design standards while Kenya, Tanzania and Uganda are using British design standard. Additionally, it should also be noted that the SADC region has its own bridge design standard, SATCC code of practice for bridge design. However, BS design standards were withdrawn from use in UK on 31st March 2010 and as a result UK is now using BS EN which is customised to Eurocodes which are being implemented across the European Union to deliver a common approach to design, whilst at the same time allowing the application of country specific construction requirements such as wind loads or earthquake resistance.

In recommending increase in GVM, PADECO Report assumed the use of BS in the EAC region. However, the continued application of BS standard which is superseded by the European Standards (EN Series) in EAC member countries will pose additional problems as BS becomes redundant.

We recommend that EAC member countries should use Euro code 1, Part 2 final draft prEN 1991 - 2. Actions on structures - Part 2: Traffic loading on bridges. In the longer term, however, it will desirable for EAC region to develop its own Bridge Design Standards.

Impact of Harmonisation

The impacts of harmonising pavement and bridge design standards include the provision of suitable pavement structures that will meet functional and structural performance criteria through the service life of the pavement, and uniform and comfortable riding surfaces to all road users thus resulting in the provision of the same quality of service throughout the region. Additionally, it is proposed that the recommended empirical-based standards be implemented in all new road designs, and road rehabilitation and reconstruction projects in the shorter term, i.e. within 10 years and beyond this period analytical methods should be used in the EAC Region.

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3. HARMONISATION OF PAVEMENT AND BRIDGE DESIGN STANDARDS

3.1 Introduction

3.1.1 General

Generally, a number of pavement types are available through modern pavement technology with such terms rigid pavement, flexible pavement, composite pavement, and full-depth asphalt pavement, among others. Each of these terms has been developed for some particular reason and each has a useful connotation. However, it is recommended to categorise pavements according to their reaction to loads. Thus, pavements are normally termed as rigid or flexible. This classical categorisation is, in some cases, an over-simplification. However, the terms rigid and flexible provide a good description of how the pavements react to loads and the environment. The essential difference between the two types of pavements, flexible and rigid, is the manner in which they distribute the load over the subgrade. Usually materials for rigid and flexible pavements are cement concrete and asphaltic concrete, respectively.

Traditionally, the main focus of pavement design is to recommend a suitable pavement structure that will meet functional and structural performance criteria through the service life of the pavement. The design philosophy now involves not only a pavement design strategy that seeks to identify the best initial structural section, but also the best combination of materials, construction technologies, and maintenance and rehabilitation strategies. Therefore the basic objective of pavement design is to provide structural alternatives that are feasible both technically and economically. This is achieved by specifying pavement layer thickness with proper types of materials based on the traffic and environmental conditions and the life cycle costs analysis. As a result a number of feasible strategies for different combinations of pavement materials and performance periods for a particular set of pavement design data can be obtained. Prior to the review and selection of standards for the road pavement design the general design principles, design philosophies and parameters are listed which form the basis for all major design standards.

3.1.2 Study Objectives

The overall objective of the assignment is to make it possible to have a reliable, efficient and safe road transport services.

The chapter give recommendations about harmonisation of pavement and materials design standards for the EAC Region. It therefore discusses established pavement and materials design standards within the EAC member countries as well as applicable SADC and other international standards, and makes recommendations for the EAC.

3.2 Overview of Pavement Design Methods

3.2.1 Design philosophies

Pavement design methods can be grouped into two broad categories: empirical and analytical. Empirical methods are those that have evolved from observation of the performance of experimental pavements laid either on public roads, and hence subjected to normal road traffic or on test tracks where loading was strictly controlled. The majority of current methods of flexible pavement design fall under the empirical category.

The analytical term includes the structural analysis of candidate pavements and the prediction of their performance from the computed parameters. Thus, in analytical pavement design it is assumed that pavements deteriorate due to repetitions of the stresses, strains and deflections generated by traffic loads, ultimately reaching a terminal condition that necessitates strengthening. Performance models attempt to relate those parameters associated with a particular mode of deterioration to the number of repetitions of these that can be sustained before a terminal condition is reached. In the more advanced design methods the models have been derived by correlating the results of laboratory and theoretical studies with empirical data. In the majority of analytical design methods it is assumed that the pavement and subgrade can be modelled as a system of elastic layers of finite thickness supported by a semi-infinite elastic mass, and that the stresses, strains and deflections generated in the system, by traffic, can be determined by solving the general elastic equations governing the behaviour of multilayer linear elastic systems.

Analytical methods involving the use design chart formats impose constraints on the design in that it is unlikely to cover all of the site-specific factors that can influence design decisions, e.g., climatic and subgrade conditions that exist at a site, anticipated loading characteristics, materials available for construction, and local construction techniques. Thus, a rigorous analysis that seeks to take account of these factors is desirable. Such analysis is now feasible with the advent of appropriate software for use on personal computers.

3.2.2 Flexible Pavements

3.2.2.1 General design principles

The objective in the design of the road pavement is to select appropriate pavement and surfacing materials to ensure that the pavement performs adequately and requires minimal maintenance under the anticipated traffic loading for the design period adopted. This selection process involves adoption of material types, thicknesses and configurations of the pavement layers to meet the design objectives. Performance objectives are to:

- Provide safe and comfortable riding conditions to all road users
- Provide minimum whole life cycle cost

3.2.2.2 Design parameters

When designing or selecting pavement, there are four fundamental design parameters to consider:

- The environment
- Design Traffic
- The characteristic of the subgrade upon which the pavement is placed,
- Pavement and Surfacing Materials

The environmental factors that significantly affect pavement performance are moisture and temperature. Both of these factors must be considered at the design stage of the pavement.

The following factors relating to moisture environment must be considered in determining the design subgrade strength/stiffness and in the choice of pavement and surfacing materials:

- Rainfall/evaporation pattern
- Permeability of wearing surface
- Depth of water table and salinity problems
- Relative permeability of pavement layers
- Whether shoulders are sealed or not
- Pavement type

The effect of changes in moisture content on the strength/stiffness of the subgrade shall be taken into account by evaluating the design subgrade strength parameters (i.e. CBR of modulus) at the equilibrium moisture content likely to occur during the design life, i.e. the Design Moisture Content. The provision of subsurface drainage may, under certain circumstances, allow a lower Design Moisture Content, and hence generally higher Design CBR.

The effect of changes in temperature environment must be considered in the design of pavements with asphalt wearing surfaces, particularly if traffic loading occurs at night when temperatures are low, thus causing a potential reduction in the fatigue life of thin asphalt surfacing. The effect of changes in temperature environment should also be considered for bound or concrete layers.

The pavement design report shall include all considerations for environmental factors, and any assumptions made that would reduce or increase design subgrade strength, or affect the choice of pavement and surfacing materials.

The other basic input data for pavement design are design life to be specified, the traffic to be carried and the strength of the subgrade. Design life is defined in terms of the cumulative traffic that can be carried before strengthening of the pavement is necessary. In this context, design life does not mean that at the end of the period the pavement will be completely worn out and in need of reconstruction; it means that towards the end of the period the pavement will need to be strengthened so that it can continue to carry traffic satisfactorily for a further period. The traffic loads are expressed in terms of accumulated equivalent standard axle loads (ESAL) per direction, passing the road during the scheduled design life.

Except where a mechanistic design approach is employed, the strength of road subgrade is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Where a mechanistic design approach using linear elastic theory is employed for flexible pavements, the measure of subgrade support is commonly assessed in terms of the elastic parameters (modulus, Poisson’s ratio). The following factors must be considered in determining the design strength/stiffness of the subgrade:

- The compaction moisture content and field density specified for construction
- Moisture changes during service life
- Subgrade variability
- The presence or otherwise of weak layers below the design subgrade level

Pavement materials can be classified into essentially four categories according to their fundamental behaviour under the effects of applied loadings:

- Unbound granular materials, including modified granular materials
- Bound (e.g., cemented) granular materials
- Asphaltic Concrete
- Cement Concrete

3.2.3 Rigid Pavements

3.2.3.1 Pavement Design and Selection Considerations

Unlike flexible pavements, concrete pavements do not require the base or subbase for structural support and subgrade strength is not a critical element in the thickness design. Subgrade has minor impact on the overall thickness in terms of structural design but is a consideration for drainage. However, proper design and construction of rigid pavements are related to uniform support. The applied load is transferred across the rigid structure so that only a small bearing stress is applied to the underlying foundation. Bases or subbases provide a working platform during construction. A permeable subbase is often used under a rigid pavement for drainage purposes and can be either stabilised or unstabilised. If a rigid pavement is being constructed over a poor subgrade material, it is generally desirable to use subgrade stabilisation in expansive soils or install subdrains to eliminate or reduce subgrade moisture levels.

The key design parameters required by any cement concrete pavement design procedure are design traffic, foundation support, subbase material properties, environment, concrete material properties, performance criteria, and design reliability (Hall, 2002). The design of cement concrete pavements also includes many other elements other than thickness design. Such elements include joint design, load transfer design, steel reinforcement design and the selection and specification of other design-related features of concrete pavements.

3.2.3.2 Basic Components and Types of Cement Concrete Pavements

Concrete pavements have a number of features. As a result of the versatility and the range of features not all concrete pavements are similar. The basic features of cement concrete pavements include foundation support (subgrade), subbase, concrete materials, slab thickness, transverse joints, dowel bars, longitudinal joints, tiebars, surface smoothness and texture, subsurface drainage provisions, lateral edge support, joint spacing, and joint sealant material with related construction details and requirements. Therefore a variety of options is available for each of concrete pavement features. For instance, subbases under concrete highways are typically constructed of dense graded granular material, lean concrete, open-graded (drainable) granular material, cement or asphalt stabilised material, both dense and open-graded. Choices for paved shoulders include asphalt, partial-depth concrete, and full-depth concrete. Gravel shoulders are also used when standards permit.

Concrete pavements have been categorised into three common types: Jointed Plain Concrete Pavements (JPCP), Jointed Reinforced Concrete Pavements (JRCP), and Continuously Reinforced Concrete Pavements (CRCP) (Hall, 2002). Prestressed concrete pavements (PCP) have also been used to a lesser extent. The one item that distinguishes each type is the jointing system used to control crack development. For various reasons concrete shrinks, contracts and expands and that these actions induce cracks. It is equally important to know that this natural cracking can be easily controlled by the appropriate use of joints and/or reinforcing steel within the pavement.

3.2.3.3 Concrete Pavement Design Parameters

The key input parameters in any concrete pavement design procedure are as follows.

a) Traffic over Design Period

The number of heavy (truck) axle loads anticipated over the design period must be estimated on the basis of current truck traffic weights and volumes along with growth projections.

b) Subgrade

The modulus of subgrade reaction (k-value) of the pavement foundation is usually measured by plate bearing tests but it can also be estimated from correlations with soil type, soil strength measures such as the California bearing ratio (CBR), or by backcalculation from deflection testing on existing pavements.

c) Environment

Daily and seasonal variations in temperature and moisture influence the behaviour of concrete pavements in many ways, including:

- Opening and closing of transverse joints
- Upward and downward curling of the slab
- Permanent upward curling of the slab
- Upward warping of the slab
- Erosion of base and foundation materials
- Corrosion of dowel bars, steel reinforcement, or both

Although the effects of climate on concrete pavement behaviour and performance are recognised, concrete pavement thickness design practice traditionally has not explicitly considered most of these climatic effects. Several recent field and analytical studies have contributed greatly to better understanding and quantifying these effects so that they may be more adequately considered in thickness design.

d) Concrete Material Properties

For the purpose of pavement thickness design, concrete is characterized by its flexural strength as well as by its modulus of elasticity. Concrete flexural strength is usually characterized by the 28-day modulus of rupture (MR) from third-point loading tests of beams, or it may be estimated from compressive strengths. The corresponding elastic modulus (E) can also be measured but is usually estimated from strength data. In addition to its strength and stiffness, the durability of the concrete mix is important to the long-term performance of the pavement.

e) Base

A base course provides a stable platform for construction of the concrete slab, improves the smoothness achieved in the paving of the slab and the drainage of the pavement structure, and protects the foundation from frost penetration. Some types of bases also significantly reduce bending stresses and deflections in the slab and improve load transfer at joints and cracks. The estimated elastic modulus of the base, its erodibility, its potential for friction and bond with the concrete slab, and its drainability are factors considered in characterizing the support to the concrete slab and the quality of subsurface drainage.

f) Performance Criteria

All pavement thickness design procedures incorporate performance criteria that define the end of the performance life of the pavement.

g) Design Reliability

The reliability level or, generally speaking, the safety factor for which a pavement is designed reflects the degree of risk of premature failure that the pavement may undergo. Facilities of higher functional classes and higher traffic volumes warrant higher safety factors in design.

3.3 Pavement Design in the EAC Member States

3.3.1 Pavement Design Standards

Burundi and Rwanda have been following French standards for pavement design while Rwanda is considering to shifting from French standards to American standards, i.e. AASHTO. Kenya, Tanzania and Uganda are using their own design standards which are largely empirical-based methods. Table 1 provides a summary of design standards that are used by EAC Partner States. It was also found that Kenya is currently reviewing its design standards.

Table 1: Pavement Design Standards in EAC Partner States

Country	Design standard	Year of latest version of standard	Remarks
Burundi	AFNOR (French design standards)	1997	-
Kenya	Road Design Manual. Part III, Materials and Pavement Design for New Roads	1987	A new design manual is currently under preparation
Rwanda	AFNOR (French design standards)	1997	Considers shifting to AASHTO (for flexible and rigid pavements)
Tanzania (Mainland and Zanzibar)	Pavement and Materials Design Manual	1999	-
Uganda	Road design manual: Volume 3: pavement design, Part I: Flexible Pavements	2010	-
	Road design manual: Volume 3: pavement design, Part II: Rigid Pavements	2010	-
	Road design manual: Volume 3: pavement design, Part III: Gravel Roads	2010	-
	Road design manual: Volume 3: pavement design, Part IV: Pavement Rehabilitation	2010	-

The design manuals have particular reference to the prevailing conditions in the respective countries and reflect EAC member countries' experience gained in road pavement design during the last 40 years. For Tanzania and Uganda, the manuals supersede the previous design guides of 1989 and 1994, respectively. Generally, revision of a design manual is only possible when new technical information and performance data become available.

3.3.2 Areas of Commonality and Divergence

A review of the documents listed in Tables 1 revealed a number of areas which are common and unique to particular EAC Partner State as well as areas which need further improvement. These areas are discussed in detail in the following section with a view of identifying potential areas for harmonisation.

On the other hand, French design standards, which is followed by Burundi and Rwanda, combine the useful aspects of rational mechanics and lessons gained from experimentation. They employ mechanical methods to define models of the pavement structure and calculate stresses produced by a standard load for the legal axle, utilise laboratory-based fatigue damage test results, and knowledge derived from observation of the behaviour of real pavement is used to specify criteria to check pavement strains of the subgrade and unbound layer and to tie in with the results of the computation model. They therefore specify materials strengths in terms of modulus values (MPa).

3.4 Potential Areas for Harmonisation and Improvement

3.4.1 Flexible Pavement Design Standards

3.4.1.1 Environment

One of the factors that affect pavement performance is the issue of environmental factors. The environmental factors that have the greatest effect on pavement performance are:

- Moisture regime in the pavement structure
- Pavement temperature
- Unfavourable subgrade conditions related to the environment

a) Moisture regimes

The moisture regime has a major influence on a pavement's performance as the stiffness and strength of subgrade soils and granular materials vary with their moisture content. Moisture changes in pavements usually ensue from one or more of the sources shown in Figure 1. Table 2 shows measures that can be employed to reduce/minimise moisture variation in subgrades and pavements.

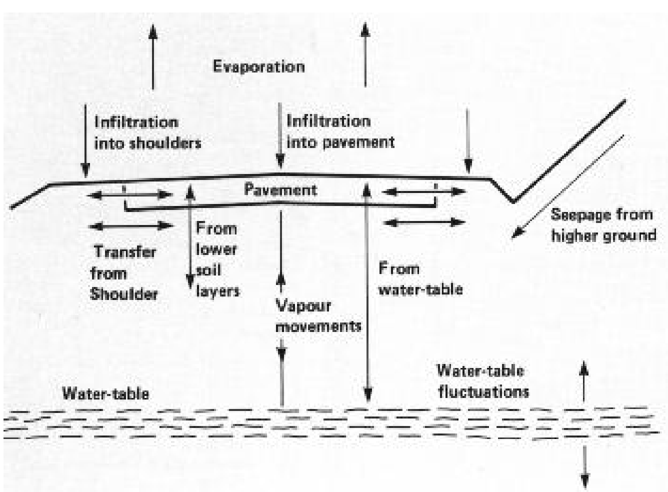


Figure 1: Ways through which moisture can enter/leave the subgrade and pavement

Table 2: Source of moisture in subgrade soil and pavement layers

Source	**Control Measure**
Relative permeability of pavement layers and subgrade	Both can be controlled through appropriate design of the cross section and internal drainage of the pavement layers
Infiltration from the surface and shoulders	
Seepage from higher ground	Both can be controlled by adequately installed subgrade and pavement drains
Fluctuation in the water table	
Transfer of water in vapour form due to differences in temperature and/or moisture content	Cannot normally be controlled other than by attempting to keep the moisture content near constant

Existing design manuals in the EAC regions indicate that the region may be subdivided into two climatic zones. Within each climatic zone there may be localised areas with different moisture conditions.

Kenya design manual (1981) reported a very wide variety of climates as follows:

- Afro-alpine climate
- Equatorial climate
- Wet-tropical climate
- Semi-arid climate
- Arid climate
- Very arid climate.

The climates are largely governed by altitude and this diversity is illustrated by the mean annual rainfall map and by the air temperature charts given in the manual. For the purpose of pavement design, however, the manual divides Kenya climatic condition into two climatic zones.

- Wet areas – mean annual rainfall greater than 500 mm
- Dry areas – mean annual rainfall less than 500 mm

The pavement design manual for Tanzania specifies three climatic zones and each zone is characterised by different environmental factors. The climatic zones are demarcated on the basis of the number of months in a year with surplus of rainfall over potential evaporation. Within each climatic zone there may be localised areas with different moisture conditions.

- A dry zone as an area in which the number of months per year with higher rainfall than evaporation is less than 1 month
- A moderate zone as an area in which the number of months per year with higher rainfall than evaporation is 1 to 3 months
- A wet zone as an area in which there are more than three months per year with higher rainfall than evaporation.

However, the design catalogues provided by the manual considers only two climatic zones; wet and dry/moderate, which means the is no difference between dry and moderate climatic zones in terms of their moisture effects on subgrade performance as shown in the pavement catalogues.

On the other hand, the pavement design manual for Uganda defines two climatic zones; wet and dry zones. A dry region is a region where annual rainfall is less than 250 mm and there is no likelihood of moisture ingress due to factors such as significant flooding (in low-lying flood plain areas, or in tidal basins, for example), underground springs or wells, or any other detrimental conditions. A wet region is defined as a region with rainfall more than 250 mm and that there are periods in which conditions will lead to significant possibility of moisture ingress to the pavement.

Recommendation

EAC should consider subdividing the EA region into the categories of wet, moderate and dry climatic zones for the pavement design purposes. A dry zone is as an area in which the number of months per year with higher rainfall than evaporation is less than 1 month. A moderate zone is as an area in which the number of months per year with higher rainfall than evaporation is 1 to 3 months, whereas a wet zone is as an area in which there are more than 3 months per year with higher rainfall than evaporation.

b) Pavement temperature

The pavement temperature is taken into consideration in mix design, and more specifically in the design of bound materials e.g. bituminous mix design. However, the effect of temperature in unbound materials is not specifically taken into account in the pavement design. The performance of surface treatments depends largely on pavement temperature and is taken into account in the surfacing design as it is related to bleeding and loss of aggregate, and also to the rate of binder ageing in the long term.

It should be noted that refinement to the concepts of bitumen mix design procedures involves refined methodologies for materials selection, selection of the design aggregate structure, selection of the design binder content, and evaluation of the mixture for moisture sensitivity. For the case of binder selection, for example, a refined methodology involves characterisation of binders at the actual pavement temperatures that they will experience, and at the periods of time when bitumen distresses are most likely to occur.

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

Recommendation

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

3.4.1.2 Drainage

Drainage systems can be categorised as surface drainage or subsurface drainage system. Surface drainage encompasses all means by which surface water is removed from the pavement and right of way of the road.

The surface drainage system include

- adequate transverse and longitudinal slopes on both the pavement and shoulder to ensure positive run-off,
- longitudinal channels (ditches),
- culverts, and
- Bridges to provide for the discharge of the surface water to the natural waterways.

Storm drains and inlets are also provided on the median of divided highways.

In subsurface drainage system, it is attempted to keep the variation of moisture in the subgrade and pavement layers to a minimum at the same time reducing the amount of the same in the subgrade soil and pavement.

A review of the design manuals in the EAC region indicates that manuals for Tanzania and Uganda provide guidance on the cover the issue of drainage.

Drainage of the subgrade

Provision of sufficiently deep open side drains or alternatively, special drainage facilities such as subsurface drains will ensure proper drainage of the subgrade. Special consideration to design and construction details is required where the occurrence of rock may trap water in the subgrade or pavement structure.

Open side drains - general

Open side drains shall at no point be less than 0.5 metres deep, measured from the bottom of the drain up to the formation level.

Open side drains in cuttings

The general requirement for the depth of open side drains in cuttings is minimum 1.0 metres measured from the bottom of the drain up to the Formation level. This depth can be reduced to 0.5 metres provided cement or lime modification of the subgrade is employed (see Figure 2). In such cases the minimum depth of cement or lime modification is 200 mm. The given requirements refer to cuttings in soils. For cuttings in solid rock the required drainage measures depend on site conditions and shall be decided in each individual case.

The method for drainage of cuttings shall be specifically described in the detailed design of projects. The need for subsurface drains as an alternative to open side drains in cuttings shall be assessed.

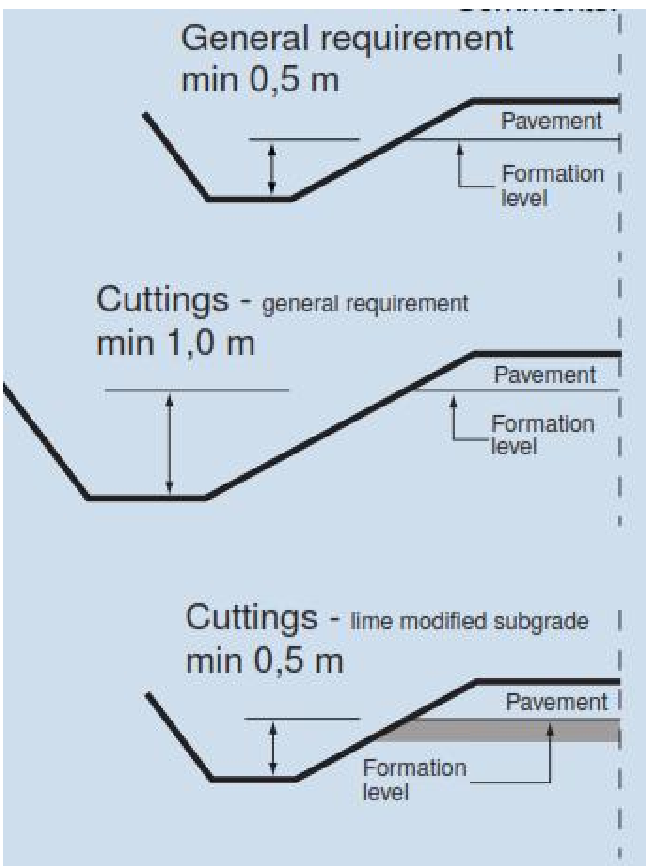


Figure 2: Drainage in cutting (Source: MoW-URT, 1999)

Subsurface drains

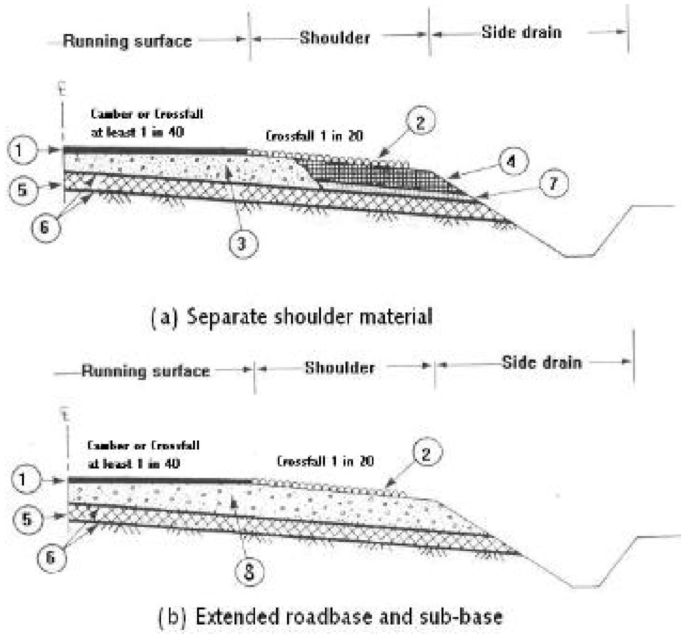
The need for subsurface drains depends on site conditions and requires careful consideration due to the high construction cost of these facilities. Urban areas, occurrence of subsoil wells and cuttings are among typical conditions where use of subsurface drains shall be considered.

Drainage of the pavement layers

Proper drainage of granular pavement layers is essential for their performance and is ensured by appropriate attention to cross section details.

Granular base course

Where a granular base course and paved shoulders are used, the base course and subbase layers shall be extended to the full width of the shoulders, as shown in Figure 3.



1. Impervious surfacing
2. Shoulders surface dressed (giving contrasting texture to running surface)
3. Roadbase extending under shoulder for at least 500mm
4. Shoulder material capable of supporting occasional traffic
5. Impervious subbase carried across full width of construction
6. Formation and sub-base constructed with crossfall of 1 in 30 (providing drainage path for any water that enters and also a thicker and stronger pavement on the outside wheel track)
7. Drainage layer of previous material
8. Roadbase extending through shoulder

Figure 3: Cross section of road showing drainage arrangement (Source: MWT-RU, 2010)

Cemented or bituminous base course

Where economically possible the base course should be extended to the full width of the shoulders.

Boxed-in pavements

Boxed-in pavement structures, where water may be trapped in the pavement, shall not be used. Appropriate measures to ensure proper drainage of the pavement layers shall be included in the design where internal drainage of the layers may be impaired for any reason. The following circumstances carry particular risks of attaining a boxed-in structure:

- where shoulders are designed with different materials than the carriageway using unfavourable combinations of materials
- where kerbstones are extended into granular layers of the pavement
- where un-paved shoulders made of near impermeable materials are used

Drainage of the road surface

Drainage of the road surface is ensured by providing sufficient crossfall of the carriageway and shoulder in accordance with the standard cross sections.

Filter Layers

Large difference between the grading of materials in adjacent layers carries risk of undesirable infiltration of fines into the matrix of the coarser material, e.g. between earthworks and pavement layers. In such cases the grading of the materials shall be assessed against the filter criteria given below and construction of a filter layer shall be carried out if required. Alternatively, geo-textiles can be used where cost calculations show this option to be more economical.

Filter criteria for soil/gravel materials

The criteria for grading of materials in adjacent layers are fulfilled when the following two requirements are met.

$$\frac{d_{15}\ \textit{for the filter material}}{d_{85}\ \textit{for the subsoil}} \leq 5$$

and

$$\frac{d_{50}\ \textit{for the filter material}}{d_{50}\ \textit{for the subsoil}} \leq 25$$

The criteria below should be met if a filter material with better drainage properties than the subsoil is desired, such as in locations where water flows out of the subsoil. The designer should carefully consider whether a filter layer with draining properties benefits the integrity of the structure.

$$\frac{d_{15}\ \textit{for the filter material}}{d_{15}\ \textit{for the subsoil}} \geq 5$$

3.4.1.3 Traffic

a) Design period

Tanzania Manual specifies the length of pavement design period as 20 years for new or rehabilitated bitumen surfaced pavements. However, the manual provides for the Ministry of Works, at its discretion, to change the design period depending on circumstances of individual projects.

On the other hand, Kenya manual recommended stage construction for an initial design period of 15 years while Uganda recommends a maximum design life of 20 years and a minimum of 10 years depending on a number of factors and uncertainties which must be specified by the designer.

Some countries, including France, Germany, USA and the UK, already permit design periods in excess of the frequently used 20 years (FEHRL, 2004). However, it should be emphasised that in Europe, solely the UK states that their pavements are designed for 40 years (FEHRL, 2004). In countries such as the Netherlands, flexible pavements are designed for an initial 20 year period, and if no sign of structural deterioration can be observed following 20 years of service, the existing pavement can be overlaid to become a long-life-pavement (Stage-by-stage concept). In other countries, e.g. France, pavement design life is guided by an economic strategy, which usually is carried out over 30 years. This means that French design method uses an initial 20-year investment period with a low level of risk, and the economic comparisons between structures types are carried out over a period of 30 years (40 years on conceded motorways).

Pavement design of new roads and reconstruction projects should normally be based on estimated traffic 20 years after completion of construction. With economic justification, design periods other than 20 years may be approved by the relevant authorities.

Recommendation

A design period of maximum of 20 years should be considered for adoption by EAC Partner States. With economic justification, design periods other than 20 years may be approved by the relevant authorities.

b) Design Traffic Loading

Heavy Vehicle Traffic Data

The procedure to determine the design traffic loading involves heavy vehicles counts. Heavy vehicles are defined as vehicles having a registered un-laden weight of 3 tonnes or more in the design manual for Tanzania. Large buses having a seating capacity of 40 or more are also considered as heavy vehicles. Traffic counts are classified into four heavy vehicle categories while the design manuals for Kenya and Uganda defines a heavy vehicle as a vehicle (including bus and goods vehicle) of more than 15 kN un-laden weight. In Kenya, three (3) categories of heavy vehicles are outlined while there are four (4) vehicle classes are specified in Uganda design standard. Table 3 provides a summary of the vehicle categories definitions.

On the other hand, French design manual, which is followed by Burundi, defines a heavy lorry as a vehicle with a payload of 5 tonnes or more.

Table 3: Definition of Heavy Vehicle Categories

Heavy vehicle category	**Kenya**	**Tanzania**	**Uganda**
	Definition		
Small cars	-	-	Passenger cars, minibuses (up to 24 passenger seats), taxis, pick-ups, and land cruisers, land rovers etc.
Medium Goods Vehicle MGV	2 axles goods vehicles	- 2 axles, including steering axle, and - 3 tonnes empty weight, or more	Small and medium sized trucks including tankers up to 7 tons load
Heavy Goods Vehicle HGV	All goods vehicles having more than 2 axles	- 3 axles, incl. steering axle, and - 3 tonnes empty weight, or more	Trucks above 7 tons load
Very Heavy Goods Vehicle VHGV	-	- 4 or more axles , incl. steering axle, and - 3 tonnes empty weight, or more	Articulated truck- trucks with trailer or semi-trailer and tanker trailers

Buses	Passenger vehicles having seating capacity of more than 9 including the driver	Seating capacity of 40, or more	Medium and large size buses above 24 seating capacity

The most simple vehicle classification used is a system that distinguishes between light and heavy vehicles, i.e. the number of heavy vehicles is expressed as a percentage of the total number of vehicles. Smith and Visser (2004) noted that this system was used for many years but due to the large diversity in the characteristics of different heavy vehicles, a need for a more sophisticated classification system has developed. They reviewed previous work and identified a system that takes into consideration the difference in characteristics between heavy vehicles. According to this system heavy vehicles are classified into short heavy vehicles (rigid-chassis two or three-axle heavy vehicles), medium heavy vehicles (horse-plus-semi-trailer combination) and long heavy vehicles (horse-plus-semi-trailer combination with a full trailer). This classification system furthermore assumed that short heavy vehicles had an overall length of not more than 11 meters, medium heavy vehicles with an overall length between 11 meters and 18 meters and long heavy vehicles with an overall length of more than 18 meters.

Smith and Visser (2004) noted further that international vehicle classification system approaches vary extensively from country to country. The type of vehicle in a specific country or region is a function of the legislation stipulating the permissible vehicle lengths and axle loads, the available vehicles on the local markets and the commodity that is being transported. Most of the different heavy vehicle classification systems provide heavy vehicle combinations commonly known. Smith and Visser reported on previous works which indicated a vehicle classification, which is mainly used for pavement design and a TRL vehicle classification system which is used in England and other European Economic Community (EEC) countries. The emphases of these classification systems are on the type of heavy vehicle, i.e. rigid-chassis, linked-chassis or a combination.

Jiang et al (2008) reported that the Federal Highway Administration (FHWA), USA, vehicle classification defines 13 types of vehicles as shown in Figure 4. This vehicle classification system is based on the number of axles per vehicle and is exclusively used for collecting the traffic data needed for mechanistic-empirical pavement design by State Highway Agencies (SHAs) throughout the USA. The first three types of vehicles (Classes 1, 2, and 3) are not considered in pavement design and Class 9, which are typically five axle single trailer type vehicles, are responsible for 80 to 90 percent of traffic loading on the interstate system.

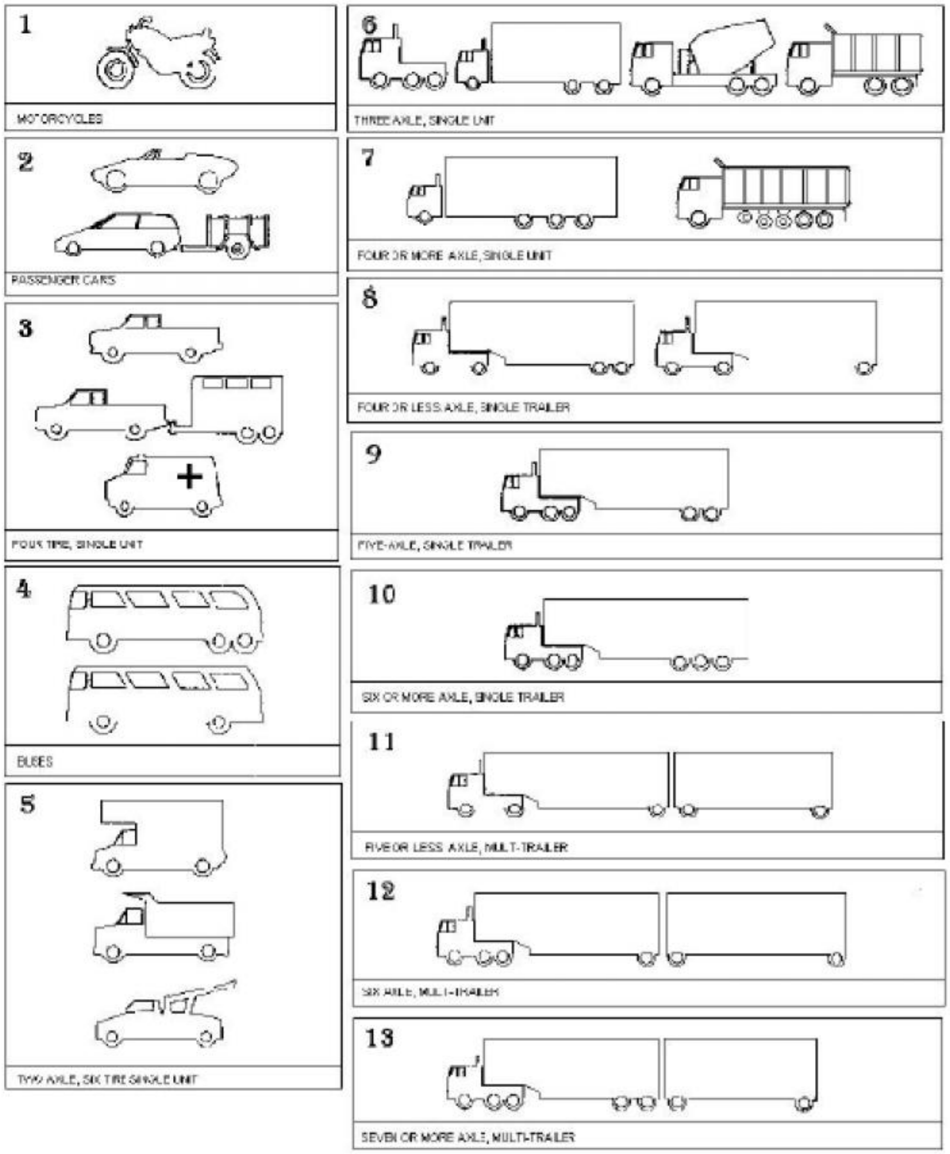


Figure 4: FHWA Vehicle Classifications (Source FHWA)

Recommendation

EAC should consider adopting heavy vehicle classifications of medium goods, heavy goods, very heavy goods, and buses having seating capacity of more than 24 including the driver*. Where a Medium Goods Vehicle (MGV) covers small and medium sized trucks including tankers up to 7 tons load, Heavy Goods Vehicle (HGV) covers Trucks above 7 tons load, and Very Heavy Goods Vehicles (VHGV) are articulated trucks with trailer or semi-trailer and tanker trailers.*

By definition, a heavy vehicle is a vehicle having a registered un-laden weight of 3 tonnes.

c) Determination of Traffic Loading

To determine traffic load, the value of 4 which is commonly cited "fourth-power damage factors" is usually used in the computation of traffic load equivalency factors for different heavy axle loads. It has been noted that the value is influenced by various factors, with the most significant being pavement configuration. In this regard, the manual for Uganda recommends relative damage exponents for various pavement base/subbase configurations (i.e. granular/granular n=4, granular/cemented n=3, cemented/cemented n=4.5, bituminous/granular n=4, bituminous/cemented n=4). Such guidance is not provided in other design manuals. However, the manual for Tanzania uses a damage exponent of 4.5. Generally, the value of n varies as a function of the pavement thickness, axle and tire configuration, and the modular ratio of pavement materials and therefore it becomes difficult to justify the use of different factors. COST 334 (2001) concluded that the precise exponent value is not very important and for exponent value between 2 and 6, most actual axle load spectra were found to translate to roughly the same equivalent number of standard axles. For low exponents, the multitude of smaller axle loads contribute to the bulk to the total equivalent number while for high exponents, the few overloaded axles contribute the most.

The cumulative damaging effect of all individual axle loads (i.e. traffic spectrum) is expressed as the number of equivalent 80 kN single-axle loads (ESAs or E80). Generally, it is assumed that this is the number of equivalent 80 kN single-axle loads that would cause the same damage to the pavement as the actual traffic spectrum of all axle loads. For the purposes of the pavement design catalogue, Tanzania design manual outlines seven (7) traffic load classes (TLC), the design manual for Kenya provides fives traffic classes (T), and in Uganda the design manual provides 8 design classes (T) as shown in Table 4.

Table 4: Comparison of traffic load classes

Kenya Manual (1981)	Tanzania Manual (1999)	Uganda (2010)
-	TLC 02, i.e. E80 < 0.2 million	T1, E80 <0.3 Million ESAs
T5, i.e. E80 is between 0.25 to 1 million	TLC 05, i.e. E80 is between 0.2 to 0.5 million	T2, i.e. 0.3 – 0.7 million ESAs
-	TLC 1, i.e. E80 is between 0.5 to 1 million	T3, 0.7 – 1.5 million ESAs
T4, i.e. E80 is between 1 to 3 million	TLC 3, i.e. E80 is between 1 to 3 million	T4, 1.5 – 3 million ESAs
T3, i.e. E80 is between 3 to 10 million	TLC 10, i.e. E80 is between 3 to 10 million	T5, 3 – 6, million ESAs
T2, i.e. E80 is between 10 to 25 million	TLC 20, i.e. E80 is between 10 to 20 million	T6, 6 – 10, million ESAs
T1, i.e. E80 is between 25 to 60 million	TLC 50, i.e. E80 is between 20 to 50 million	T7, 10 – 17 million ESAs
-	-	T8, 17-30 million ESAs

For Tanzania, where the heavy (>13 tonnes) axles proportion of E80 is 50% or higher the TLC is given an H index e.g. TLC 50-H in the Manual. It can be noted that Kenya and Uganda design manuals do not provide traffic classes for heavy axles (>13 tonnes).

Traffic design classes of E80 less than 0.3 million usually provide for very light to extremely light traffic, and may include pavements in the transition from gravel to paved roads. The Uganda manual appears to meet this requirement.

On the other hand, the French design manual groups traffic load into six major categories as shown in Table 5. Also, unlike Kenya, Tanzania and Uganda where the standard axle load is taken as 80 kN, French design guide, which is followed by Burundi and Rwanda, specifies a maximum axle load of 1130 kN.

Table 5: Definition of traffic loading classes

Classe	T5	T4	T3		T2		T1		T0		TS		TEX
			T3-	T3+	T2-	T2+	T1-	T1+	T0-	T0+	TS-	TS+	
MJA	0	25	50	85	150	200	300	500	750	1200	2000	3000	5000

Source: French design guide, 1997

The maximum number of design traffic that can be dealt with using the design manuals for Kenya, Tanzania and Uganda are 60, 50 and 30 million ESAs, respectively. The manual for Uganda recommends use of UK, US, Australian or South African TRH 4 guide for design traffic above 30 million ESAs while manuals for Kenya and Tanzania do not indicate alternative design methods for traffic design classes above their upper limits of ESAs.

The UK design methods are based on a number of governmental publications including the study reported by Nunn et al (1997), cited by O'Flaherty (2002), on the design of long-life pavements, i.e., roads with thick bitumen bound layers. The theoretical calculations versus layer thicknesses performed using relationships developed by the study included rutting, road base fatigue, surface cracking of the investigated road sections. At the time the study was initiated the most heavily trafficked motorways and trunk roads had carried in excess of 100 million standard axles, whereas the traffic levels had not exceeded 20 msa at the time that the design curves were developed. On the other hand, the US Federal Highway Administration's 1995-1997 National Pavement Design Review found that some 80 percent of States use the 1972, 1986, or 1993 AASHTO Guides. All those versions are empirically based on performance equations developed using 1950's AASHO Road Test data. Some of the major limitations which hindered continued use of previous AASHTO Guides were the inability to incorporate significant material properties and to estimate the consequences of increased/new loading conditions (ARA, 2004). The South African TRH 4 guide specifies that the structural performance criteria used to develop the different catalogue of structures are based on the South African Mechanistic Design Method (SAMDM) and were subsequently calibrated by practical considerations to conform with practical layer thicknesses.

The aforementioned review underscores the importance of analytical approach to the design of pavement structures carrying more than 20 msa.

Recommendation

1. We recommend that pavement design classes shown in Table 6 should be adopted by EAC member states, whereas for design traffic above 30 million ESAs analytical methods of design should be adopted.
2. To determine traffic load, we recommend a damage exponent of 4.5 as it represents the worst case scenario.

Table 6: Recommended Traffic Load Classes

T1, E80 <0.3 Million ESAs
T2, i.e. 0.3 – 0.7 million ESAs
T3, 0.7 – 1.5 million ESAs
T4, 1.5 – 3 million ESAs
T5, 3 – 6, million ESAs
T6, 6 – 10, million ESAs
T7, 10 – 17 million ESAs
T8, 17-30 million ESAs

3.4.1.3 Subgrade

a) Design Depth

The design depth is defined as the depth from the finished road level to the depth that the load bearing strength of the soil no longer has an effect on the pavement's performance in relation to traffic loading. Unlike Kenya, paved trunk roads subgrade in Tanzania have to meet general depth requirement which is 0.8 m and 1.2 m for heavy load classes. For other roads, the general depth requirement is 0.6 m while for heavy load classes 1.0 m depth is required. Likewise, the design manual for Uganda recommends minimum subgrade depths of 0.25 m to 0.65 m to which a designer should confirm that subgrade strength is available.

Recommendation
We recommend the depths of 0.8 m and 1.2 m as subgrade design depths for normal traffic load categories and heavy traffic loading, respectively.

b) Centreline Soil Surveys

Generally, test pits are excavated for the purpose of sampling the subgrade along the road centreline, and materials testing carried out at a minimum average frequency. Tanzania manual recommends investigation of soils along the road centreline to be carried out in order to establish its strength in terms of CBR. The manual recommended a minimum of 2 CBR strength testing per kilometre for paved trunk roads, a minimum of 1 CBR strength testing per kilometre for other roads and a minimum of 1 CBR strength testing per 2 kilometres for Gravel roads. The manual directs to determine the CBR design of all homogenous sections along the road centreline and the CBR design should be the 90%-ile value of the CBR test results for a section with homogenous strength. The value obtained is compared with the ranges of subgrade strength classification in order to classify the subgrade and the required improvements.

The soil centreline survey concept is not addressed by the design manuals for Kenya and Uganda.

Recommendation
We recommend a minimum of 2 CBR strength testing per km for paved trunk roads, a minimum of 1 CBR strength testing per km for other roads and a minimum of 1 CBR strength testing per 2 km for gravel roads as a minimum materials CBR testing frequency for EAC Partner States.

c) Characterisation of the Subgrade Strength Design Value

The road design manual for Kenya specifies subgrade strength values based on either CBR values measured after 4 days soak or measured at optimum moisture content where it has been established that prolonged soaking may occur. Unlike Kenya, design manuals for Tanzania and Uganda specify the CBR design for a section as the 90%-ile value of the CBR test results for a section with homogenous strength. However, the pavement design guide for Tanzania recommends further that the lowest CBR value encountered should be used as the CBR for design sections through cuttings. On the other hand, French design manual which is essentially specifies the mechanical characteristics of subgrade and of the capping layer by a resilient value of Young's modulus. The subgrade is generally treated as an elastic half-space described by values of Young's Modulus and Poisson's ratio. For Poisson's ratio a mean value of 0.35 is taken into calculation since this parameter varies with the nature of thee grounds, their hydrous condition and the stresses applied. The subgrade is also described by the thickness of soft soils, when bedrock has been found to lie at a shallower depth.

The current practice in Kenya, Tanzania and Uganda follows empirical design method while Burundi follows French design standard which combines the useful aspects of rational mechanics and lessons gained from experimentation.

Recommendation
We recommend the statistical approach to estimating subgrade CBR design value for a section, which takes the 90%-ile CBR test value for a homogeneous section as the CBR design value.

d) Subgrade Classes and Improved Subgrade Layers

Subgrade classes defined in the manuals for Kenya, Tanzania and Uganda are as shown in Table 7. It can be noted that subgrade classes in the Kenya design manual provide for subgrade CBR design values which are overlapping and consequently, may cause unnecessary confusion over the correct class of subgrade. Subgrade classes should be wide enough to take advantage of range of strong subgrades.

Table 7: Comparison of subgrade classes and CBR design values

Soil Subgrade Class	**CBR Range**				
	Kenya	**Tanzania**			**Uganda**
		Wet or moderate climatic zones 4 days soaked value	**Dry climatic zones (both requirements shall be met)**		
			Tested at OMC	**4 days soaked**	**Wet/ dry region**
S1	2 – 5	NA	NA	NA	2
S2	5 – 10	NA	NA	NA	3 – 4
S3	7 – 13	3 - 6	3 – 6	2 – 6	5 – 7
S4	10 – 18	NA	NA	NA	8 – 14
S5	15 – 30	NA	NA	NA	15 – 29
S6	>30	NA	NA	NA	30+
S7	NA	7 - 14	7 – 14	3 – 14	NA
S15	NA	Min 15	Min 15	Min 7	NA

On the other hand, the manual for Tanzania recommends all subgrade to be brought to the CBR of 15% as a minimum strength by constructing one or more improved subgrade layers where necessary. The numbers of improved subgrade layers depend on subgrade classes and the climatic zones. In Kenya, however, the manual recommends improved subgrade layers to be placed on S1, S2 and S3 subgrade classes and that the costs of subbase and improved subgrade materials should be taken into account. This means improved subgrade should only be considered when subgrade strength is below 13% CBR value. The catalogues of pavement structures provided in the design manual for Uganda show that improved subgrade layers are provided only when the subgrade CBR design value falls under S1, S2, S3 and S4 subgrade classes. This means selected subgrade layers are provided when subgrade CBR design value is below 15%.

Recommendation

1. *We recommend that subgrade design classes should be categorized into three classes: S3, S7 and S15 where G15 is natural gravel/soils with nominal CBR value of minimum 15, G7 is natural gravel/soils with nominal CBR value of minimum 7, and G3 is natural gravel/soils with nominal CBR value of minimum 3.*
2. *Additionally, we recommend that improved subgrade layers should only be applied where subgrade strength values are S3 and S7.*

3.4.1.4 Pavement Materials

Unlike the design manual for Kenya and Uganda, all materials in the Tanzania design manual are indicated by means of codes, e.g. G80, C2, CM, etc, which refer to materials with certain defined properties. All manuals including the French design guide, however, have included as much as possible all material types commonly used in the respective countries.

a) Unbound Materials

The unbound materials discussed in the manuals for Kenya, Tanzania and Uganda and their recommended physical properties and requirements are natural gravel and crushed materials. Table 8 presents a list of materials that are used in these countries for pavement subbase, base course and surfacing. Unlike the above mentioned manuals which characterise materials properties for pavement design in term of CBR and UCS (for stabilised materials) and index properties, French design standard specifies materials in terms of modulus values and Poisson ration.

Table 8: Materials Specified in Design Standards for EAC Partner States

Input data	Kenya	Tanzania	Uganda
Subbase materials	Natural gravels Graded crushed stone	G25 G45	Natural gravel
Base materials	Natural gravel Crushed stone	G60 G80 CRR CRS	Natural or crushed gravel Crushed stone
Wearing course		GW	

The characteristics of the above materials and the requirements to be met are further discussed in Chapter 5 on specifications for road and bridge works.

Recommendation

We recommend materials codes shown in Table 9 for unbound materials.

Table 9: Recommended codes for unbound materials

GW	Gravel wearing course for gravel roads or unpaved shoulders
G25	CBR minimum 25%
G45	CBR minimum 45%
G60	CBR minimum 60%
G80	CBR minimum 80% and includes crushed materials which less than 50% by mass of particles retained on the 5 mm sieve has a crushed face
CRR	Crushed rock made by crushing and screening of fresh quarried rock or clean, un-weathered boulders of minimum 0.3 m diameter. All particles should be crushed, no soil fines allowed.
CRS	Crushed stone made by crushing and screening of blasted rock, stones, boulders and oversize from natural gravel. Min 50% by mass of particles larger than 5 mm shall have at least one crushed face. Max 30% of material passing 5 mm can be soil fines.

b) Cemented and surfacing materials

The cemented materials discussed in the manuals for Kenya, Tanzania and Uganda and their recommended physical properties and requirements are shown in Table 10.

Table 10: Cemented Materials Specified in Design Standards of EAC Partner States

Input data	Kenya	Tanzania	Uganda
Subbase materials	Improved material	C1, CM	Cement treated material
Base materials	Cemented material Sand bitumen mix DBM Lean concrete Improved material	CM C1 C2 CRR CRS FBMIX BEMIX PM80 PM60 PM30 DBM40 DBM30 LAMBS	Bitumen macadam Cement treated gravel
Surfacing materials	Surface dressing Asphalt concrete	ST AC20 AC14 AC10	Surface dressing Asphalt concrete

Kenya manual recommends unsuitable natural materials which do not meet the requirements to be improved by treatment with cement or lime. It also covers the use of cement stabilized natural gravels and coarse (clayey) sands, sand bitumen mixes, dense bitumen macadam, dense emulsion macadam, and lean concrete for base course. For surfacing materials it specifies surface dressing and asphalt concrete (type I and type II). Cemented materials described in the manuals for Tanzania and Uganda include all cemented natural or crushed materials where a stabilizer of cement or lime has been admixed and bituminous base courses. They include dense bituminous macadam, Large Aggregate Mix for Base Course (LAMPS) which are essentially the same as Dense Emulsion Macadam described in the Kenya design manual. Other materials include Penetration Macadam (PM), Foamed bitumen mix (FBMI) and Bitumen emulsion mix (BEMIX). For surfacing materials, the manuals recommend use of surface dressing, Otta seals, sand seal, combined seals, slurry seal, surface enrichment, and asphalt concrete (AC10, AC14, and AC20).

The above classes of cemented and surfacing materials and their characteristics as well as material requirements and type of stabilizer to use are discussed in details in Chapter 6.

It is worthy noting 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 roadbase layer of Hot Rolled Apshalt (HRA), with DBM or Dense Tarmacadam (DTM) as the reminder of the roadbase material.

Recommendation

We recommend materials codes shown in Table 11 for cemented and surfacing materials.

Table 11: Recommended bound materials codes

CM	Earthworks quality soils/gravel - UCS min. 0.5 MPa — modified material made from source materials of quality nominally as G7 - with modified requirements
C0.7	Subbase quality soils/grave. - UCS min. 0.7 MPa and is made from source materials of quality nominally as G25 - with modified requirements
C1.0	UCS min. 1 MPa and is made from source materials of quality nominally as G45 - with modified requirements
C1.5	UCS min. 1.5 MPa - used as subbase in concrete pavements and is made from source materials of quality nominally as CRS - with modified requirements
FBMIX	cold mixed material of foamed bitumen and aggregate
BEMIX	cold mixed material of bitumen emulsion and aggregate
CC	Cement concrete
PM80	penetration macadam where the main fraction of the aggregate has largest particle size of a 80 mm
PM60	penetration macadam where the main fraction of the aggregate has largest particle size of a 60 mm
PM30	penetration macadam where the main fraction of the aggregate has largest particle size of a 30 mm
DBM40	Hot mixed material of bitumen and aggregate having a maximum particle size of 40 mm
DBM30	Hot mixed material of bitumen and aggregate having a maximum particle size of 30 mm
LAMBS	Hot mixed material of bitumen and aggregate have large particle size as prescribed in the Specifications
AC10	Hot mixed asphalt concrete material of bitumen and aggregate having a maximum particle size of 10 mm
AC14	Hot mixed asphalt concrete material of bitumen and aggregate having a maximum particle size of 14 mm
AC20	Hot mixed asphalt concrete material of bitumen and aggregate having a

	maximum particle size of 20 mm
S1	Bituminous seal made with approved chipping of near similar particle size, placed in a single layer
S2	Bituminous seal made with approved chipping of near similar particle size, placed in two layers as specified

3.4.1.5 Bituminous Surfacings

This section reports on the design standards of surfacing layers made of bituminous materials which are used for new roads, or rehabilitation or maintenance of existing roads.

Priming

The design manual for Kenya recommended that all non-bituminous road bases should be primed preferably with medium curing cut-backs MC 30 and MC 70. The binders proposed for tack coat are Rapid Curing cut-backs (RC 250, 800, or 3000) or Medium Curing cut-backs (MC 250, 800 or 3000) or Quick Breaking Emulsions (A1 or K1).

Likewise, design manuals for Tanzania and Uganda also recommend the use of MC30 and MC70 for priming and that MC30 should be used unless excessive absorption into the surface is observed and requiring the heavier MC70. If there is a delay of more than one month before the bituminous surfacing is placed MC70 is recommended. It recommend the spray rates of prime to be determined on site as required and if there will be temporary passage of traffic or a risk of the prime being picked up on tyres when applying next layer the manual recommend the use of crusher dust or a suitable sand seal.

Recommendation

We recommend the use of MC30 for priming. If excessive absorption into the surface is observed a heavier MC70 should be used.

Surface Dressing

a) Single surface dressing

Kenya design manual recommends the use of single surface dressing to be limited in dry areas only and it should be for resealing purposes. The chipping sizes recommended is 10 and 6 mm with spray rates of bitumen related to Average Least Dimension (ALD).

The design manuals for Tanzania and Uganda recommend single surface dressing for maintenance resealing works. Bitumen spray rates for single surface dressing and reseals is influenced by the Average Least Dimension (ALD) of the aggregates and site conditions. Aggregate sizes used are 14 and 10 mm. Aggregates requirements for surface dressing include gradation, flakiness index, and TFV both dry and wet.

b) Double surface dressing

The design manual for Kenya recommends double surface dressing on new roads and the second seal chippings are to be half the size of first seal chippings. The manual also recommends triple seal in areas where traffic is heavy. The bitumen spray rate is also related to ALD.

Likewise, double surface dressing is recommended for new roads and rehabilitation (maintenance) in Tanzania and Uganda and the second seal chippings are to be half the size of first seal chippings. The bitumen spray rates are also related to ALD and site conditions, and the above aggregate requirements must be observed. Nominal aggregate sizes are 10 mm (2^{nd} layer) and 20 mm (first layer) as well as 7 and 14 mm for fine surfacing type.

c) Binder type

Design manual for Tanzania recommends the use of penetration grade bitumen type 80/100 or 150/200 for surface dressing unless site conditions require the use of other grades of bitumen. MC 3000 cutbacks are only recommended for surface dressing operation in cold conditions at temperatures below 15^{o} C. In Uganda, however, the main types of binder that can be used for surface dressing for traffic up to 6,000 vehicles per day are:

- Cationic emulsion KI-70
- Medium curing cutback MC 3000 and
- Penetration grade bitumen 80/100

Recommendation

We recommend the use of single surface dressing for maintenance resealing works and aggregate sizes of 14 and 10 mm for the purpose.

On the other hand, we propose the use of double surface dressing is for new roads and for maintenance, and the second seal chippings to be half the size of first seal chippings. Nominal aggregate sizes should be 10 mm (2^{nd} layer) and 20 mm (first layer) as well as 7 and 14 mm for fine surfacing type.

The main types of binder that can be used for surface dressing are Cationic Emulsion KI-70 Medium Curing Cutback MC 3000 and Penetration Grade Bitumen 80/100.

Otta Seals

Tanzania and Uganda design manuals defined Otta Seal as a sprayed bituminous surfacing using graded aggregates ranging from natural gravel to crushed rock instead of the single sized crushed chipping used in surface dressing. They recommended Otta Seal to be used for new construction as well as maintenance purposes. The binder for Otta Seal range from MC800 cutback bitumen to 150/200 penetration grade bitumen and excluded the use of 80/100.

We recommend that:

Otta Seal should be used for new construction as well as maintenance purposes. The binder for Otta Seal should range from MC800 cutback bitumen to 150/200 penetration grade bitumen.

Sand Seals

Kenya design manual recommends sand seal to be used on low traffic roads and can be used in combination with surface dressing. The binders reported for use in sand seals include emulsion K1-60 and medium curing cut-backs MC 800, MC3000 or 800/1400. Sand Seal is also recommended for use in Tanzania and Uganda as a permanent bituminous surfacing on low traffic roads when constructed in two layers. It is also recommended for use as a maintenance remedy on existing surface treated roads. The binder recommended for Sand Seal is MC3000.

Recommendation

Sand Seal is recommended for use as a permanent bituminous surfacing on low traffic roads when constructed in two layers. It is also recommended for use as a maintenance remedy on existing surface treated roads. The binder recommended for Sand Seal is MC3000.

Slurry Seals

In Kenya, slurry seals are used as a surfacing for new roads and are to be applied directly on a primed base and also as a final seal on surface dressing. They are recommended for restoration of chippings and waterproofing the surface. Two layers of equal thickness can be applied and the binder recommended is bitumen emulsion. Slurry seal is also reported in the design manuals for Tanzania and Uganda and is used as a maintenance remedy for resealing to arrest loss of chipping in existing surface dressing and for restoration of surface texture. It can be used in new construction as a grout seal following a single surface dressing or in multiple layers directly on the base course on low traffic roads. The binder for slurry seal is bitumen emulsion.

Recommendation

We recommend the use of slurry for resealing works to arrest loss of chipping in existing surface dressing and for restoration of surface texture. It can be used in new construction as a grout seal following a single surface dressing or in multiple layers directly on the base course on low traffic roads. The binder for slurry seal is bitumen emulsion.

Asphalt Concrete

The design manual for Kenya proposes Asphalt Concrete Type I (High Stability) to be placed in a thin layer (50 mm or less) on only rigid or semi-rigid pavements or in a thick layer (minimum 75 mm) on a flexible pavements. It also recommended Asphalt Concrete Type II (Flexible) to be placed in a thin layer (maximum 50 mm). The materials requirements, traffic and use limitations and construction procedures concerning asphalt concrete (Type I and II) are also provided. Pen grade 60/70 binders are used for wearing course in mix type I and 80/100 in type II. The void ratio proposed is 3 – 5% for type I and 3 -8 in type II.

In Tanzania and Uganda, asphalt concrete (AC) is recommended for use on severely loaded areas such as climbing lanes, approaches to major junctions, all major town roads and areas where traffic is channelled or slow moving. The proposed air voids content is 3% and three types of mix; AC20, AC14 and AC10; are recommended for use.

Recommendation

We recommend the use of Asphalt concrete (AC) on all major roads.

Three types of AC mix; AC20, AC14 and AC10; are hereby recommended for use.

3.4.1.6 Pavement Design – New Roads

a) Input Data for Structural Design of the Pavement

A review of the design manual revealed that the same input data is used by design manuals for Kenya, Tanzania and Uganda. However, the major differences between the manuals include:

- In Kenya, climate consideration is not directly linked to the categories of structures while in Tanzania and Uganda, categories of structures are provided for each climatic zone.
- Pavement materials are assigned material codes for the case of Tanzania.
- Unlike the other design manuals, Tanzania design manual recommends the substitution of subbase materials without change in the thickness of the layer as follows
    - C1 can be replaced by C2
    - CM can be replaced by C1, or C2
    - G45 can be replaced by CM, C1, G60, G80, or CRS
    - G25 can be replaced by CM, C1, G45, G60, G80 or CRS
- Other differences are as noted in the previous sections concerning subgrade strength, traffic data and classes.
- Further, differences in material requirements and testing methods are discussed in Chapter 5 of Thematic Area 1 Working Paper.

b) Design Catalogue

Table 12 presents a summary of catalogues of pavement structures covered by the design manuals for Kenya, Tanzania and Uganda. While cement treatment is a proven method for improving the strength and durability of soils and aggregates, cement hydration causes shrinkage strains in the cemented base layer that can lead to reflection cracking in asphalt surfaces. Cracking may then cause increased pavement roughness and lead to poor ride quality. It is therefore of utmost importance to investigate the use and classification of cemented base layers and evaluate the relative impact of cement content on the durability of the cemented layer and the development of cracking in asphalt pavements constructed using cemented layers.

The use of properly designed and constructed cement-stabilized bases can actually reduce the occurrence of failure-related cracking. Fatigue cracking is decreased because the stiff, stabilized base reduces vertical deflection and tensile strain in the asphalt surface. Base failures are decreased because cement stabilization helps keep moisture out of the base and improves base material performance in saturated or freezing conditions. In addition, subgrade failure is decreased because cement stabilized bases spread traffic loads over wide areas and can span weak subgrade locations. However, cement-stabilized bases can also be the source of shrinkage cracks in the stabilized base layer, which can reflect through the asphalt surface. Thankfully, there are a number of measures that can be taken to minimize the chance that cracks will occur in a cement-stabilized material. These techniques include selecting the proper materials, optimizing mix designs, following proper construction, compaction, and curing practices, providing a stress relief layer in the pavement structure, delaying final surface paving, and microcracking. A well designed, constructed, and properly maintained cement-stabilized base will normally outlast several asphalt overlays, providing decades of low maintenance service.

There are a number of preventative measures that can be taken to minimize the chance that wide cracks will occur in a cement-stabilized material. One of the measures is the use of admixtures. Various admixtures have been investigated for reducing the shrinkage potential of soil-cement.

Table 12: Summary of Catalogues of Pavement Structures

Country	Design standard	Climatic region	Pavement Structure				Remark
			Surface	Base	Subbase	Improved layer/subgrade	
Burundi							
Kenya	MoTC (1987)	-	Double and triple surface dressings	Natural gravel	Natural material	S1 to S6	Covers T1 to T5 load classes
			Double surface dressing	Cemented material Lime improved material	Natural material		
			Double surface dressing	Cement or lime improved material	Cement or lime improved material		
			Hot mix asphalt concrete	Cement gravel	Cement or lime improved material		
			Hot mix asphalt concrete	Cemented gravel	Cement or lime improved material		
			Double and triple surface dressings	Crushed stone	Natural gravel		
			Double surface dressing	Crushed stone	Cement or lime improved material		
			Hot mix asphalt concrete	Dense bitumen macadam	Crushed stone		
			Hot mix asphalt concrete	Lean concrete	Cement or lime improved material		
			Hot mix asphalt concrete	Lean concrete	Crushed stone		
Rwanda	Uses AASHTO design method						
Tanzania (Mainland and Zanzibar)	MoW (1999)	Wet	Surface treatment	Crushed stone Crushed rock	Natural gravel Lime/cement modified material	CBR ≥ 15%	For TLC –H class
			Hot mix asphalt concrete	Crushed rock	Lime/cement modified material Cemented material		
			Surface treatment	Natural gravel Crushed stone	Natural gravel Lime/cement modified material		For general TLC
			Hot mix asphalt concrete	Crushed rock	Cemented material		
		Dry/moderate	Surface treatment	Crushed stone Crushed rock	Natural gravel Lime/cement modified material		For TLC –H class
			Hot mix asphalt concrete	Crushed rock	Cemented material		
			Surface treatment	Natural gravel Crushed stone Crushed rock	Natural gravel Cemented material		For general TLC
			Hot mix asphalt concrete	Crushed rock	Cemented material		

Table 12: Summary of Catalogues of Pavement Structures (Cont.)

Country	Design standard	Climatic region	Pavement Structure				Remark
			Surface	**Base**	**Subbase**	**Improved layer/subgrade**	
Tanzania (Mainland and Zanzibar)	MoW (1999)	Wet & dry/moderate	Surface treatment	Cemented material	Lime/cement modified material Cemented material		For TLC –H class
			Hot mix asphalt concrete	Cemented material	Lime/cement modified material Cemented material		
			Surface treatment	Lime/cement modified material Cemented material	Natural gravel Lime/cement modified material Cemented material		For general TLC
			Hot mix asphalt concrete	Cemented material	Cemented material		
		Wet & dry/moderate	Surface treatment	LAMBS, DBM, FBMIX, BEMIX	Natural gravel		
			Hot mix asphalt concrete	LAMBS, DBM	Lime/cement modified material		
		Wet & dry/moderate	Surface treatment	PM	Natural gravel Lime/cement modified material		
			Hot mix asphalt concrete	PM	Lime/cement modified material Cemented material		
Uganda	MoWT (2010)	Dry	Surface dressing or hot mix asphalt	Granular base	Granular subbase	Selected layer	Covers T1 to T8 classes
			Surface dressing or hot mix asphalt	Granular base	Cemented upper subbase Cemented subbase	Selected layer	
			Surface dressing	Cemented base	Cemented subbase	Selected layer	
			Surface dressing or hot mix asphalt	Bituminous base	Granular subbase	Selected layer	
			Surface dressing or hot mix asphalt	Bituminous base	Cemented subbase	Selected layer	
		Wet	Surface dressing or hot mix asphalt	Granular base	Granular subbase	Selected layer	
			Surface dressing or hot mix asphalt	Granular base	Cemented upper subbase Cemented subbase	Selected layer	
			Surface dressing or hot mix asphalt	Cemented base	Cemented subbase	Selected layer	
			Hot mix asphalt	Bituminous base	Granular subbase	Selected layer	
			Hot mix asphalt	Bituminous base	Cemented subbase	Selected layer	

Among these are shrinkage-compensating cement, gypsum, water reducers, fly ash, and ground granulated blast-furnace slag. Admixtures often reduce water demand, aid in the mixing process, extend mixing time, and for many granular soils, provide a filler material that can effectively reduce the need for excess cement.

Recommendation

The performance of cemented base layers and crushed rock base layers need to be reviewed in relation to the field performance of hot mix asphalt and surface treated pavements. Also, consideration should be given to the use of admixtures that have proven to reduce the chance that wide cracks will occur in a cement-stabilized material.

3.4.1.7 Gravel Roads Design

Design requirements

Kenya design manual recommends gravel road pavements where reported initial daily number of commercial vehicles is greater than 150 (up to 500) for gravel road class 1 and less than 150 for gravel road class 2 while in Tanzania gravel road pavements are to be designed for roads where AADT is less than 300. Likewise, design manual for Uganda recommends gravel road pavements for roads where design traffic flow Annual Average Daily Traffic (AADT) is less than 300 at the time of construction.

This variation in traffic volume for gravel roads translates to different levels of traffic at which gravel roads are cost effective. Gravel roads are costly, however, paved roads also are costly to maintain. Maintenance costs for paved and unpaved roads are rising. There is a need optimise those costs to best serve the public.

Recommendation

We propose that a decision to keep a road as a gravel road or upgrade to a paved surface should be based on the comparison of the cost of maintaining a gravel road with the cost of upgrading to and maintaining a paved surface. This analysis can be modified to address local conditions as well, and may be used as a tool to assist in making decisions about upgrading a gravel road to a paved surface.

Improved subgrade

Depending on the CBR design of the subgrade, improved subgrade layers are usually constructed on which the gravel wearing course is placed. The design manual for Kenya recommends the use of improved subgrade materials which meet the minimum recommended subgrade strength class as a foundation for a wearing course and no gravel wearing course on S6 subgrade class. For the case of Tanzania, the design manual reported that materials for improved subgrade layers and fill should meet requirements for class G15 and G7 as well as G3 and dump rock (DR). On the other hand, the manual for Uganda recommends two materials, G15 and G7 for improved subgrades. Thus, the proposed improved layers are specified in terms of the subgrade classes.

Recommendation

The materials for improved subgrade layers for gravel roads should meet requirements for class G15 and G7.

Gravel wearing course (GW)

Two conflicting requirements are to be met by materials for gravel wearing course:

- The need for a sufficient cohesion to bind the particles and prevent the surface from ravelling and becoming corrugated in dry seasons
- Limiting the amount of fines and the plasticity so as to avoid the occurrence of a slippery surface in wet weather

Kenya design manual recommends gravel wearing course materials grading envelope and plasticity requirements. Also, a minimum CBR of 20 after 4 days soak and for very low traffic, less than 15 commercial vehicles per day, a minimum CBR of 15 is acceptable. Since excessive oversize material in the gravel wearing course affects the riding quality in service and makes effective shaping of the surface difficult at the time of maintenance, materials for fully engineered gravel wearing course in Tanzania and Uganda are required to meet the requirements for CBR, gradation, shrinkage product, grading coefficient, and field density material properties. Thus, for major gravel roads in Tanzania and Uganda, a minimum CBR at 95% of MDD (BS-Heavy compaction) in wet climatic zones is proposed to be 25% after 4 days soak and in moderate or dry climatic zones 25% at OMC. The percentage passing 37.5 mm should be min. 95 and the shrinkage product to be in the range of 120 – 400 but in built up areas a maximum shrinkage product of 270 is desirable to reduce dust problem. The recommended Grading Coefficient is in the range of 16 – 34. For minor gravel roads, the CBR for major gravel roads is reported to be reduced to 15% and the material standards should be aimed for wherever it is economically possible.

Recommendation

The governing material properties for fully engineered gravel wearing course are CBR, gradation, shrinkage product, grading coefficient, and field density and the gravel wearing course should meet the requirements for the same properties.

Gravel Thickness Design Method

Several factors are known to affect gravel road surface performance during its life span. Some of these factors are axle load, which is referred to as the 80 kN equivalent single axle load (ESAL); cover aggregate characteristics; surface/subsurface drainage; subgrade properties; resilient modulus; and moisture change, to name a few. The ESAL factor is considered vital to gravel road thickness design and must be calculated. The design manual for Kenya outlines a gravel thickness design approach which involves the determination of minimum thickness necessary to avoid excessive strain in subgrade (D1), the extra thickness needed to compensate for the gravel loss under traffic during the period (N-periods in years) between regravelling operations (D2), and the total gravel thickness required by adding the two thicknesses (D1 + ND2). Gravel loss estimation model used is based on the work of the TRRL Laboratory Report 673 entitled “The Kenya Road Transport Cost Study – Research on Road Deterioration”. The model input variables are the total traffic volume in the first year in both directions, the average annual rainfall, the total rise and fall as a percentage of the road length, and f-factors of different gravel material types.

The same approach is reported in the gravel roads design manual for Uganda. The manual, however, documents other models that can be used as an aid to the planning of regravelling of unpaved roads. The models are the World Bank Model (HDM-4) and Uganda model. Additionally, like the design manual for Tanzania, the manual for Uganda provides different gravel thicknesses in relation to the strength of subgrade and improved layers. It may be argued that the models used to estimate gravel material loss, as documented in design manuals for Kenya and Uganda, provide a theoretically sound design approach.

In recent years there has been a growing awareness of the need to improve knowledge of the relationships between the construction and maintenance standards, the climate, the traffic, and vehicle operating costs on unsurfaced roads. The need arises because of the desire of highway planners to improve the quality of investment decisions in the rural road sector in developing countries. A considerable amount of unrecorded local knowledge about the interaction between many of these factors exists in developing countries, but quantified data is very scarce. One of these is the growing interest among highway planners and engineers in quantifying, for a particular climatic zone, the relationships between traffic volume, construction standards, rate of deterioration, and maintenance. Knowledge of these relationships is required if cost-effect decisions are to be made about the construction of unsurfaced roads, or economic maintenance strategies are to be devised.

Recommendation

We propose that EAC Partner States should consider developing further the wearing gravel thickness design models by incorporating other critical factors that are not addressed by the current design models.

3.4.1.8 Pavement Rehabilitation Design

An increase in economic activity in EAC has led to major expansion and improvement of the road network in the region. A good number of these roads are reaching a condition which warrants improvements of their riding quality and strengthening of the pavement structure. Good practices have shown that if a deficient pavement is rehabilitated timely, the costs involved often amount to a mere fraction of the cost of reconstruction. This is achieved by using the remaining structural strength and behaviour of the existing pavement in the rehabilitation design for the road. Thus, rehabilitation design should include procedures that allow for the quantification and evaluation of the behaviour of the existing pavement structure.

The design manuals for Tanzania and Uganda allow for the pavement evaluation which is used to determine pavement condition. The outcome of pavement evaluation forms the basis to carry out a rehabilitation design using appropriate design methods so as to identify appropriate rehabilitation measures. The design methods covered in the manual for Tanzania include maximum deflection method, structural number method and mechanistic method while the rehabilitation design manual for Uganda covers only two methods; maximum deflection method and the structural number method. A suitable design method can only be chosen depending on pavement type, condition, and base course material types and main mode of distress as shown in Table 13.

Table 13: Selection of appropriate design method

Base course material type	Main mode of distress	Rehabilitation design method		
		Maximum deflection	Structural number	Mechanistic
Granular or lightly cemented	Cracking	NA	√	√
	Deformation	√*		
Cemented or bituminous	Cracking, deformation, or both	NA	√	√
*The maximum deflection method only applies for traffic load class TLC 10 or lower and where deformation originates from the subgrade				

Source: Tanzania PMDM (MoW, 1999)

Maximum deflection method is based on empirically derived relationships between pavement performance and surface deflection under loading. This method is recommended if the following conditions prevail:

- Distresses are originating from the subgrade, and
- Granular base course or lightly cemented and not cement stabilized, and
- There is remaining structural life, and
- Future design traffic less than 10 million. E80

Structural number method is based on empirical correlation between tested material properties and expected pavement performance. Laboratory tests (CBR) and in-situ measurements (DCP) are required to determine material strength, expressed as the material coefficient.

Mechanistic method is theoretically derived through linear elastic theory. The manual recommends the use of the South African mechanistic design which requires the following information about the existing pavement to be gathered:

- Pavement type
- Pavement state
- Layer state
- Layer thickness
- Layer moduli

Recommendation

EAC member countries should consider adopting the three pavement rehabilitation design methods; maximum deflection method, structural number method and mechanistic method.

3.4.1.9 Influence of Design Parameters on Flexible Pavement Performance

There are many pavement design parameters such as pavement layer thicknesses and moduli, axle load, tyre pressure, temperature and moisture. Their interaction is considered to influence the performance of pavements. Since the structural design of a pavement system must be done with a clear understanding of the factors that affect the life and serviceability of the pavement, the sensitivity analysis study of these parameters on the pavement performance is worthy undertaking with a view to improving existing design methods and hence enhancing pavement life and serviceability. In order to do this, this study part can draw heavily on work already completed by Mfinanga and Salehe (2008). The latter study was carried out to determine the influence of some of the design parameters on pavement performance. The study focused on analysing pavement structures that are widely used in Tanzania and provided for in the current pavement design manual. The aim of the study was to improve pavement life through design and thus, the analysis was carried out using state-of-the-art pavement performance models; namely mechanistic-empirical models.

The South African pavement performance model was used in the study as it is the one suggested for use in Tanzania as the nearest available to local verification criteria. The pavement responses that provide input to the model were calculated using Elastic Layer System 5 (ELSYM5) program. The design parameters used were those that could be used in the ELSYM5 program used to determine pavement responses and the South African pavement performance model. This approach, therefore, has the same limitations as those of the multi-layer elastic system analysis as well as those of the adopted transfer functions used in South Africa.

Pavement structures that are commonly used in Tanzania were selected for the study and the design parameters were those that can easily be simulated with the available models for predicting pavement performance using the mechanistic-empirical approach. The practical range of values for each design parameter from available literature and the Tanzania Ministry of Works reports were also established for the study. The sensitivity analysis was carried out by varying individual input parameters within the known practical range of values while keeping other parameters constant and observing changes in pavement service lives so as to enable the evaluation and determination of the most sensitive and or influential design parameters in relation to pavement performance.

Pavement Types

Three pavement types that are recommended by the Tanzania design manual were studied. Pavement type I consists of a mean thickness of 65 mm asphalt concrete over 150 mm crushed stone base and 225 mm granular subbase on top of subgrade soil. Type II consists of a mean thickness of 65 mm asphalt concrete over 150 mm bituminous base and 225 mm granular subbase on top of subgrade soil, and Type III is composed of a mean thickness of 65 mm asphalt concrete over 150 mm cemented base and 225 mm granular subbase on top of subgrade soil. The mean layers' stiffnesses were 5500 MPa for asphalt concrete, 350 MPa for crushed stone base, 1750 MPa for bituminous base, 5000 MPa for cemented base, 110 MPa for granular subbase and 70 MPa for the subgrade.

Data collected

Tables 14 to 16 show data types that were acquired from the literature, in particular the Tanzania pavement design manual, the South Africa mechanistic design Method reports (SARB, 1985;

Jordaan, 1994; Theyse et al., 1996), and relevant information from the Ministry of Works in Tanzania. Vehicles axle configurations that are commonly found in the country were determined from the sections being monitored by the Ministry and corresponding equivalent load factors were established as shown in Table 17.

Table 14: Range of design input parameters' values used in the analysis of pavement type I

Input parameter	**Variation range**										**Mean**	**STD**	**COV (%)**
Tyre pressure (kPa)	500	550	600	650	700	750	800	850	900	950	725	151.4	20.88
Axle wheel load (kN)	25	30	35	40	45	50	55				40	10.8	27.00
AC modulus (MPa)	1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	5,500	3,028	55.05
Base modulus (MPa)	100	200	300	400	500	600					350	187.1	53.45
Subbase modulus (MPa)	20	50	80	110	140	170	200				110	64.81	58.92
Subgrade modulus (MPa)	20	45	70	95	120						70	39.53	56.47
AC thickness (mm)	30	40	50	60	70	80	90	100			65	24.49	37.68
Base thickness (mm)	100	125	150	175	200						150	39.53	26.35
Subbase thickness (mm)	150	175	200	225	250	275	300				225	54.01	24.00

Source: Mfinanga and Salehe (2008); STD = standard deviation, Cov = coefficient of variation

Table 15: Range of design input parameters' values used in the analysis of pavement type II

Input parameter	**Variation range**										**Mean**	**STD**	**COV (%)**
Tyre pressure (kPa)	500	550	600	650	700	750	800	850	900	950	725	151.4	20.88
Axle wheel load (kN)	25	30	35	40	45	50	55				40	10.8	27.00
AC modulus (MPa)	1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	5,500	3,028	55.05
Base modulus (MPa)	1,000	1,300	1,600	1,900	2,200	2,500					1,750	561.2	32.07
Subbase modulus (MPa)	20	50	80	110	140	170	200				110	64.81	58.92
Subgrade modulus (MPa)	20	45	70	95	120						70	39.53	56.47
AC thickness (mm)	30	40	50	60	70	80	90	100			65	24.49	37.68
Base thickness (mm)	100	125	150	175	200						150	39.53	26.35
Subbase thickness (mm)	150	175	200	225	250	275	300				225	54.01	24.00

Source: Mfinanga and Salehe (2008)

Table 16: Range of design input parameters' values used in the analysis of pavement type III

Input parameter	Variation range										Mean	STD	COV (%)
Tyre pressure (kPa)	500	550	600	650	700	750	800	850	900	950	725	151.4	20.88
Axle wheel load (kN)	25	30	35	40	45	50	55				40	10.8	27.00
AC modulus (MPa)	1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	5,500	3,028	55.05
Base modulus (MPa)	3,000	4,000	5,000	6,000	7,000						5,000	1,581	31.62
Subbase Modulus (MPa)	20	50	80	110	140	170	200				110	64.81	58.92
Subgrade modulus (MPa)	20	45	70	95	120						70	39.53	56.47
AC thickness (mm)	30	40	50	60	70	80	90	100			65	24.49	37.68
Base thickness (mm)	100	125	150	175	200						150	39.53	26.35
Subbase thickness (mm)	150	175	200	225	250	275	300				225	54.01	24.00

Source: Mfinanga and Salehe (2008)

Table 17: Axle configurations, traffic volume proportions and vehicles load factors

Vehicle Category	Vehicle Type	No. of Axles	Axle Configuration	Vehicle Weight (tonnes)	% of AADT	Vehicle Load Factor
Medium Goods Vehicle	Rigid	2	1.1	16	2.5	2.89
	Rigid	2	1.2	18	12.5	
Heavy Goods Vehicle	Rigid	3	1.21	23	1	4.42
	Rigid	3	1.22	26	2	
	Articulated	3	1.2-2	28	1	
Very Heavy Goods Vehicles	Rigid	4	1.2+2.2	37	0.5	7.33
	Articulated	4	1.2-22	36	1	
	Articulated	5	1.22-22	42	1	
	Articulated	5	1.2-222	42	1	
	Rigid	6	1.22+2.22	53	3	
	Articulated	6	1.22-222	50	5	
Buses	Heavy bus	2	1.2	18	14.5	3.43
Other Vehicles	Cars/pickup			< 2	55	0

Source: Mfinanga and Salehe (2008)

Calculation Results

As it can be noted from Table 18, the parameters with greatest influence on fatigue type I pavement performance are the tyre pressure, base modulus, AC thickness and axle load while the subbase thickness, subgrade modulus, and subbase modulus were least sensitive in this regard. The input parameters that were noted to influence rutting greatly were axle load, subbase, base and AC thicknesses. On the contrary, tyre pressure, AC modulus and base modulus appeared to have the least effect on rutting. It can be concluded that tyre pressure, base modulus, AC thickness and axle load are the design variables that most influence the performance of pavement type I.

Table 18: Ranking of sensitivity of input parameters for pavement type I

Cracking failure criterion		Rutting failure criterion		Ranking according to critical failure criterion i.e. Cracking
Input parameter	Impact elasticity	Input parameter	Impact elasticity	
Tyre Pressure	4.26	Axle Load	7.78	Tyre Pressure
Base Modulus	2.31	Subbase Thickness	5.57	Base Modulus
AC Thickness	2.11	Base Thickness	4.81	AC Thickness
Axle Load	1.77	AC Thickness	4.24	Axle Load
Base Thickness	1.26	Subgrade modulus	3.10	Base Thickness
AC Modulus	1.24	Subbase Modulus	3.06	AC Modulus
Subbase Modulus	0.63	Base Modulus	1.86	Subbase Modulus
Subgrade Modulus	0.13	AC Modulus	1.32	Subgrade Modulus
Subbase Thickness	0.12	Tyre Pressure	0.70	Subbase Thickness

Source: Mfinanga and Salehe (2008)

On type II pavement, it can be noted from Table 19 that the highest influence on fatigue pavement performance were the axle load, base thickness, AC thickness and base modulus while subbase thickness, subgrade modulus and asphalt concrete modulus had the least effect on fatigue. Rutting were noted to be most influenced by axle load, base thickness, subbase thickness and subbase modulus while tyre pressure, AC modulus and base modulus had the least effect. It can be concluded that axle load, base thickness, AC thickness and base modulus are the design variables that most influence the performance of pavement type II.

Table 19: Ranking of sensitivity of input parameters for pavement type II

Cracking failure criterion		Rutting failure criterion		Ranking according to critical failure criterion i.e. cracking
Input parameter	Impact elasticity	Input parameter	Impact elasticity	
Axle load	3.67	Axle Load	7.97	Axle load
Base thickness	3.51	Base Thickness	6.07	Base thickness
AC thickness	2.13	Subbase Thickness	4.68	AC thickness
Base modulus	2.08	Subbase Modulus	4.01	Base modulus
Tyre pressure	1.04	AC Thickness	3.75	Tyre pressure
Subbase modulus	0.89	Subgrade modulus	3.04	Subbase modulus
AC modulus	0.64	Base Modulus	1.70	AC modulus
Subgrade modulus	0.49	AC Modulus	1.41	Subgrade modulus
Subbase thickness	0.18	Tyre Pressure	0.29	Subbase thickness

Source: Mfinanga and Salehe (2008)

The design parameters of base thickness, axle load, base modulus and AC thickness were noted to have the greatest effect on fatigue cracking behaviour of pavement type III (Table 20) while subbase thickness, subgrade modulus and subbase modulus had the least effect on service life in terms of fatigue cracking. On the other hand, axle load, base thickness, subbase modulus, AC thickness and subbase thickness influenced heavily rutting of type III pavement whereas tyre pressure, AC modulus and base modulus show the least effect. It can be concluded that base thickness, axle load, base modulus and AC thickness are the most influential parameters on performance of pavement type III.

Table20: Ranking of sensitivity of input parameters for pavement type III

Cracking failure criterion		Rutting failure criterion		Ranking according to critical failure criterion i.e. cracking
Input parameter	Impact elasticity	Input parameter	Impact elasticity	
Base thickness	4.46	Axle Load	8.04	Base thickness
Axle load	4.32	Base Thickness	6.62	Axle load
Base modulus	2.92	Subbase Modulus	4.14	Base modulus
AC thickness	2.33	AC Thickness	3.79	AC thickness
Tyre pressure	1.33	Subbase Thickness	3.76	Tyre pressure
AC modulus	0.95	Subgrade modulus	3.01	AC modulus
Subbase modulus	0.83	Base Modulus	2.83	Subbase modulus
Subgrade modulus	0.72	AC Modulus	1.71	Subgrade modulus
Subbase thickness	0.17	Tyre Pressure	0.15	Subbase thickness

Source: Mfinanga and Salehe (2008)

Basing on the analysis of impact elasticity values, it can be noted that the design parameters were more sensitive to service life to failure through rutting than fatigue cracking as shown in Figure 5. Thus, the most effective design parameters affecting pavement service life considering all types of pavements are axle load, base thickness, AC thickness and base modulus while the least effective are subbase modulus, subgrade modulus and subbase thickness.

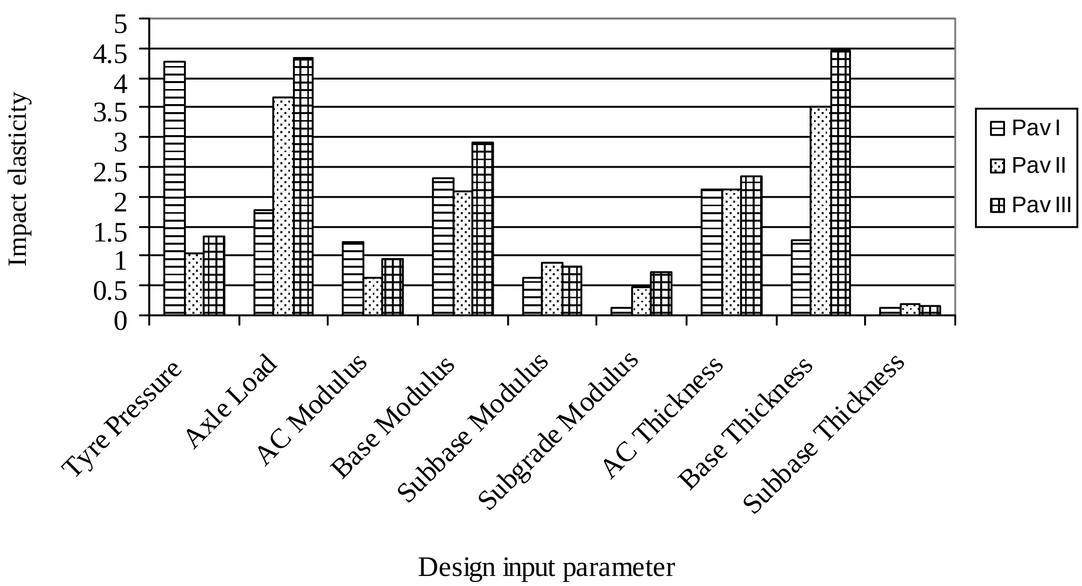


Figure 5: Impact elasticity against design variables for the fatigue failure criterion (Source: Mfinanga and Salehe (2008)

Recommendation

It is our considered view that EAC Partner States should consider and start adopting an analytical pavement design method in the region. The method should be based on experience, theory of pavement structural and material behaviour. It should take into account of local conditions of climate, traffic, available local materials, and other factors. With such a method it will be possible to carry out structural analysis of pavements and the prediction of their performance from the calculated parameters. However, in order for such a method to be successful attention should be paid to the methods of characterisation of the pavement layers and subgrade to meet the requirements of a theoretical model, calculation of parameters considered to have a primary influence on selected aspects of pavement performance and utilisation of these parameters in performance models to evaluate the structural adequacy of the pavement under consideration.

3.4.2 Rigid Pavements

The most widely used procedures for design of concrete pavements are the AASHTO guide procedure for the Design of Pavement Structures published in 1986 and 1993 by the American Association of State Highway and Transportation Officials (AASHTO) and the Portland Cement Association (PCA) procedure. Regardless of the method, the key input parameters in any concrete pavement design procedure are traffic over a design period, subgrade, environment, concrete material properties, base properties, performance criteria, and design reliability. These key input parameters are outlined in the following subsections.

3.4.2.1 AASHTO Design Procedure

Key Parameters in the AASHTO Rigid Pavement Design method are discussed below briefly.

Traffic

The number of heavy truck axle loads anticipated over the design life must be estimated from current truck traffic weights and volumes and growth projections. In the AASHTO methodology, the anticipated spectrum of truck loads over the design period is expressed in terms of an equivalent number of 18-kip single axle loads, computed using load equivalency factors that relate the damage done by a given axle type and weight to the damage done by this standard axle. The 80 kN equivalency for single axle loads, also known as an ESAL, is a widely accepted standard axle in the U.S. and around the world. A standard legal axle load limit has generally been imposed for highway travel, hence maximum gear loads in highway design have not appreciably increased with time. For this reason, the effects of other vehicles have normally been accounted for in the design phase by the use of 18-kip single axle loads.

It is important to note that the 2002 Design Guide no longer utilise ESALs and rely instead upon load spectra. Load spectra are simply the weight distributions of various axle configurations. In fact, load spectra are used in the 1993 AASHTO method to compute ESALs.

Subgrade

The modulus of subgrade reaction of the foundation can be measured by plate bearing tests. Load is applied at a predetermined rate until a pressure of 10 psi is reached. The pressure is held constant until the deflection increases not more than 0.001 in. per minute for three consecutive minutes. The average of the three dial readings is averaged to determine the deflection. The modulus of subgrade reaction (k) is important in rigid pavement design because it defines both the subgrade and subbase support. Since the plate loading test is time-consuming and expensive, the k-value is usually estimated from correlations to simpler tests such as California Bearing Ratio (CBR) and R-value tests.

The subgrade and subbase support strengths varies over the course of a year. However, k-values do not have a great effect on required thickness of concrete pavements. Other methods, such as the PCA method, avoid the tedious method of considering seasonal variations in k-values by using normal summer or fall k-values for design purposes (PCA, 1984).

Climate

The climatic variables, particularly temperature variation, can greatly influence concrete behaviour. Daily and seasonal variations in temperature and moisture influence the behaviour of concrete pavements in many ways, including:

- Opening and closing of transverse joints in response to daily and seasonal variation in slab temperature, resulting in fluctuations in joint load transfer capability, which is the ability of each slab group to transfer wheel loads from one slab to the next.
- Upward and downward curling of the slab caused by daily cycling of the temperature gradient through the slab thickness. Permanent upward curling of the slab, which in some circumstances may occur during construction, as a result of the dissipation of a large temperature gradient that existed in the concrete while it cured.
- Upward warping of the slab caused by seasonal variation in the moisture gradient through the slab thickness. Erosion of base and foundation materials caused by inadequate drainage of excess water in the pavement structure, primarily from precipitation.
- Corrosion of dowel bars, steel reinforcement, or both, especially in coastal environments

Climatic effect is more adequately considered in thickness design in the 2002 Design Guide through a program that compiles much of the field and analytical studies into weather stations that allow the designer to triangulate the design location or to input its exact latitude and longitude from weather stations in major cities in the USA. The program is called the Enhanced Integrated Climate Model (EICM).

Concrete Properties

For the purpose of pavement thickness design, concrete is characterized by its flexural strength as well as its modulus of elasticity (E). Concrete flexural strength is usually characterized by the 28-day modulus of rupture from third point loading tests of beams or it may be estimated from compressive strengths. The flexural strength of concrete is a measure of the quality and durability of the concrete. A higher flexural strength of the concrete will most likely result in a lower concrete slab thickness. The modulus of elasticity can also be predicted from compressive strength.

Performance Indices

All pavement thickness design procedures incorporate performance criteria that define the end of the performance life of the pavement. In the current AASHTO methodology, the performance criterion is the loss of serviceability, which occurs as a result of accumulated damage caused by traffic load applications. The Portland Cement Association (PCA) procedure uses both fatigue cracking and erosion criteria.

Reliability

The reliability level for which a pavement is designed reflects the degree of risk of premature failure that the agency is willing to accept. Facilities of higher functional classes and higher traffic volumes warrant higher safety factors in design. In the AASHTO methodology, this margin of safety is provided by applying a reliability adjustment to the traffic ESAL input. The magnitude of the adjustment is a function of the overall standard deviation associated with the AASHTO model, which reflects error associated with the estimation of traffic and strength inputs and error associated with the quality of fit of the model to the data on which it is based (AASHTO, 1993). When reliability adjustments are made to the traffic input in this manner, AASHTO recommends that average values should be used for the material inputs.

Before the introduction of reliability concepts in pavement thickness procedures, the traditional approach to introducing a margin of safety into concrete pavement thickness design was to apply a safety factor to the concrete modulus of rupture. This approach is still used in the PCA procedure. The PCA procedure also accounts for uncertainty by the reduction of the concrete modulus of rupture by one coefficient of variation and increasing the traffic weights by a percentage depending upon the type of roadway.

3.4.2.2 PCA Method

The PCA rigid pavement design procedure evaluates a candidate pavement design with respect to two potential failure modes: fatigue and erosion. This M-E procedure was developed using the results of a finite element analysis of stresses induced in concrete pavements by joint, edge, and corner loading. The analysis took into consideration the degree of load transfer provided by dowels or aggregate interlock and the degree of edge support provided by a concrete shoulder (PCA, 1984). The PCA procedure, like the 1986-1993 AASHTO procedure, employs the composite k concept in which the design k is a function of the subgrade soil k, base thickness, and base type.

The fatigue analysis incorporates the assumption that approximately 6% of all truck loads will pass sufficiently close to the slab edge to produce a significant tensile stress. The erosion analysis quantifies the power with which a slab corner is deflected by a wheel load as a function of the slab thickness, foundation k-value, and estimated pressure at the slab-foundation interface (Hall, 2000). For each load level considered, the expected number of load repetitions over the design life is expressed as a percentage of allowable load repetitions of that load level with respect to both fatigue and erosion. An adequate thickness is one for which the sum of the contributions of all axle load levels to fatigue and erosion levels is less than 100 percent.

The latest version of PCA uses the load spectra analysis to calculate the bending stress in the concrete due to various axle loads and configurations. Load spectra analysis is more theoretically sound than ESAL analysis because fundamental stresses and strains are calculated and related to the performance of laboratory concrete fatigue beam tests. Load spectra analysis also allows for calculation of pavement stresses due to axle loads and configurations not originally considered in the AASHO Road Test. The limitations of the PCA guide include no ability to analyze widened lanes or different joint spacing and no consideration of load transfer across the shoulder-lane joint

(Packard, 1984). This is significant because widened lanes are an often used type of edge support, and also because differences in length between joints in concrete slabs are often encountered in a rigid pavement design.

3.4.2.3 Other Methods

Other concrete pavement design methods range from empirical adaptations of the AASHTO method to calibration and mechanistic-empirical extension of the AASHTO method and methods that combine mechanistic stress calculation with an empirical fatigue cracking model. Design catalogues have also been developed in several other countries.

3.4.2.4 Rigid Pavement Design Alternatives and Construction in EAC Partner States

Concrete pavement design and construction has not been extensively applied in the EAC region. They have mostly been used on a small scale for surfacing roads, e.g. in hilly areas in Tanzania and on climbing lanes in Kenya. The construction of concrete pavements in such areas focused mainly in solving the problems associated with high stresses due to lower speeds of heavy vehicles, which result in ruts formation and and other surface defects such as shoving, corrugations, and ravelling.

3.4.2.5 Economic Viability of Cement Concrete Pavements

In general life-cycle cost of an asset is the total cost of operating that asset over its life, including the initial costs of providing the asset and the costs of using the asset over its life (PIARC, 2000a). The costs are incurred by the agency responsible for the asset, the people using the asset and others influenced by the existence of the asset. The most obvious costs for a road are costs for planning, design, construction, maintenance, reparation and rehabilitation of the road. Other costs are those for the road user such as vehicle operating costs, accident costs, and the environment.

The AASHTO Guide for Design of Pavement Structures suggests two categories of costs to be considered in the life-cycle cost analysis of alternative pavement strategies with different design features; agency costs and user costs. Agency costs include initial construction costs, future construction or rehabilitation costs, maintenance costs recurring throughout the design period, salvage or residual value at the end of the design period, engineering and administrative costs, and traffic control costs. User costs include travel time, vehicle operation, accidents, discomfort, time delay and extra vehicle operating costs during resurfacing or major maintenance (AASHTO, 1994). In practice, however, life-cycle cost analysis is limited to the construction and maintenance costs of the road over a specified period of time (PIARC, 2000b). One of the reasons for this is that little information exists to enable comparative analysis of differences in road user costs on the alternative types of construction. Thus, what is commonly available has tended to concentrate on construction and maintenance costs.

Comparative analysis of cost-effectiveness of concrete pavements versus flexible pavements is complicated by the selection of design lives, discount rates (which can be made to show that either concrete or flexible pavements cost less), the stage construction strategy for some flexible pavements, and the definition of acceptable or desirable levels of serviceability of the pavement at some time in the future (Parry, 1985). In addition comparison of costs will also vary depending on the type of road to be built, the type of funds available to pay for it, and the available machinery and labour.

Packard (2005) noted that in California, the USA, maintenance costs vary considerably depending on the pavement facility, age and condition of pavement, agency policy, availability of funds, and many other factors. For instance, if the pavement is a major roadway, the comparative cost between asphalt and concrete (average annual maintenance costs for the service life of pavement) may be about two or three to one. For lower road classifications, road agencies often spend very little on concrete maintenance so the ratio of costs rises to much higher values, perhaps as high as eight to one. On the other hand, cost information provided by Mfinanga (2002) showed that the costs for maintaining asphalt concrete pavements was five times more than the costs for maintaining concrete pavement. In Canada, Smith et al. (2001) observed that the costs for maintaining asphaltic pavements were twelve (12) times more than the costs for maintaining concrete pavements.

Other economic benefits that may be accrued by the use of concrete pavements include:

- Reduced maintenance delays.
- Provides fuel savings for heavy vehicles as more energy and therefore more fuel is required to drive on deflected flexible pavements.
- Uses less non-renewable resources.

3.4.2.6 Special Advantages of Concrete Pavements for EAC

Some of the factors which are likely to place cement concrete construction in the EAC on a competitive edge include the availability of several indigenous cement manufacturing industries as opposed to the use of bitumen which relies on importation. Besides higher initial cost of concrete pavements, the reduced maintenance requirement over the design life may make this type of pavements more economical in the long term.

Another factor is the availability of equipment and human resource with intensive skills and knowledge learned in the building construction industry. This should not be misrepresented as to suggest that workers may be taken from a building site and set to lay pavements without some clear instructions from a trained supervisor. The hand laying procedure observed in Philippines was successful because the labourers were willing and skilful (Parry, 1985). Although the long term objective would be to use the more efficient equipment-based construction, such hand laying procedure evidences that cement concrete pavements of good quality can be laid using only very basic equipment. Other potential technical advantages of concrete pavements in tropical countries:

- In tropical climates where there is little variation in diurnal (being near the equator) and annual temperatures it may not be necessary to provide thermal reinforcement in the concrete slab.
- Unlike asphaltic pavements, concrete stiffness is not affected by temperature changes. Concrete pavement design methods evolved from experience in temperate regions are likely to be applicable in tropical countries.
- Concrete pavements are ideal in EAC where there is a problem of heavy vehicle overloading.
- One of the influences on the structural performance of concrete pavements is the strength of concrete and its coefficient of thermal expansion. Concrete made with limestone, which is widely found in tropical countries; tend to have low coefficients of thermal expansion. The use of limestone, however, should be limited to lightly traffic roads in wet climate because limestone is likely to be polished under heavy traffic.
- Maintenance costs of concrete pavements are relatively less than those of asphaltic pavements. This is an important advantage for a developing country where many asphaltic pavements are failing prematurely for lack of routine and periodic maintenance.

3.4.2.7 Conclusion and Recommendation

The discussion has indicated a number of issues which deserve special attention for successful and sustainable application of cement concrete pavement construction. A wide range of technical aspects of concrete pavement design have been covered. It has been pointed out that the design of concrete pavements is not just a matter of considering key design inputs which influence thickness design but also the selection, specification and constructability of features of concrete pavements. Also, it has been indicated that concrete roads have been used successful in some developing countries and there is great potential for their use in EAC.

Taking into account the special advantages of concrete pavement, we propose that concrete roads should be used on all heavily trafficked interurban and urban roads in the EAC region.

3.4.3. Effects of Super-Single or Wide-Based Tires on Pavements

Dual tires have traditionally provided the largest footprint to adequately distribute the axle load onto the pavement surface. As the name implies, wide-base single tires provide a wider footprint than conventional single tires and attempt to distribute the load over a contact area similar to that of standard dual tires. Al-Qadi et al. (2004) noted that, historically, wide-base tires have evolved considerably since the introduction of the first generation in the early 1980s. Over the years, wide-base tires have become increasingly wider than their predecessors, see Figure 6, which shows the general trend in wide-base technology (different patterns of dual and wide-base tires exist and are not shown in this figure for simplicity). The first generation of wide-base tires (385/65R22.5 and 425/65R22.5), often referred to as Super Singles, were introduced in the early 1980's. These tires ultimately proved to cause an increase in pavement contact stress and in turn generated even greater pavement damage. They were shown to clearly decrease the tire contact area and, therefore, increase the pavement contact stresses. The increase in vertical and lateral stresses induced by the Super Single tires significantly increased the likelihood of top-down cracking and near-surface rutting of asphalt pavements. The acceleration of the top-down crack failure mechanism is usually attributed to the high tensile strains and stresses induced at the top of the pavement layer by the tire.

Figure 6: Photography illustrating Super Singles and Wide-Based Tires

There have been a number of innovations in tire technology to address the evolving needs of the trucking industry for efficiency. Recently, a new generation of wide-base tires (NGWB) with greater tread widths has been introduced. This generation include the 445/50R22.5 and 455/55R22.5 in the North America, and 495/45R22.5 in Europe, which became available after 2000. The trucking industry has encouraged their use due to their increased pavement contact area, and promise of economical and safety benefits. In the new generation of wide-base tire, a better stress distribution was achieved while reducing the contact stresses. It appears that tire manufacturers are currently paying more attention to pavement damage than they did when the first generation of wide-base tire was introduced in the early 1980s. Since its introduction in the early 1980s, the wide base tire has been the subject of several studies to investigate its potential pavement damage as compared to regular dual assemblies. Table 21 presents some of the literature related to this subject.

Table 21: Summary of some of the studies on wide base tires vs conventional tires

Study/reference	Tire type	Pavement structure	Instrumentation and/ or measurements	Analysis tool	Contact stress	Other test/study conditions	Damage potentials
Akram et al. (1992)	11R22.5 425/65R22.5	1.5 in and 7.0 in HMA	Multiple depth deflectometer to compare the response of two test sections under dual and wide base tires on tandem axle for different speeds of a water tanker.			Dual tires inflated at 820 kPa (120 psi) and wide base tire inflated at 900 kPa (130 psi) and the load was 33 kips for both case.	The wide base tire assembly were found to be approximately 2.8 times more damaging to the thin pavement and 2.5 times more damaging to the thick pavement based on a design equation using vertical compressive strain.
Huhtal et al. (1992)	12R22.5 265/70R19.5 445/65R22.5 385/65R22.5 350/75R22.5	AC thickness 80 mm and 150 mm	Pressure cells and strain gauges			Tire pressure varied from 480 to 1080 kPa while axle load varied from 71 – 107 kN.	It was found that steering axle is the most detrimental and wide base tires are more aggressive than dual tires by a factor of 2.3 to 4.0
COST 334 (2001)	Seventeen tire types including 385/45R22.5 495/45R22.5 295/60R22.5 315/80R22.5 385/65R22.5 315/70R22.5	Three classes of HMA pavements < 100 mm, = 200 mm, > 300 mm	Accelerated pavement testing Pavement instrumentation Strains Rutting depth Fatigue performance	MLE FEM Regression models	Square Uniform Vertical	Varying speeds	1.2 – 1.6 times more damage Suggested use of wide base signle tires on the steering axle
Al-Qadi et al. (2004)	445/50R22.5 455/55R22.5	Seven types of HMA wearing surface, Intermediate HMA layer, Three sections had SuperPaveTM SM9.5A under BM-25-0, Open-graded drainage layer, Cement stabilised subbase, Subbase layer	Pressure cell, HMA strain gauge, and thermocouple for measuring vertical stress, dynamic and static tranverse and longitudinal strains in bounded layers, and temperature, respectively. Truck was used to run the experimental program	FEM	Vertical	Four failure mechanisms were considered: fatigue cracking, hot mix asphalt rutting, subgrade rutting, and top-down cracking. Pavement damage was calculated at three pavement temperatures (5, 25 and 40^{0}C) and at four vehicle	The subgrade rutting damage ratio between 445/50R22.5 and dual tire assemblies ranged between 1.05 and 1.40 depending on the vehicle speed and pavement temperature. The 455/55R22.5 tire cause less subgrade rutting damage than the dual tire assembly at slow speed; while more damage at high speed. The 445/50R22.5 tire causes slightly greater primary rutting damage in the pavement structure than the conventional dual-tire assembly. The 455/55R22.5 tire induced less primary rutting damage than the

						speeds (8, 24, 72 and 105 km/h)	dual tire assembly at slow speed and was as damaging or slightly more damaging at high speed. The 445/50R22.5 and 455/55R22.5 tires would result in significantly less top-down cracking.
Ponniah et al. (2009)	455/55R22.5 275/80R22.5 285/70R19.5 295/75R22.5 11R22.5	200 mm asphalt layer, granular base 300 mm, and granular subbase 300 mm.	Trucks equipped with movable loads on the deck and liftable dual axle Tensile strains, speed, pavement surface and subsurface temperatures Additional investigation used FWD.	Statistical analysis, ESAL and fatigue damage approach	Uniform	Varies axle loads	Based on the analysis of equivalent standard axle loads, the single wide base tire could potentially cause 2 – 3.5 times the damage due to dual tires, while fatigue analysis showed that the damage due to single wide base tire could be 1.7 – 1.9 times the damage caused by dual tires.
Al-Qadi and Wang (2009)	455/5522.5 425/65R22.5 11R22.5	Full-depth flexible pavement sections with three different HMA thicknesses (152, 254, and 420 mm)	Accelerated pavement testing Measured parameters.	FEM	Moving load and 3D contact stress	Utilised implicit dynamic analysis	Wide-base 455 tire causes much less fatigue damage than the first generation.
CSIR (2010)	9R22.5 11R22.5 315/80R22.5 385/65R22.5 425/65R22.5 445/65R22.5	Five pavement structures		Mechanistic-empirical analysis with mePADS software to calculate layer life	Uniform Circular	Average load equivalent factors calculated for single axle and axle units. Assumption 1: tire inflation pressure = tire contact stress Assumption 2: contact area diameter = nominal width of the tire	LEF for a single axle fitted with single tyres at an axle load of 8 t is higher than for a single axle with dual tyres at an axle load of 10 t. This same trend was observed for tandem and tridem axle units. The LEF for a single axle fitted with wide base tyres at an axle load of 10 t is higher than the LEF of a single axle fitted with conventional tyres at an axle load of 8 t; and The LEF for a tandem axle unit fitted with wide base tyres at an axle unit load of 18 t is higher than the LEF of a tandem axle unit fitted with conventional single tyres at an axle unit load of 16 t.
Greene et al.	11R22.5	Six test lanes (open	Unidirectional APT	FEM	Vertical		Pavement damage induced by the

((2010)	425/65R22.5 445/50R22.5 455/55R22.5	and dense graded asphalt surface textures), base 265 mm limerock base, and 305 mm stabilised limerock subgrade	Rutting and strain measurements	(ADINA software)			455-mm wide-base tire could be considered similar to that of the dual tire in terms of rutting and bottom-up cracking and slightly improved in terms of top-down cracking.
Moazami et al. (2011)	445/50R22.5 455/55R22.5 425/65R22.5	Dense asphalt pavement 40 mm thick (3000 MPa), A-1-b base course 250 mm (500 MPa), A-2-4 subbase 200 mm (250 MPa), and A-6 subgrade (75 MPa)		Modified layer linear elastic method (KENPAVE)	Uniform		Wide-base tires (445/50R22.5 and 455/55R22.5) reduce vertical contact stress and pavement damage, since they provide wider area of contact and require lower inflation pressure. Older generation of wide-base tire (425/65R22.5) was considered more detrimental to the pavement in terms of bottom-up fatigue cracking
Wang and Al-Qadi (2011)	455/55R22.5 Conventional dual tire	Secondary road 76 mm HMA layer, an aggregate base layer with various thicknesses (203 mm, 305 mm, 457 mm)	Linear viscoelastic model to simulate HMA behaviour and resilient modulus values for subgrade was estimated from its CBR value and for aggregate was estimated as the average of modulus from repeated-load triaxial tests	3D FEM	Moving load and 3D tire-pavement contact stress	Utilised implicit dynamic analysis and available damage models	New generation of wide-base tire causes greater fatigue damage, subgrade rutting, and HMA rutting (densification) compared to conventional dual-tire assembly when carrying the same load.

Christison et al. (1980) cited in Akram et al. (1992) investigated a variety of tire configurations on asphalt pavements under comparable test conditions and found that one application of a single wide base tire is equivalent in its potential damaging effect to between 1.2 and 1.8 applications of a dual tire in terms of asphaltic fatigue life and limiting surface deflections. According to Cebon (1999), cited by Hjort et al. (2008) noted that various experimental and theoretical studies have indicated that single and wide based single tires can cause up to 10 times more fatigue damage on thin flexible pavements, compared to dual tires carrying the same static load. Moreover, tyre contact conditions are less important for rutting of thicker flexible pavements for which wide single tires are only 1.5 – 2 time more damaging than dual tires, and that the tire type has little influence on fatigue damage of rigid pavements.

Bonaquist (1992) cited in Priest et a. (2005) investigated the effect of 425/65R22.5 tire on the flexible pavement response and performance at the accelerated loading facility. They found that pavement strain and stress significantly increased under single wide-base tires, and both the fatigue and rutting life of the pavement decreased dramatically. Akram et al. (1992) compared the 11R22.5 and 425/65R22.5 tires. They found that on both types of axles (drive, or trailer axle), the wide base tire assembly induced larger compressive strain to the subgrade. The wide base tire assembly were found to be approximately 2.8 times more damaging to the thin pavement and 2.5 times more damaging to the thick pavement based on a design equation using vertical compressive strain. It was also found that for both assemblies, vertical strains decrease as the speed of the test vehicle increases. Priest et al. (2005) reported about that other studies using computer models found similar results, for example a study by Siddharthan et al (1998) (cited in Priest et al. 2005) used a continuum-based finite-layer model with dynamic loading to investigate the effect of wide base tires and found that the wide base tire 425/65R22.5 causes 33 and 16 % increase in calculated strain under a 150 mm and 250 mm asphalt layer, respectively. Siddharthan et al focused on the induced strain at the bottom of the asphalt layer because that particular response is linked to fatigue damage and is used in pavement thickness design. Priest et al (2005) noted that because of such studies, many agencies placed restrictions on the use of wide-base or single tires to preserve the integrity of their highway infrastructure. Second generation of wide-base tires have a much wider tread width and a lower profile than those developed in the 1980s. Further they carry the load differently than their predecessors and are designed to be run at higher speeds and lower tire inflation pressures (Kilcarr, 2001 cited in Priest et al., 2005).

AL-Qadi and Wang (2009) noted that the average peak longitudinal tensile strain ratios between the wide base tire and the dual-tire assembly are 1.25 for the wide-base 425 tire and 1.16 for the wide-base 455 tire. In addition, the pressure differential in a dual-tire assembly induces higher longitudinal strains in thin and medium-thickness HMA layers, compared to the longitudinal tensile strains caused by a dual-tire assembly with equal tire pressure. Results of pavement response and damage analysis indicate that the wide-base 455 tire causes greater fatigue damage and subgrade rutting than does the conventional dual tire assembly when carrying the same load. However, the relative damage ratios between various configurations decrease as the pavement thickness increases. The relative fatigue damage potential caused by the wide-base tire in thin pavements could be reduced when considering the wandering effect and possible pressure differential in dual tires. On the other hand, the wide-base 455 tire causes less top-down cracking, near-surface cracking, and HMA rutting damage than does the conventional dual-tire assembly. This suggests that the new wide-base 455 tire causes less damage near the pavement surface, while it causes greater damage at a deeper pavement depth.

CSIR (2010) the calculated Load Equivalent Factors for the various legal axles and axle units and the axles and axle units fitted with wide base tires. The study concluded by recommending that the road wear limit for single axles fitted with wide base tyres with a width of 425 mm and wider could be increased by 500 kg to 8 500 kg, while the road wear limit for tandem axle units fitted with wide base tyres with a width of 425 mm and wider could be increased by 1 000 kg to 17 000 kg. As shown in Table 22, it was noted that the current legal axle and axle units fitted with conventional single tyres cause far more road wear than legal axles and axle units fitted with dual tyres at a higher axle load and the study, therefore, recommended that the use of axles and axle units fitted with conventional single tyres should be discouraged on vehicles transporting mass freight. The study suggested that this could be achieved by lowering the road wear limits for these axles and axle units to such an extent that their use would only be viable on vehicles transporting volume freight. Basing on the principle that axles and axle units fitted with wide base tyres should not cause more road wear than similar axles and axle units fitted with dual tyres, the study noted that the recommendation would however not be achievable as it was found that axles and axle units fitted with wide base tyres cause considerably more road wear than similar axles fitted with dual tyres (see Table 22). For this reason it was concluded that the road wear limits for axles and axle units fitted with wide base tyres should not be higher than the limits for the equivalent axles and axle units fitted with conventional dual tyres.

Table 22: LEFs for legal axles and axles fitted with wide base tires at different axle loads (source CSIR, 2010)

Axle Type	Axle Load (kg)	Legal Axle			Axles fitted with wide base tyres		
		Diagramme	Tyre Size	LEFs	Diagramme	Tyre Size	LEFs
Single	8000		315/80R 22.5	2.700		385/65R 22.5	2.562
						425/65R 22.5	2.333
						445/65R 22.5	2.333
	10000		315/80R 22.5	1.444		385/65R 22.5	n.a
						425/65R 22.5	4.118
						445/65R 22.5	3.898
Tandem	16000		315/80R 22.5	5.346		385/65R 22.5	5.07
						425/65R 22.5	4.614
						445/65R 22.5	4.614
	18000		11R22.5	1.801		385/65R 22.5	7.075
						425/65R 22.5	6.093
						445/65R 22.5	5.864
Tridem	24000		315/80R 22.5	7.998		385/65R 22.5	7.584
						425/65R 22.5	6.899
						445/65R 22.5	6.899
	24000		11R22.5	1.696		385/65R 22.5	7.584
						425/65R 22.5	6.899
						445/65R 22.5	6.899

Huhtal et al. (1992) investigated the effects of several tire types (12R22.5, 265/70R19.5, 445/65R22.5, 385/65R22.5, and 350/75R22.5) on asphalt pavement. They found that steering axle is the most detrimental and wide base tires are more aggressive than dual tires by a factor of 2.3 to 4.0 (e.g. a six ton axle with the smallest wide base tire has the same aggressiveness as the ten tons axle with normal dual tires). Since the load is very seldom evenly distributed on both dual tires, uneven load was simulated by tire pressures. Despite this, wide base tires were more aggressive by a factor of 1.2 to 1.9 if they were compared to the most common dual type. Likewise, Priest et al. (2005) compared the effect of 445/50R22.5 with the standard dual tire 275/80R22.5 using an instrumented test track (including strain and pressure gauges) which can compare the dynamic response of the pavement under a moving load. The study concluded that a wide base single tire 445/50R22.5 caused similar, if not identical, pavement response as a conventional dual tire assembly. The two configurations for the three measured responses (asphalt strain, base stress, subgrade stress) produced, statistically, the same results. Moreover, our review found that Green et al. (2010), on the contrary, found that 445-mm tire was shown to rut dense-graded pavement surfaces more than twice as fast as a standard dual. The 445-mm tire was also predicted to create slightly more bottom-up cracking damage than a dual tire but less or similar surface cracking damage. The pavement damage trends shown in 2008, particularly rutting damage on dense graded surfaces, would likely increase if the 445-mm tire is included in the future. Green et al. suggested that new technology should be thoroughly evaluated in terms of cost–benefit and performance to both the transportation industry and the highway infrastructure where it will be applied. They further suggested that by increasing the tire contact area, pavement damage can be decreased.

COST 334 (2001) framework included the determination of the relative effects of several single tyres, wide base single tyres and dual tyres on the wear of pavement structures. Since the effects of different tyres on pavement wear are only partly (if at all) caused by the differences in the tyre type as such, COST 334, therefore, concentrated on those tyre types, loading conditions, pavement structures and types of pavement wear, that are most relevant for European present-day practice. Regarding the influence of from using different tire pressures, COST 334 reports that the ratio of actual to recommended inflation pressure was shown to be influential for the cases of primary rutting on thick pavements and secondary rutting on thin and medium pavements. An inflation pressure 10% higher than the recommended for the actual tire load results in about 15% increase in pavement wear. In order to improve the efficiency of transportation operations, while concurrently balancing the overall damage of the vehicle on the road infrastructure, COST 334 has suggested the use of wide-base single tires on the steering axle. COST 334 noted that replacement of single tyres on steering axles by wide base singles results in a reduction of pavement damage. Table 23 illustrates the overall damage of the vehicle as compared to the reference axle if wide-base tires were used in the steering axle.

Table 23: Variation of pavement damage for different axles of a typical European truck (Source: COST 334, 2001)

				Primary Roads		Secondary Roads	
Axle	Tire Type	W mm	D mm	TCF	Wide base vs. dual or single	TCF	Wide base vs. dual or single
Towed	Dual	410	973	1.57		1.43	
	Wide Base Single	328	1049	1.84	+17%	2.82	+97%
Driven	Dual	455	1038	1.04		1.00	
	Wide Base Single	427	1013	1.22	+17%	1.64	+64%
Steering	Single	245	1023	3.25		5.86	
	Wide Base Single	307	1035	2.07	-36%	3.21	-45%

As shown in Tables 24 and 25, Al-Qadi et al (2004) found that the 455/55R22.5 induced approximately the same pavement response or damage as the standard dual assembly tested (275/80R22.5). Tire aggressiveness was found to be a function of temperature, speed, and pavement structure. The dynamic loading effect was not covered in this study phase. The new generation of wide base tire has a lower radial stiffness, which reduces the dynamic impact of the tire. Pavement damage at high speed will be reduced when dynamic loading is considered in the analysis. The other wide base tire tested (445/50R22.5) was found to slightly increase the induced damage. It was suggested that load reduction on an axle equipped with the 445/50R22.5 tire should range between 4.0 to 6.0% at a speed of 8km/h and between 5.0 to 11.0% at a speed of 105km/h to maintain the same effect on flexible pavement as that of dual tires. Finite element analysis was also used to determine the load that can be used with the 455/55R22.5 tire to cause an equitable damage to flexible pavements as that caused by the dual-tire assembly. Using the 455/55R22.5 tire would reduce pavement damage at slow speed and would increase it at high speed. It was found that the axle load could be increased by 5 to 10%, depending on the pavement temperature, when 455/55R22.5 tires are used at low speed and should be reduced by 2 to 5% at high speed. If the 455/55R22.5 tires are used, Al-Qadi et al suggested that it is reasonable to uphold the current load limits that are applied to dual-tire assembly at this point of research.

Table 24: Recommended load limits at a speed of 8 km/h (Source: Al-Qadi et al., 2004)

Temperature (°C) Tire	Dual (kg)	Wide-Base Tire 445/50R22.5 (kg)	Wide-Base Tire 455/55R22.5 (kg)
5	9000	8700	9900
25	9000	8400	9500
40	9000	8600	9500

Table 25: Recommended load limits at a speed of 105 km/h (Source: Al-Qadi et al., 2004)

Temperature (°C) / Tire	Dual (kg)	Wide-Base Tire	
		445/50R22.5 (kg)	455/55R22.5 (kg)
5	9000	8600	8900
25	9000	8200	8900
40	9000	8000	8500

Ponniah et al. (2009) also investigated the effects of 455/55R22.5, 275/80R22.5, 285/70R19.5, 295/75R22.5, and 11R22.5 tires. They found that single wide based tires at 9,000 kg axle load could potentially cause more pavement damage than the best-performing dual tires at 10,000 kg. Dynamic loading test showed that the dual tire produces less strain than single wide base tire under normal and dynamic loadings. Contact stress results indicated single wide base tires produce 20% higher stress than the dual tires under the same axle loading. Single axle load on dual tires equivalent to the single wide base tires at 9,000 kg appears to vary with pavement strength; the weaker the pavement, the higher the equivalent axle load on dual tires. Based on the analysis of equivalent standard axle loads, the single wide base tire could potentially cause 2 – 3.5 times the damage due to dual tires. Based on fatigue analysis, the damage due to single wide base tire could be 1.7 – 1.9 times the damage caused by dual tires.

Moazami et al. (2011) found that damage ratio of 2.58 was obtained between 455/55R22.5 and 425/65R22.5. Moreover, damage ratio was 1.65 between 445/50R22.5 and 425/65R22.5 tires. These ratios indicate that for equal normal loading, and thin asphalt layer, wide-base 425/65R22.5 is 2.58 and 1.65 times more damaging than 455/55R22.5 and 445/50R22.5, respectively in terms of bottom-up fatigue cracking. Overall, Moazami et al found that Wide-base tires (445/50R22.5 and 455/55R22.5) reduce vertical contact stress and pavement damage, since they provide wider area of contact and require lower inflation pressure. Older generation of wide-base tire (425/65R22.5) was considered more detrimental to the pavement in terms of bottom-up fatigue cracking.

Cebon (1999) cited by Hjort et al. (2008) reported that several studies have indicated that fatigue damage due to tensile strain at the bottom of a thin asphalt pavements is likely to increase rapidly with average contact pressure, while the inflation pressure has little effect on subgrade rutting. Al-Qadi and Wang (2009) observed though many researchers have used the circular contact area in the pavement loading analysis, however, the contact area of a truck is in reality closer to a rectangular than a circular shape. The length of the contact area depended primarily on the applied load, while the width remained almost constant. The gross contact area increased as the tire load increased, while the effect of the inflation pressure was not so significant.

Wang and Al-Qadi (2011) found that new generation of wide-base tire causes greater fatigue damage, subgrade rutting, and HMA rutting (densification) compared to conventional dual-tire assembly when carrying the same load. On the other hand, the new wide-base tire causes less HMA rutting (shear) and base shear failure potential than the conventional dual-tire assembly. These damage ratios vary with base layer thickness, HMA temperature, and possibly, loading. The study found that wide-base tires' impact on secondary road pavement damage depends on roads' predominant failure mechanisms.

Conclusion and Recommendation

It can be concluded that first generation of wide base tires (385/65R22.5, 425/65R22.5) slightly increases the induced pavement damages. However, with the exception of 445/50R22.5, results on the performance of second generation of wide base tires are still mixed in the sense that some studies have found 455/55R22.5 tires to induce approximately the same pavement response or damage as the standard dual assembly while others have found that these tires still cause greater fatigue damage, subgrade rutting, and HMA rutting (densification) compared to conventional dual-tire assembly when carrying the same load. Additionally, the 455/55R22.5 is noted to cause less HMA rutting (shear) and base shear failure potential than the conventional dual-tire assembly. Our review, and more particularly studies by COST 334 and CSIR (2010), showed that replacement of single tyres on steering axles by wide base singles results in a reduction of pavement damage. Accordingly, CSIR (2010) give recommendations which need to be considered by the EAC. In line with the conclusion and recommendations drawn by CSIR (2010) and the studies reviewed in the previous sections, we recommend that the use of super single tyres of 385 mm and conventional tyres of 315 mm as single tyres on legal axles and axle units should be banned forthwith.

Additionally, we recommend that:

a) Because of the continued development of new and improved tires, it should not be concluded that all new generation of wide-base tires will increase pavement damage. Rather, EAC should initiate and sustain continued research and investigation into the immediate and prolonged effects of tires as they are developed. This includes analysis through mechanical models, measured pavement models, and field performance.

b) Since the investigation of effects of wide base tires on pavements involve assessment of pavement responses in terms of stress, strain, and deflections, it is high time for the EAC Partner States to use analytical methods for the design of new or rehabilitation of pavements.

3.4.4 Application of Analytical Pavement Design Method in the Longer Term

In analytical pavement design it is assumed that pavements deteriorate due to repetitions of the stresses, strains and deflections generated by traffic loads, ultimately reaching a terminal condition that necessitates strengthening. Performance models attempt to relate those parameters associated with a particular mode of deterioration to the number of repetitions that can be sustained before a terminal condition is reached. In the more advanced design methods the models have been derived by correlating the results of laboratory and theoretical studies with empirical data. In the majority of analytical design methods it is assumed that the pavement and subgrade can be modelled as a system of elastic layers of finite thickness supported by a semi-infinite elastic mass, and that the stresses, strains and deflections generated in the system, by traffic, can be determined by solving the general elastic equations governing the behaviour of multilayer linear elastic systems.

3.4.4.1 AASHTO 2002 Design Guide

The design procedure depends heavily on the characterization of the fundamental engineering properties of paving materials. It requires a number of input data in four major categories: traffic, material characterization and properties, environmental influences, and pavement response and distress models. The guide accounts for the environmental conditions that may affect pavement responses which, in turn, are determined by mechanistic procedures. The mechanistic method determines structural response, or strain and stress, in the pavement structure. The transfer function is utilized to directly calculate individual distresses (top-down cracking, bottom-up cracking, transverse cracking, and rutting) in an empirical manner. Pavement design inputs include traffic (full load spectra for various axle configurations), material and sub-grade characterization, climatic factors, performance criteria, and many other factors.

One of the most interesting aspects of the design procedure is its hierarchical approach: that is, the consideration of different levels of inputs.

- Level 1 requires the engineer to obtain the most accurate design inputs (e.g., direct testing of materials, on-site traffic load data, etc.). It provides the highest level of accuracy and, accordingly, would have the lowest level of uncertainty or error. Generally, it requires project-specific input such as material input measured by laboratory or field testing, site-specific axle load spectra data, or nondestructive deflection testing. Because such inputs require additional time and resources to obtain, Level 1 inputs are generally used for research, forensic studies, or projects in which a low probability of failure is important. Site and/or material-specific inputs for the project are to be obtained through direct testing or measurements. This level of input uses the state of the art techniques for characterization of the materials, such as the dynamic modulus of HMA, as well as characterization of traffic through collection of data from (WIM) stations.
- Level 2 requires testing, but the use of correlations is allowed (e.g., sub-grade modulus estimated through correlation with another test). Level 2 input supplies an intermediate level of accuracy that is closest to the typical procedures used with earlier editions of the AASHTO guide. Level 2 input would most likely be user-selected from an agency database, derived from a limited testing program, or be estimated through correlations. Examples of input includes estimating asphalt concrete dynamic modulus from binder, aggregate, and mix properties; estimating Portland cement concrete elastic moduli from compressive strength tests; or using site-specific traffic volume and traffic classification data in conjunction with agency-specific axle load spectra. Level 2 input is most applicable for routine projects with no special degree of significance. This level uses correlations to determine the required inputs. For example, the dynamic modulus could be estimated based on results of tests performed on binders, aggregate gradation and mix properties. The level of accuracy for this category is considered as intermediate.
- Level 3 generally uses estimated values. Level 3 input affords the lowest level of accuracy. This level might be used for designs where there are minimal consequences of early failure, as with lower volume roads. Inputs typically would be user-selected values or typical averages for the region. Examples include default unbound materials, resilient modulus values, or the default Portland cement concrete coefficient of thermal expansion for a given mix classes, and aggregates used by an agency. This level produces the lowest accuracy. Inputs are typically user selected from national or regional default values, such as characterizing the HMA using its physical properties (gradation) and type of binder used.

Thus, Level 1 has the least possible error associated with inputs, Level 2 uses regional defaults or correlations, and Level 3 is based on the default values. This hierarchical approach enables the designer to select the design input depending on the degree of significance of the project and availability of resources.

Climatic Inputs

To use the Enhanced Integrated Climatic Model (EICM) model, a relatively large number of input parameters are needed and include the following (NCHRP 1-37A 2004):

- General information
- Weather-related information
- Groundwater table depth
- Drainage and surface properties, and
- Pavement structure materials.

Traffic Inputs

For traffic analysis, the inputs are much more complicated than those required by the 1993 AASHTO design guide. In the 1993 design guide the primary traffic-related input was the total design 80 kN equivalent single axle loads, ESALs, expected over the design life of the pavement. In contrast, the more sophisticated traffic analysis in the new guide uses axle load spectra data. The following traffic related input is therefore required (NCHRP 1-37A 2004):

- Base year truck-traffic volume (the year used as the basis for design computation)
- Vehicle (truck) operational speed
- Truck-traffic directional and lane distribution factors
- Vehicle (truck) class distribution
- Axle load distribution factors
- Axle and wheel base configurations
- Tire characteristics and inflation pressure
- Truck lateral distribution factors, and
- Truck growth factors

Material Inputs

There are a number of material inputs for the design procedure and various types of test protocols to measure material properties.

Processing of Input over Design Analysis Period

The design period is divided into incremental analysis periods typically by the month or for two weeks for frost conditions. The design inputs are processed to obtain seasonal values of the traffic, material and climatic inputs needed for each analysis increment. The traffic level and the layer moduli are computed for each increment period. Temperature and moisture profiles in pavement layers are obtained using the Enhanced Integrated Climatic Model (EICM) built in the software. The asphalt global aging model modifies the hot-mix asphalt properties for long-term aging so the dynamic modulus of the asphalt mix is increasing over time.

Pavement Structural Response and Distress Models

Two flexible pavement analysis methods have been incorporated in the Design Guide.

Multilayer linear elastic analysis method: The assumptions associated with this method are that each layer is homogeneous, has finite thickness except for the subgrade, is isotropic, full friction is developed between layers at each interface, and there are no surface shearing forces. The elastic modulus and Poisson's ratio of each layer are essential for the stress solution of the problem.

Finite element method: The finite element module is initiated when the unbound material's non-linear behaviour is considered and the coefficient and exponents of the resilient modulus prediction model are entered for level 1 input (of unbound layers).

Critical stress and/or strain values computed by the structural model include:

- Tensile horizontal strain at the bottom/top of the HMA layer to predict fatigue cracking;
- Compressive vertical stresses/strains within the HMA layer for prediction of HMA rutting;
- Compressive vertical stresses/strains within the base/subbase layers for prediction of rutting of unbound layers; and
- Compressive vertical stresses/strains at the top of the subgrade for prediction of subgrade rutting

Distress Prediction

The critical stress and/or strain values obtained from the structural response model are converted to incremental distresses, either in absolute terms, such as in rut depth calculation or in terms of a damage index in fatigue cracking. Distress models, sometimes called transfer functions, relate the calculated damage to field observed distresses. The cumulative damage is converted to physical cracking using calibrated distress prediction models and the output at the end of each analysis period is tabulated by the Design Guide software and plotted for each distress type. The structural distresses considered in flexible pavement design and analysis includes:

- Bottom-up fatigue cracking (alligator);
- Surface-down fatigue cracking (longitudinal);
- Fatigue in chemically stabilized layers (in semi-rigid pavements);
- Thermal cracking; and
- Permanent deformation (rutting)

Smoothness

All types of structural distresses mentioned above contribute to the loss of pavement smoothness. Users usually rate the road performance by its roughness. The smoothness is defined as the variation of surface elevation that induces vibrations in traversing vehicles. The new guide adopted the international roughness index (IRI) as a measure for smoothness. In addition to the structural distresses, the performance criteria for smoothness are achieved through specifying a terminal IRI at a defined level of design reliability. The smoothness model adds changes to the initial smoothness (IRI) over the design period. These changes are due to the increase in individual distress, site conditions and maintenance activities.

Reliability

The design of flexible pavements is associated with many factors that introduce a substantial measure of variability. These factors include traffic levels, material properties and construction quality, in addition to model prediction errors and calibration measurement errors. There are two methods of pavement design to cater to these uncertainties: deterministic and probabilistic. In the deterministic method, each design factor has a fixed value based on the factor of safety assigned by the designer. In the probabilistic method, each design factor is assigned a mean and a variance.

Reliability is defined as the probability that each of the key distress types and smoothness levels will be less than a selected critical level over the design period. For each trial design, the software provides a prediction based on mean or average values for all inputs that correspond to 50% reliability. The designer usually specifies a higher probability that the design will meet the performance criteria over the design life. The distribution of the error term for a given distress about the mean expected prediction is a function of the many sources of variation and uncertainty mentioned earlier. Distress and IRI are approximately normally distributed. The standard deviation for each distress type was determined from the model prediction error from calibration results. The standard deviation of IRI was determined using a closed form variance model estimation approach while the standard deviation of the distribution of distress was determined as a function of the predicted cracking. The reliability of the design can be calculated using the mean and standard deviation of a normal distribution in two steps

Predict the cracking level over the design period using mean inputs to the model. Estimate cracking at the desired reliability level Reliability is defined as the probability that each of the key distress types and smoothness levels will be less than a selected critical level over the design period.

For each trial design, the software provides a prediction based on mean or average values for all inputs that correspond to 50% reliability. The designer usually specifies a higher probability that the design will meet the performance criteria over the design life. The distribution of the error term for a given distress about the mean expected prediction is a function of the many sources of variation and uncertainty mentioned earlier. Distress and IRI are approximately normally distributed. The standard deviation for each distress type was determined from the model prediction error from calibration results. The standard deviation of IRI was determined using a closed form variance model estimation approach while the standard deviation of the distribution of distress was determined as a function of the predicted cracking. The reliability of the design can be calculated using the mean and standard deviation of a normal distribution in two steps

Predict the cracking level over the design period using mean inputs to the model. Estimate cracking at the desired reliability level

Traffic

The traffic characterization method proposed in the new guide is more precise than the conventional ESALs technique adopted in previous guides. The load spectra for single, tandem, tridem and quad axles are introduced. Traffic data, including truck count by class, by direction and lane, are required for traffic characterization. Axle load spectra distributions are developed for each vehicle class from axle weight data. Traffic volumes by vehicle class are forecasted for the design analysis period. The traffic module determines the total number of axle applications for each axle type and load group over the design period. The number of applications for each axle type and load increment is then used in the computation of pavement responses, damage and distress prediction.

Additional data related to the axle configuration, such as average axle width, dual tire spacing, tire pressure and axle spacing are used in the pavement response module. Traffic wander influences the number of load applications over a point. This parameter affects prediction of fatigue and permanent deformation. Wander is assumed normally distributed and the standard deviation is used as a measure for it.

An important feature introduced in the guide is the provision for the special axle configuration. This option allows the designer to analyze pavement performance due to special, heavy, non-conventional off-road vehicle systems. Another important input for the flexible pavement design is the vehicle operational speed. It directly influences the stiffness response of the visco-elastic asphalt concrete layers. The magnitude and duration of stress pulses caused by the moving traffic depend on the vehicle speed, type and geometry of pavement structure, and the location of the element under consideration. The frequencies corresponding to various speeds are used to calculate the dynamic modulus of the asphalt concrete layer.

Environmental Effects

Environmental variations can have a significant impact on pavement performance. Moisture and temperature changes can significantly affect the pavement material's properties and, hence, its strength, durability and load carrying capacity. The asphalt bound materials modulus value can rise during the cold winter months by 20 times its value during the hot summer months. Excessive moisture can drastically lead to the stripping of asphalt mixture. Similarly, the resilient modulus of unbound materials at freezing temperatures exhibits high values compared to the thawing months. The moisture content affects the state of stress of unbound materials and it breaks up the cementation between soil particles. Increased moisture contents lower the modulus of unbound materials.

The Design Guide incorporated the Enhanced Integrated Climatic Model (EICM) to simulate changes in the behaviour and characteristics of pavement and subgrade materials that concur with climatic conditions over the design period. The model computes and predicts the modulus adjustment factors, pore water pressure, water content, frost and thaw depths, frost heave and drainage performance. The EICM consists of three components as shown in Figure 3.3. The model role is to record the user-supplied resilient modulus MR of all unbound layer materials at an initial or reference condition, generally at near optimum moisture content and maximum dry density. The model then evaluates expected changes in moisture content and the effect on the user-entered resilient modulus.

The model also evaluates the effect of freezing on the layer MR and the effect of thawing and recovery from the frozen MR condition. The model provides varying MR values in the computation of critical pavement response parameters and damage at various points within the pavement system. For the asphalt bound layer, the model evaluates the changes in temperature as a function of time to allow for the calculation of the dynamic modulus and thermal cracking. The climate module in the Design Guide software requires the user to specify a climate file for the project location stored in the database. The database includes over 800 weather stations around the United States. The user has an opportunity to generate a weather file by interpolating climatic data from selected locations inversely weighted by the distance from the required location.

Performance Prediction Models

Permanent Deformation

The Design Guide predicts the rutting of each layer in the pavement structure as a function of time and traffic. The Guide models only the initial and secondary permanent deformation stages shown in Figure 4. The primary stage is modelled using an extrapolation of the secondary stage trend. The tertiary stage is not considered in the model. The structural response model calculates the vertical strain in a layered pavement cross section at any given depth using the elastic properties of the material.

The total permanent deformation needs to be accumulated for different conditions over the design period. To account for the variation in temperature, resilient modulus, and moisture over the incremental periods, the Guide use a special approach called the strain hardening approach to incorporate these variable parameters in a cumulative deformation subsystem. Details of the procedure for calculating the permanent deformation of each layer are given in Part 3-Chapter 3 of the guide documentation.

Fatigue Cracking

Repeated traffic loads cause tensile and shear stresses to develop in the asphalt layer. Fatigue cracking initiates at locations of critical strain and stress. These locations mainly depend on the stiffness of the asphalt layer and the load configuration. Two types of fatigue cracks initiate in the asphalt layer. The first is the commonly known fatigue cracking that initiates due to bending action, which results in flexural stresses at the bottom of the asphalt layer. The second type, which propagates from the surface to the bottom, is believed to be due to critical tensile and or shear stresses developed at the surface due to high contact pressures at the tire edges-pavement interface. Highly aged thin asphalt layers facilitate the initiation of fatigue type two cracks. Modelling of both types of fatigue cracking is based on one approach. The estimation of the fatigue damage is based on Miner’s law. The transfer function for bottom-up cracking and top-down cracking is calculated from the fatigue damage model and calibrated separately.

Thermal Cracking Model

Thermal fracture analysis in the new Design Guide is based on the visco-elastic properties of the asphalt mixture. The thermal cracking model, TCMODEL, is an improved version of a model developed earlier by SHRP. The procedure requires the characterization of the HMA mix in an indirect tensile mode to measure the creep compliance at one or three temperatures depending on the level of analysis. The thermal cracking model assumes that failure occurs when the average crack depth reaches the thickness of the asphalt layer.

Smoothness Models (IRI)

Distresses predicted by the mechanistic-empirical models, such as fatigue cracking, permanent deformation and thermal cracking are correlated to smoothness. In addition the smoothness model optionally considers other distresses, such as potholes, longitudinal cracking outside the wheel path, and block cracking if there is potential of occurrence. There are three models for predicting IRI depending on whether the base type is an unbound aggregate base and subbase, asphalt treated base, or chemically stabilized base.

3.4.4.2 The South African Mechanistic Design Method (SAMDM)

The purpose of this review is to give an overview of the current mechanistic design analysis procedure and not the complete mechanistic design method.

The mechanistic design process starts off with the load and material characterization. The standard design load for South Africa is a 40 kN dual wheel load at 350 mm spacing between centres and a uniform contact pressure of 520 kPa due to the legal axle load of 80 kN allowed on public roads. The material characterization includes layer thickness and elastic material properties for each layer in the pavement structure under consideration. The structural analysis will usually involve a linear elastic, static analysis of the multi layer system. Resulting in the pavement response to the loading condition expressed in terms of stresses and strains at critical positions in the pavement structure determined by the material type used in each layer of the pavement structure.

The pavement response serves as input to the transfer functions for each material type. The transfer functions relate the stress/strain condition to the number of loads that can be sustained at that stress/strain level before a certain terminal condition is reached.

Structural Analysis

The structural analysis is normally done with a static, linear elastic multi layer analysis program. A few points related to the structural analysis that will influence the design analysis procedure should be noted. The maximum horizontal tensile strain at the bottom of asphalt layers and the maximum tensile strain at the bottom of cemented layers are used as the critical parameters determining the fatigue life of these two material types. The position of the maximum tensile strain in a particular layer will not necessarily occur at the bottom of the layer. The position of the maximum horizontal strain will rather be determined by the modular ratios of the layers in the pavement structure. The transfer functions for these materials were however, developed as a function of tensile strain at the bottom of the layer and are used as such.

Pavement Life Prediction

Three concepts are involved in the pavement life prediction. The first is to predict the individual layer life for each of the layers in the pavement structure. Secondly the occurrence of crushing in cemented layers is investigated and thirdly the ultimate pavement life should be predicted.

Transfer Functions

The basic material types used in South Africa are asphalt, granular, cemented and subgrade materials. Each material type exhibits a unique mode of failure. The failure mode for each material type is linked to critical parameters calculated at specific positions in the pavement structure under loading. Transfer functions provide the relationship between the value of the critical parameter and the number of load applications that can be sustained at that value of the critical parameter, before the particular material type will fail in a specific mode of failure.

3.4.4.3 Recommendation

The above review shows that both the AASHTO and South African Mechanistic Design Method (SAMDM) are potential analytical methods for EAC Partner States. The most interesting aspects of the AASHTO design procedure is its hierarchical approach: that is, the consideration of different levels of inputs while the SAMDM constitutes of the basic material type with its accompanying critical parameters, mode(s) of failure and applicable transfer functions which most likely to be similar to our condition in the EA region.

We recommend that EAC needs to establish its own analytical design method suitable for its distinct environmental conditions. In such endeavour, detailed comparison of the AASHTO and SAMDM methods may be useful.

3.5 Review of Bridge Design Standards

3.5.1 Overview of Bridge Design Standards

Designing bridges according to a standard specification became the norm in the 20th century. This will continue in the next century. However, the process of designing will be much different in the future because of changes in specifications, loads, testing, and computerization.

Most of the current design standards allow the selection of the numerical values for partial safety factors and other allowables. In this way, the designers are allowed, within limits, to choose the level of safety, considering local conditions, applicable to bridges in their countries. Justifications for these choices include the following:

- Differences in geographical or climatic conditions
- Differences in traffic loads
- Different levels of safety provided or desired in the jurisdiction

The determination of safety levels, including aspects of durability and economy, has always been considered to be within the competence and authority of individual design units. Possible differences in geographical or climatic conditions, as well as different levels of protection that may exist at national, regional, and local levels, can be taken into consideration through specific design parameters, which are provided for in each most design standards. Therefore, EAC Partner States have choices in the current design codes and the design parameters provided in the standard guides may be used unless divergence is essential.

The AASHTO LRFD Bridge Design Specifications categorize analysis methods as approximate or refined. The approximate methods of analysis, specified in LRFD are those for which a live-load distribution factor is quantified through tabularized equations and used in the analysis of single beams (sometimes termed one-dimensional analysis). These lateral live-load distribution factors and the tributary dead-load areas are applied to a one-dimensional model. Refined methods of analysis, discussed in LRFD, are all other methods in which distribution factors are not used and the bridge is represented as a 2-D or 3-D model. In the United States, their application is limited to unique or complex bridges, bridges deemed substandard using approximate analysis, analysis of nonstandard permit loads, and other special cases. While developing the lateral live-load distribution factors of the LRFD Specifications, some designers have found little benefit in the application of 3-D models beyond simpler 2-D models.

3.5.2 Bridge Design Standards and Practices in East Africa

The EAC Partner States are basically using developed countries bridge design standards. Table 26 presents a summary of codes of practices for the design of bridges as used in the EAC region. It can be noted that Burundi is using both German and French design standards. Rwanda is using the two standards as well as British and American standards. On the other hand, Kenya, Tanzania and Uganda are using British design standard.

Table 26: Code of Practices Used in EA

Country	Code of Practice
Burundi	DIN 1072 + 1055 AFNOR
Kenya	BS 5400
Rwanda	DIN 1072 +1055, BS 5400 AFNOR
Tanzania	BS 5400
Uganda	Own Code but derived from BS 5400

Additionally, it should also be noted that the SADC region has its own bridge design standard, SATCC code of practice for bridge design.

A comparison of the BS 5400, AASHTO and French standards was presented by PADECO (2011). This section uses the same design parameters to compare the BS 5400, SATCC, and European standards. The comparison revealed a number of areas of divergences and commonality as summarised in Table 27.

This document was withdrawn from use in UK on 31st March 2010 and as a result UK is now using BS EN which is customised to Eurocodes. The Eurocodes are a suite of harmonized European standards that cover the basis of structural design, actions on structures, the design of concrete, steel, composite steel and concrete, timber, masonry and aluminium structures, geotechnical design and the design of structures for earthquake resistance. Considered by many leading engineers as the most technologically advanced standards in the world, the Eurocodes are written as a set of principles and are viewed as more versatile than the old standards. They are being implemented across the European Union to deliver a common approach to design, whilst at the same time allowing the application of country specific construction requirements such as wind loads or earthquake resistance. They will also help to open up access to this market. In the longer term, the development and adoption of European Standards (EN Series) will pose additional problems with respect to EAC Partner States design standards linked to British or French standards (AFNOR) or other European National Standards as these become redundant.

Eurocode 1 Part 2-prEN 1991-2-2002 considers the loading of 600 kN per 4-axle line with a notation of 600/150 to 3600 kN per 9 axle-lines of 200 kN (spacing 12 m)+ 9 axle-lines of 200 kN with a notation of 3600/200/200. The code provides basic models of special vehicles that are defined in Tables 28 and 29. It should be noted that the basic models of special vehicles correspond to various levels of abnormal loads that can be authorised to travel on particular routes of the European highway network while vehicle widths of 3.00 m for the 150 and 200 kN axle-lines, and of 4.50 m for the 240 kN axle-lines are assumed.

Table 27: Comparison of Code of Practices for Bridge Design

Items	**United Kingdom**	**SADC Region**	**European standard**
Design Standard	BD37/01:Loads for Highways Bridges (BS5400 Part)	SATCC Draft Code of Practice for the Design of Road Bridges and Culverts sept 1998	Euro code 1, Part 2 final draft prEN 1991-2.Actions on structures-Part 2: Traffic loading on bridges.
Design Method	Partial Factor Design Method	Partial Factor Design Method	Partial Factor Design Method
Design period	120 years	100 years	100 years
Live Load	Type HA Loading (Type HB Loading: Special load)	Type NA normal Loading Type NB abnormal Loading Type NC Super Loading (loading represent multi-wheeled trailer combination)	
Loading on Carriageway: B(m)	2.5m <B<3.65m The number of lanes are determined by the width of the carriageway (W) 2 carriageways :5m<W<7.5m 3 carriageways :7.5m<W<10.95m 4 carriageways :10.95m<W<14.6m 5 carriageways :14.6m<W<18.25m 6 carriageways :18.25m<W<21.96m	2.4m <B<3.7m 2 carriageways :4.8m<W<7.4m 3 carriageways :7.4m<W<11.1m 4 carriageways :11.1m<W<14.8m 5 carriageways :14.8m<W<18.5m 6 carriageways :18.5m<W<22.2m	1 carriageways W<5.4m 2 carriageways :5.4m<W<9m 3 carriageways :9m<W<12m
Uniformly Distributed Load (UDL)	L <50m W = 336 x(1/L)(0.6) (kN) 50m<L<1600m W = 36 x(1/L)(0.1) (kN) L= Loading length	q_{r1}=(180/$\sqrt{L}$) +6 for L<18m q_{r1}={ (180/$\sqrt{L}$) +6}x1/2 for L>36m q_{r1}={ (180/$\sqrt{L}$) +6} x 2/3 for 18m<L<36 L= Loading length q_{r1} = average load per meter of notional lane KN.	Lane 1 ,UDL = 9 kN/m^2 Lane 2 ,UDL = 2.5 kN/m^2 Lane 3 ,UDL = 2.5 kN/m^2 Other Lanes ,UDL = 2.5 KN/m^2
Truck Load	120 kN (1 axle)	= (144/$\sqrt{n}$) kN per notional lane n = no of notional lane, for 1=144kn/lane	80KN + 2 axles@140
Impact Load	The impact load is included in uniform distribution load (UDL) and Truck load	The impact load is included in uniform distribution load (UDL) and Truck load	Impact of 1000 kN in direction of vehicles travel or 500 kN perpendiculars to that direction is included.
Live Load for Slab Design	1@100 kN (Diameter = 34 cm circle)	2@100 kN Circular or square contact area of 0.1m2 each not less than one meter apart.	2@120 kN contact area (40 cm x 40 cm)

Table 28: Classes of special vehicles

Total weight	Composition	Notation
600 kN	4 axle-lines of 150 kN	600/150
900 kN	6 axle-lines of 150 kN	900/150
1200 kN	8 axle-lines of 150 kN or 6 axle-lines of 200 kN	1200/150 1200/200
1500 kN	10 axle-lines of 150 kN or 7 axle-lines of 200 kN + 1 axle line of 100 kN	1500/150 1500/200
1800 kN	12 axle-lines of 150 kN or 9 axle-lines of 200 kN	1800/150 1800/200
2400 kN	12 axle-lines of 200 kN or 10 axle-lines of 240 kN or 6 axle-lines of 200 kN (spacing 12m) + 6 axle-lines of 200 kN	2400/200 2400/240 2400/200/200
3000 kN	15 axle-lines of 200 kN or 12 axle-lines of 240 kN + 1 axle-line of 120 kN or 8 axle-lines of 200 kN (spacing 12 m) + 7 axle-lines of 200 kN	3000/200 3000/240 3000/200/200
3600 kN	18 axle-lines of 200 kN or 15 axle-lines of 240 kN or 9 axle-lines of 200 kN (spacing 12 m) + 9 axle-lines of 200 kN	3600/200 3600/240 3600/200/200

Table 29: Description of special vehicles

	Axle-lines of 150 kN	Axle-lines of 200 kN	Axle-lines of 240 kN
600 kN	n = 4×150 e = 1,50 m		
900 kN	n = 6×150 e = 1,50 m		
1200 kN	n = 8×150 e = 1,50 m	n = 6×200 e = 1,50 m	
1500 kN	n = 10×150 e = 1,50 m	n = 1×100 + 7×200 e = 1,50 m	
1800 kN	n = 12×150 e = 1,50 m	n = 9×200 e = 1,50 m	
2400 kN		n = 12×200 e = 1,50 m n = 6×200 + 6×200 e = 5×1,5+12+5×1,5	N = 10×240 e = 1,50 m
3000 kN		n = 15×200 e = 1,50 m n = 8×200 + 7×200 e = 7×1,5+12+6×1,5	N = 1×120 + 12×240 e = 1,50 m
3600 kN		n = 18×200 e = 1,50 m	N = 15×240 e = 1,50 m n = 8×240 + 7×240 e = 7×1,5+12+6×1,5
NOTE n number of axles multiplied by the weight (kN) of each axle in each group e axle spacing (m) within and between each group.			

3.5.3 Recommendation

In recommending increase in GVM, PADECO Report assumed the use of BS in the EAC region. However, the continued application of BS standard which is superseded by the European Standards (EN Series) in EAC member countries will pose additional problems as BS becomes redundant.

We recommend that EAC member countries should use Euro code 1, Part 2 final draft prEN 1991 - 2. Actions on structures - Part 2: Traffic loading on bridges. In the longer term, however, it will desirable for EAC region to develop its own Bridge Design Standards.

3.6 Impact of Harmonisation

The impacts of harmonising pavement and bridge design standards include the provision of suitable pavement structures that will meet functional and structural performance criteria through the service life of the pavement, and uniform and comfortable riding surfaces to all road users thus resulting in the provision of the same quality of service throughout the region. Additionally, it is proposed that the recommended empirical-based standards be implemented in all new road designs, and road rehabilitation and reconstruction projects in the shorter term, i.e. within 10 years and beyond this period analytical methods should be used in the EAC Region.

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FINAL REPORT

ANNEX A 4

HARMONISATION OF SPECIFICATIONS FOR ROADWORKS

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LIST OF TABLES

GLOSSARY AND ACRONYMS

AASHTO	American Association of State Highway and Transport Officials
AC	Asphalt Concrete
AFNOR	Association Française de Normalisation
ALD	Average Least Dimension
ASTM	American Society for Testing and Materials
BEMIX	Bitumen emulsion mix
BS	British Standards
CBR	California Bearing Ratio
CML	Central Materials Laboratory
CRR	Crushed Rock
CRS	Crushed Stone
DBM	Hot Mix of Bitumen and Aggregate
DR	Damp Rock
EAC	East African Community
FBMI	Foamed Bitumen Mix
G	Soil/ Natural Gravel Material
GW	Gravel Wearing
FI	Flakiness Index
LL	Liquid Limit
LS	Linear Shrinkage
MC	Medium Curing
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
SP	Standard Penetration
TFV	Ten Percent Fine Values
UCS	Unconfined Compressive Strength

EXECUTIVE SUMMARY

The Standard Specifications for Road and Bridge Works are the specifications developed and adopted or followed by EAC Partner States for highway construction and improvement contracts. They are incorporated into the written agreement (Contract) between the Contracting Agency and the Contractor, except where the Contract indicates that a particular specification has been amended or replaced with a special provision to resolve project-specific issues. Standard specifications are to be used only as a guideline during the design process, and once they have been incorporated into a Contract, however, they become the legal and enforceable part of the Contract. Generally, they are meant to outline the general requirements and legal agreements applicable to all highway construction and improvements. The main focus of this chapter, however, is to make recommendations for harmonisation of materials (requirements and test methods) standard specifications for road construction in the EAC region.

Specifications for Road Works Adopted by EAC Partner States

Specifications for Burundi and Rwanda are based on the French and American standards respectively, while Kenya, Tanzania and Uganda are using their own specifications derived from British and American standards.

Potential Areas for Harmonisation and Improvements

There are a number of convergences and divergences in the requirements for earthworks, pavement layers of natural gravel materials, cemented layers, crushed aggregate base course, bituminous layers in the existing Standards Specifications for the EAC partner states. Based on the detailed review and comparison of the requirements, we have recommended higher standard specifications for each of the above items.

Earthworks

G15, G7, G3 and geotexile and/or rock fill (DR) materials are the most common materials for earthworks. Table E1 shows the requirements we recommend for materials for earthworks.

Table E1: Requirements for earthworks

Material Type	**Requirements**				
	CBR (%)	**CBR Swell (%)**	**PI (%)**	**Maximum particle size**	**Maximum layer thickness**
G15	Minimum 15 after 4 days soaking	Max. 1.5	25	1/2 of compacted layer thickness but not > 50 mm	250 mm compacted thickness placed in one operation
G7	Minimum 7 after 4 days soaking	Max. 2	30		
G3	3 after 4 days soaking, measured at 90% of MDD of BS-Heavy compaction	Max. 2	-		
DR				Two thirds of compacted layer	1 m placed in one operation

Source: Adopted from General Specifications for Road and Bridge Works, Uganda (2005)

Natural Gravel Materials for Pavement Layers

Gravel materials are used widely throughout EAC Partner States for various applications as pavement layers in flexible pavement and also as a wearing course in engineered gravel. Table E2 summarises the materials property, including grading, requirements we recommend for natural gravel materials for road pavements.

Table E2: Requirements for natural gravel materials

Material Type	Requirements									
	Compaction	CBR	CBR swell (%)	Max. LL	% passing 37.5 mm sieve	Shrinkage product, SP	Grading coefficient(GC0	Max. PI	Max. LS	TFV
GW G80[a2] G60[a2] G45[b2] G30[d2]	95% of BS-Heavy 98% of BS-Heavy 98% of BS-Heavy 95% of BS-Heavy 95% of BS-Heavy	Min 25[d] Min 80 [d] Min 60[d] Min 45 [d] Min 30 [d]	0.5 1.0 0.5 1.0	30[3]/35[4] 35[3]/40[4] 40[3]/45[4] 45[3]/45[4]	Min. 95	120-400	16 - 34	8[3]/10[4] 10[3]/12[4] 14[3]/16[4] 16[3]/18[4]	4[3]/5[4] 5[3]/6[4] 7[3]/8[4] 8[3]/9[4]	80kN[5] 60%[6] 50kN[5] 60%[6]

Grading, BS 1377: Part 2		
Sieve size (mm)	G80 grading limits (% passing sieve)	G80 grading limits (% passing sieve)
63	100	100
37.5	80 – 100	80 – 100
20	60 – 95	60 -95
5	30 – 65	30 – 65
2	20 – 50	20 – 50
0.425	10 – 30	10 – 30
0.075	5 -15	5 - 15

[a2] d_{max} is ½ of compacted layer thickness but not > 50 mm and GM is 2.0
[b2] GM is 1.5
[c2] GM is 1.2
[d] After 4 days of soaking
[3] General requirements; **[4]** Calcrete or other pedogenic materials; [5]Minimum TFV dry; **[6]** Ratio soaked to dry value of TFV

Source: General Specifications for Road and Bridge Works, Uganda (2005)

Chemically stabilised Materials

Stabilised materials are widely used in the EAC Partner States and currently have varying specifications. Table E3 show the recommended specifications for stabilised materials.

Table E3: Specifications for stabilised materials

Material properties	C2	C1.5	C1.0	C0.7	CM
Min CBR (%) for lime or liquid ionic stabilised material		80	60	45	30
After stabilisation					
Min UCS (MPa)	2.0	1.5	1	0.7	0.5
Before stabilisation					
Soaked CBR (%) at 95% MDD of BS-Heavy	Min 30	Min 30	Min 20	Min 20	No requirement
Plasticity Index (%)	Max 20	Max 20	Max 25	Max 25	Max 35
Aggregate strength TFV_{dry}	Min 50 kN	Min 50 kN	No requirement	No requirement	No requirement
Grading Modulus (GM)	Min 1.5	Min 1.5	Min 1.2	Min 1.0	No requirement
Organic Content	Max 0.5%	Max 0.5%	Max1%	Max 1%	Max 25
Sulphate (SO3) content	Max 0.25%				
Max. particle size	½ of compacted layer thickness but not > 50 mm				

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

When a stabilising agent is selected the following factors need to be considered:

- The physical composition and properties of the material, for example liquid limit, plasticity index (PI), clay content, and initial consumption of lime.
- The purpose of stabilisation, for example modification or cementation and whether rapid or slow increase in strength is required.
- The relative availability and cost of the different stabilising agents.

We therefore recommend use of guidelines in Table E4 for selecting a stabiliser.

Table E4: Guidelines for stabiliser selection

% passing the 0.075 mm sieve BS 1377-2	Plasticity index (%) BS 1377: Part 2	Best suited stabiliser
Less than 25%	PI is less than 6 or PI x (% passing 0.075 mm) is less than 60	Cement only
	6 – 10	Cement preferred
	More than 10	Cement and/or lime
More than 25%	Less than 10	Cement preferred
	10 – 20	Cement and/lime
	More than 20	Lime preferred

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Crushed aggregate Base Course

This material is widely used in the EAC partner states as base course. The recommended specifications are shown in Table E5. Section 4.3.4 in Annex A4 provide a detailed discussion about the practical considerations to be taken into account when selecting and preparing crushed aggregate for base course.

Table E5: Specifications for crushed aggregate base course materials

(a) Materials requirements

Material Type	Requirements					
	Compaction	Max. PI	Max. LS	TFV	TFV: soaked/ dry	Max. FI
CRS	98% of BS-Heavy	6	4	110kN	60%	35%
CRR	102% of BS-Heavy	NP	3	110kN	75%	35%

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(b) Grading requirements

Sieve size (mm)	Grading limits (% passing sieve)			
	CRS		CRR	
	Coarse type	Fine type	Coarse type	Fine type
50	100			
37.5	90 – 100	100	100	
28	75 – 95	90 – 100	87 – 97	100
20	60 – 90	65 – 95	75 – 90	87 – 97
10	40 – 75	40 – 70	52 – 68	62 – 77
5	29 – 65	29 – 52	38 – 55	44 – 62
2	20 – 45	20 – 40	23 – 40	27 – 45
1.18	17 – 40	15 – 33	18 – 33	22 – 38
0.425	12 – 31	10 – 24	11 – 24	13 – 27
0.075	5 – 12	4 – 12	4 – 12	5 - 12

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Conventional Bituminous Materials

Bituminous materials are used widely in the EAC partner states for various road construction activities mostly for spray applications and also for construction of asphalt structural layers. Table E6 shows the recommended specifications for priming and bituminous binders for bituminous base course and asphalt concrete surfacing, respectively.

In addition to the requirements set out in Table E6, foam bitumen should meet the following requirements when tested using a 10 litre cylindrical bucket:

- The ratio between the volume of the bitumen in a foamed state and in an un-foamed state should be minimum 15.
- At least one of the following requirements should be met:
    - the time from the foam is ejected into atmospheric pressure until the volume has decreased to half its maximum volume should be minimum 15 seconds.
    - the ratio between volume of bitumen in a foamed state and in an un-foamed state should be minimum 7.5 after 15 seconds have elapsed since the foam was ejected into atmospheric pressure.

Table E6 Binder specifications for bituminous base and asphalt concrete surfacing

(a) Binder specifications for bituminous base course

Material Type	Requirements				
	Compliance standards	Aggregate for blinding	Application Rate	Temperature (^{0}C) Storage	Temperature (^{0}C) Spraying
MC-30 cut-back bitumen MC-70 cut-back bitumen Invert bitumen emulsion	BS EN 12591:2000 BS EN 12591:2000 BS 432 or SABS 308	Crushed rock or river sand (100% pass. 6.3 mm and should not have more than 10% passing the 2.36 mm sieve)	0.4 – 2.7 l/m^2 and 1.0 l/m^2 for bidding purposes	60/40[1] 80/50[1] ambient	45-60 60-80 Ambient above 10^{0}C

[1] Up to 24 hrs/Over 24 hrs
Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(b) Binder specifications for asphalt concrete surfacing

Type of binder	Specifications
Penetration grade bitumen	BS EN 12591:2000
Performance Grade asphalt	SP-1[a]
Cutback bitumen	BS EN 12591:2000
Bitumen emulsions, anionic	BS 434:1984
Bitumen emulsions, cationic	BS 434:1984

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

Modified bituminous binders

a) Bitumen-Rubber binder

We recommend to the EAC partner states the following specifications for bitumen-rubber binder

- Base bitumen of 60/70, 80/100 or 150/200 penetration grade bitumen or a blend of any two or all three grades to provide a product with the required viscosity.
- Rubber particles shall obtained by processing and recycling pneumatic tyres. It should be pulverised, free from fabric, steel cord and other contaminants. A maximum of 4% by mass of fine particle size calcium carbonate, or talc, may be added to the rubber crumbs to prevent the rubber particles from sticking together. At the time of use the crumbs should be free flowing and dry and comply with the requirements given in Table E7.

Table E7: Requirements for rubber particles for use in bitumen-rubber binder

Sieve analysis		**Test Method**
Sieve size (mm)	Percentage passing by mass	
1.18 0.60 0.075	100 40-70 0-5	BR6T (Sabita)*)
Other requirements		
Natural rubber hydro-carbon content	30% (minimum)	BS 903 Parts B11 and B12
Fibre length	6 mm (maximum)	-
Relative density (kg/m^3)	1100-1250	BR9T (Sabita)

*Southern African Bitumen and Tar Association
Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Extender oils should be petroleum derived material of high aromaticity and are required to comply with specifications shown in Table E8.

Table E8: Requirements for extender oils

Property	**Requirements**
Flash point	180^{0}C
% by mass of saturated hydrocarbons	25% (max)
% by mass of aromatic, unsaturated hydrocarbons	50% (min)

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

For a bitumen-rubber blend, we recommend a distillate of hydrocarbon and bitumen-rubber, including extender oil and /or diluents, which comply with the requirements given in Table E9.

Table E9: Requirements for bitumen-rubber blend

(a) Composition, reaction temperature and time

Property	**Requirement**
% rubber by mass of total blend	20%-24%
% extender oil by mass of total blend	6% (max)
% of diluents by mass of total blend	7% (max)
Blending/Reaction temperature	1 70^{0}C – 210^{0}C
Reaction time	0.5 - 2 hours

(b) Test methods

Property	Requirements	Test Method (or equivalent method on approval of the Engineer)
Minimum compression recovery: • after 5 minutes • after 1 hour • after 4 hours	 70% 70% 48-55%	BR3T (Sabita)
Ring-and-ball softening point	minimum 55^{0}C	BS EN 12591:2000
Resilience	13%-35%	BR2T (Sabita)
Dynamic viscosity (Haake at 190^{0}C)	20 35 cPas	BR5T (Sabita)
Flow (mm)	20 – 75	BR4T (Sabita)

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

b) Homogeneous Cold Applied Modified Binders

Requirements for cationic modified bitumen emulsion containing SBR or SBS solids are provided in Table E10.

Table E10: Requirements for cationic bitumen emulsion

Polymer modifier	Required properties, grade of base bitumen	Minimum modified binder content (%)	Minimum viscosity at 50^{0}C Saybolt Furol (sec.)	Maximum residue on sieving (g/100ml)	Particle charge	Sedimentation after 60 rotations
SBR	80/100 150/200	70 65	80 70	0.25 0.25	Positive Positive	Nil Nil
SBS	80/100 150/200	70 65	80 50	0.25 0.25	Positive Positive	Nil Nil
Test Method	-	ASTM D244	ASTM D244	SABS 548	SABS 548	SABS 548

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

A volatile solvent flux content of up to 3% mass by mass of the bitumen may be added to enhance emulsion performance with regard to prevailing climatic conditions. Also, we recommend the properties of the recovered modified bitumen using a rotary vacuum evaporation method or simple evaporation method to comply with the requirements in the Table E11.

Table E11: Properties for recovered modified bitumen

Polymer Modifier	Required properties				Elastic recovery (%)		
	Grade of base bitumen	Minimum Softening point (^0C)	Minimum dynamic viscosity at 135^0C (Pa.s)	Minimum ductility at 10^0C (mm)		at 5^0C (%)	at 50^0C (%)
SBR						90 90	100 100
SBS	80/100 150/200	60 47	1.2 1.0	500 500	55 60	90 90	100 100
Test Method	-	ASTM D36	ASTM D4402	DIN 52013	DIN 52013	DIN 52013	TMH methodB11
Note: Modified binder is bitumen plus polymer.							

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

c) Homogeneous Hot-Applied Modified Binders

We recommend that the requirements for any polymer other than the generic types listed in Table E12 used for the manufacture of homogeneous hot-applied modified binders should be indicated in the Special Specifications.

Table E12 Properties for homogeneous hot applied modified binders

Genetic type of modified binder		Required properties						Minimum adhesion	
		Grade of Base bitumen	Minimum Softening point (^0C)	Minimum dynamic viscosity at 135^0C (Pa.s)	Minimum ductility at 10 ^0C (mm)	Minimum elastic recovery 10^0C (%)	Maximum stability difference (^0C)	at 5^0C (%)	at 50^0C (%)
Plastomerpolymer (EVA)		150/200	48	0.5	300	45	2	90	
Elasto-merpoly-mer	SBR	80/100	47	1.0	1000	55	2	90	100
	SBS	80/100	49	1.0	500	60	2	90	
	SBR	150/200	45	0.5	1000	55	2	90	
	SBS	150/200	47	0.5	500	60	2	90	100
Test Method	-	-	ASTM D36	ASTM D4402	DIN 52013	DIN 52013	DIN 52013	TMH 1 Method B11	

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Furthermore, where applicable the following details should also be indicated in the Special Specifications:

- Generic type (plastomer or elastomer) and type polymer.
- Grade base bitumen (80/100 or 150/200) required.

Aggregates for Bituminous mixes

There are variations in the specification used by the EAC partner states in the production of asphalt mixes. Table E13 presents recommended specifications for aggregates to be used in the mixes for tests for particle shape, aggregate strength, cleanliness, soundness and coating and stripping. Detailed discussion including particle size distribution for the same can be found in this Annex.

We recommend the specifications given in Table E13 for aggregates for bituminous base courses and asphalt concrete surfacing layers.

Table E13: Requirements for aggregates for bituminous mixes

Property	Requirement	Test Method
Cleanliness	A minimum value of 45 of sand equivalent of the fine aggregate Plasticity index be less than 4%	AASHTO T176 BS 1377-2
Particle shape	The flakiness index of the course aggregates be less than 25%	BS 812-105: 1989
Aggregate strength	TFV should be not less than 110 kN and wet/dry ratio not less than 75%	BS 812-111:1990
Absorption	Water absorption of course aggregates not exceed 2% by mass	BS 812-2:1995
Soundness	The sodium soundness be less than 12%	AASHTO T104
Coating and stripping test	The coating and stripping of the coarse aggregates be greater than 95%	AASHTO T182

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

Mix design Requirements for Asphaltic Mixes

The properties and performance of asphalt mixtures are influenced by the choice and properties of materials to be used in the mix and the employed mix design method. The review has found that currently two mixture design methods that are used widely in the EAC partner states are Marshall and Super-pave mix design methods. In accordance with these mix design methods, we recommend the materials and mix design requirements for asphaltic mixes presented in Table E14.

Table E14: Material requirements for asphalt concrete mixes

(a) Aggregate gradation requirements for asphalt concrete surfacing employing Marshall Mix Design method

Sieve size (mm)	Asphalt concrete continuously graded		
	AC20	AC14	AC10
20	80-100	100	-
14	60-80	85-100	100
10	50-70	72-94	85-100
5.0	36-56	52-72	55-72
2.36	28-44	37-55	38-57
1.18	20-34	26-41	27-42
0.600	15-27	16-28	18-32
0.300	10-20	12-20	13-23
0.150	5-13	8-15	9-16
0.075	2-6	4-10	4-10
Aggregate	95%	94.5%	94%
Bitumen content ASTM 2172-88	5%	5.5%	6%

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(b) Marshall Mix Design requirement for asphalt concrete surfacing

Property of mixture and laboratory test method			AC20, AC14 & AC10
Marshall flow (mm)			2 – 4
Marshall stability at refusal compaction (N), all severely loaded areas			Min. 9000
Marshall stability (2 × 75 blows) (N)	Traffic loads	>10 × 10^6 esal	8000 - 18000
		1 - 10 × 10^6 esal	7000 - 15000
		<1 × 10^6 esal	6000 - 10000
Air voids (%)			3 – 5
Voids in mineral aggregate (%)		AC20	Min. 14
		AC14	Min. 15
		AC10	Min. 16
Voids filled with bitumen (%)	Traffic loads	>10 × 10^6 esal	65 – 75
		1 - 10 × 10^6 esal	65 – 78
		<1 × 10^6 esal	70 – 80
Requirement after refusal laboratory compaction to BS 594 (severely loaded areas only)			Min. air voids 3%
Indirect tensile strength (kPa) AASHTO T 283			Min. 800 tested at 25^0C
Indirect wet tensile strength (kPa) AASHTO T 283			80% of dry strength

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(c) Grading limits for combined aggregates for asphalt concrete surfacing courses employing Super-pave Mix Design method

Sieve size (mm)	19 mm Nominal size				12.5 mm Nominal size				9.5 mm Nominal size			
	Control points		Restricted zone*		Control points		Restricted zone*		Control points		Restricted zone*	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
25	100	-	-	-	-	-	-	-	-	-	-	-
19	90	100	-	-	100	-	-	-	-	-	-	-
12.5	-	90	-	-	90	100	-	-	100	-	-	-
9.5	-	-	-	-	-	90	-	-	90	100	-	-
4.75	-	-	-	-	-	-	-	-	-	90	-	-
2.36	23	49	34.6	34.6	28	58	39.1	39.1	32	67	47.2	47.2
1.18	-	-	22.3	28.3	-	-	25.6	31.6	-	-	31.6	37.6
0.600	-	-	16.7	20.7	-	-	19.1	23.1	-	-	23.5	27.5
0.300	-	-	13.7	13.7	-	-	15.5	15.5	-	-	18.7	18.7
0.075	2	8	-	-	2	10	-	-	2	10	-	-
	*) The Restricted Zone forms a band through which the gradation may not pass											
	Bitumen content for bidding purposes is 5.0%				Bitumen content for bidding purposes is 5.5%				Bitumen content for bidding purposes is 6.0%			

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(d): Requirements for Super-pave Mix Design Gyratory Compaction

Design traffic (esal $\times 10^6$)	$N_{initial}$	N_{design}	$N_{maximum}$
< 0.3	6	50	75
0.3 - < 3	7	75	115
3 - < 30	8	100	160
≥ 30	9	125	205

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(e): Design requirements for Super-pave Mix Design

Property of mixture and laboratory test method			Asphalt concrete, continuously graded (37.5, 25, 19, 12.5 & 9.5 Nominal Size)
Required relative density (% of G_{mm})	$N_{initial}$	$<0.3\times10^6$ esal	<91
		$0.3 - <3 \times10^6$ esal	<90
		$3 - <30 \times10^6$ esal	<89
		$\geq30 \times10^6$ esal	<89
	N_{des}	All traffic loads	96
	N_{max}	All traffic loads	<98
Voids in mineral aggregate (%)	Aggregate size	37.5 mm	11.0
		25.0 mm	12.0
		19.0 mm	13.0
		12.5 mm	14.0
		9.5 mm	15.0
Voids filled with bitumen (%)		$<0.3\times10^6$ esal	70 – 80
		$0.3 - <3 \times10^6$ esal	65 – 78
		$3 - <30 \times10^6$ esal	65 – 75
		$\geq30 \times10^6$ esal	65 – 75
Filler to binder ratio			0.6 – 1.2
Target air voids at Ndes			4%
Indirect dry tensile strength (kPa) AASHTO T 283			Minimum 800 tested at 25^0C
Indirect wet tensile strength (kPa) AASHTO T 283			80% of dry strength

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

Mix design Requirements for hot mixed bituminous base and binder courses

Hot mixed bituminous courses in the existing specifications for EAC Partner States are: DBM40 and DBM30 are common in Kenya, Tanzania and Uganda while LAMBS are specified for Tanzania and DBM courses (with 37.5 mm nominal size of aggregates) and binder course (with 25 mm nominal aggregate sizes) employing Super-pave Mix Design Method are specified for Uganda. In view of the review, comparison and discussion about hot mixed asphalt base course design given in Annex A4, we present the materials and mix design requirements for dense bitumen macadam in Table E15.

Table E15: Requirements for mix design and materials for dense bitumen macadam

(a): Grading requirements for Dense Bitumen Macadam employing Marshall Mix Design

Sieve size (mm)	DBM40 (% passing)	DBM30 (% passing)
50	100	-
37.5	95-100	100
28	70-95	90-100
20	-	70-95
14	56-76	58-82
10	53-70	52-73
5.0	39-56	40-56
2.0	24-40	24-40
1.18	19-35	19-35
0.425	9-25	9-25
0.300	7-21	7-21
0.075	2-9	2-9

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(b): Design requirements for DBM course employing Marshall Mix design

Property of mixture and laboratory test method			DBM 40 & DBM 30
Marshall flow (mm)			2-4
Marshall stability at refusal compaction (N), all severely loaded areas			Min 9000
Marshall stability (2×75 blows) (N)	Traffic loads	$>10\times10^6$ esal	Min 7000
		1.5-10 $\times10^6$ esal	Min 6000
		$<1\times10^6$ esal	Min 3500
Air voids (%)			4-8
Compaction levels	Traffic loads	Severely loaded areas*	To refusal**
		$>10\times10^6$ esal	To refusal**
		1.5-10 $\times10^6$ esal	2×75
		$<1\times10^6$ esal	2×50
*Severely loaded areas should be identified in the Special Specifications			
**Requirement after refusal laboratory compaction to BS 594. Minimum air void 3%.			

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

(c): Grading requirements for asphaltic base course and binder course employing Super-pave Mix Design

Sieve size (mm)	37.5 mm Nominal size				25 mm Nominal size			
	Control points		Restricted zone*		Control points		Restricted zone*	
	Min	Max	Min	Max	Min	Max	Min	Max
50	100	-			-	-		
37.5	90	100			100	-		
25	-	90	-	-	90	100	-	-
19	-	-	-	-	-	90	-	-
12.5	-	-	-	-	-	-	-	-
9.5	-	-	-	-	-	-	-	-
4.75	-	-	34.7	34.7	-	-	39.5	39.5
2.36	15	41	23.3	27.3	19	45	26.8	30.8
1.18	-	-	15.5	21.5	-	-	18.1	24.1
0.600	-	-	11.7	15.7	-	-	13.6	17.6
0.300	-	-	10	10	-	-	11.4	11.4
0.075	0	6	-	-	1	7	-	-
	*) The Restricted Zone forms a band through which the gradation may not pass							
	Bitumen content for bidding purposes is 4.0%				Bitumen content for bidding purposes is 4.5%			

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005)

Mix design Requirements for penetration macadam base course

Penetration macadam base course is only specified in the specifications for Tanzania. Other EAC Partner States have not included this type of base course in their specifications. We recommend the materials and mix design requirements in Table E16 for penetration macadam base course.

Table E16: Requirements for penetration macadam base course

Sieve size (mm)	Penetration macadam		
	PM80	PM60	PM30
Main fraction			
100	100	-	-
75	75-100	100	-
63	-	80-100	-
50	0-50	0-50	-
37.5	0-25	0-25	100
28	0-5	0-5	80-100
20	-	-	0-50
14	-	-	0-25
10	-	-	0-5
Key stone*			
50	100	-	-
37.5	85-100	100	-
28	0-50	85-100	-
20	0-25	0-50	100
14	0-5	0-25	85-100
10	-	0-5	0-55
6.3	-	-	0-25
5	-	-	0-10
Layer thickness (mm)	125	100	50
Bitumen type	Penetration grade 80/100 or 60/70		
Bitumen spray rate for bidding purposes (l/m2)	3.5	3.5	2.5

Adopted from specification for Tanzania

Mix design Requirements for cold mixed bituminous base course

Unlike the penetration macadam base course, cold mixed bituminous base course has been specified in the specifications for Kenya, Tanzania and Uganda. Specifications for cold mixed bituminous base course for Tanzania and Uganda are similar. We recommend specifications Table E17 for mix design of FBMIX and BEMIX cold mixed bituminous base courses to the EAC partner states.

Table E17: Specifications for cold mixed bituminous base courses

Sieve size (mm)	FBMIX (% passing)	BEMIX (% passing)
37.5	100	100
28	80-100	80-100
20	60-95	60-95
10	42-78	35-70
5.0	30-65	25-50
2.0	20-50	18-35
0.425	10-30	10-25
0.075	5-15	5-8
Properties of mixture and laboratory test method	**FBMIX**	**BEMIX**
Bitumen type	Foamed	Emulsion
Nominal consumption of bitumen (litres/m^3 of compacted material) for bidding purpose	90	90
Marshall stability tested at 400C (N), BS EN 12591:2000	6000	4500
Marshall flow tested at 400C (mm), ASTM D 1559-89	2-4	2-4
Minimum E-modulus tested at 290C by indirect tensile strength method or alternative approved methods (MPa), ASTM D 3967	1600	1200
Minimum moisture content at the time of laying (%)	Mix design moisture less 1.5% points	
Maximum moisture content at the time of laying (%)	Mix design moisture plus 0.5% points	
Minimum compacted field density (% of Marshall dry density)	96	

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Surface Dressings

a) *Bituminous binders*

In view of the technical advantages associated with the use of emulsions, we recommend the use of emulsions in surface dressing operations to the EAC region.

b) Requirements for surface dressings

Review of the available specifications revealed that aggregates for surface dressing should consist of clean, tough, durable fragments of crushed stone free from any deleterious matter and comply with the requirements for grading, particle shape, aggregate strength, and average least dimension (ALD). We recommend the specifications given in Tables E18 and E19 for single and double surface dressings, respectively, to the EAC partner states.

Table E18: Requirements for single surface dressings

Chippings				
Sieve size (mm)	**Chipping, nominal size of aggregates (% passing)**			
	20 mm	**14 mm**	**10 mm**	**7 mm**
25	100			
20	85-100	100		
14	0-30	85-100	100	
10	0-5	0-30	85-100	100
6.3	-	0-5	0-30	85-100
5	-	-	0-5	0-30
2.36	-	-	-	0-5
Fines: 0.425	<0.5	<1.0	<1.0	<1.5
Dust: 0.075	<0.3	<0.5	<0.3	<1.0

Nominal Sizes	Max flakiness index (FI)	TFV (kN)	Ratio TFV soaked to dry (%)	ALD	Binder hot spray rate (l/m^2)	Aggregate Spread rate	Test Method
20	20	For AADT>1000:160 For AADT<1000:120	75	11 - 15	1.80	19 kg/m^2	BS812:105.1 & BS812:111
14	25			8 - 10	1.50	14 kg/m^2	
10	25			5 - 6.5	1.10	11 kg/m^2	
7	30			2 - 4	0.80	7 kg/m^2	

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Table E19: Requirements for double surface dressings

Chippings				
Sieve size (mm)	**Chipping, nominal size of aggregates (% passing)**			
	20 mm	**14 mm**	**10 mm**	**7 mm**
25	100			
20	85-100	100		
14	0-30	85-100	100	
10	0-5	0-30	85-100	100
6.3	-	0-5	0-30	85-100
5	-	-	0-5	0-30
2.36	-	-	-	0-5
Fines: 0.425	<0.5	<1.0	<1.0	<1.5
Dust: 0.075	<0.3	<0.5	<0.3	<1.0

Nominal Size of aggregate (mm)	**Max flakiness index (FI)**	**TFV (kN)**	**Ratio TFV soaked to dry (%)**	**ALD**	**Nominal rates of application**		**Test Method**
					Binder spray rate (l/m²)	**Aggregate Spread rate**	
20	20	For AADT>1000:160 For AADT<1000:120	75	11-15	1.8 for AADT<200	19 kg/m²	BS812:105.1 & BS812:111
					1.5 for AADT 200-1000		
					1.2 for AADT>1000		
14	25			8-10	1.4for AADT<200	14 kg/m²	
					1.1for AADT 200-1000		
					1.0 for AADT>1000		
10	25			5-6.5	1.2 for AADT<200	11 kg/m²	
					1.0 for AADT 200-1000		
					0.9 for AADT>1000		
7	30			2-4	0.9 for AADT<200	7 kg/m²	
					0.8 for AADT200-1000		
					0.7 for AADT>1000		

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Sand Seals and Slurry

The specifications for Kenya, Tanzania and Uganda give almost similar specifications for binder for sand seals and slurry with regard to its type and required properties, and this is particularly so for the case of specifications for Tanzania and Uganda. Likewise, specifications for Tanzania and Uganda recommend crusher dust or clean river sand free from lumps of clay or any other deleterious matter as aggregate for sand seal, and it should comply with the requirements in the table below. We therefore recommend requirements presented in Table E20 for binders and aggregates for sand and slurry seals to the EAC region. Additionally, aggregate for slurry seals should be approved crusher sand obtained from a parent rock having a TFV value of not less than 110 kN or a mixture of such crusher sand and approved clean natural sand, where the mixture does not contain more than 25% of natural sand.

Table E20: Requirements for sand and slurry seals

(a) Recommended binders for sand and slurry seals

Sand Seal	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)
Slurry Seal	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

(b) Grading limits requirements for sand seal

Sieve size (mm)	**Percentage passing sieve (% by mass)**	
	Natural river sand	**Crusher dust**
10	100	100
5	85 – 100	85 – 100
1.18	20 – 60	20 – 80
0.425	0 – 30	-
0.300	0 – 15	-
0.150	0 - 5	0 - 30

c) Grading limits requirements for slurry seal

Sieve size (mm)	**Percentage passing sieve (% by mass)**	
	Fine type	**Coarse type**
10		100
5	100	85-100
2	90-100	50-90
1.18	60-90	32-70
0.425	32-60	20-44
0.150	10-27	7-20
0.075	4-12	2-8

Adopted from: General Specifications for Road and Bridge Works, Uganda (2005) and Standard Specifications for Road Works, Tanzania (2000)

Impact of Harmonisation of Specifications for Road Works

The member countries do have specifications of their own. These proposed specifications are intended to provide the EAC members states Road Agencies with a quick reference to the use of specifications roadworks. It is intended that the provisions of these standards be a reference and not the final authority and as such the proposed specifications do not have a direct impact on the performance of existing road pavements. It is advised that member states should be making reference to the proposed specifications in the design, maintenance and rehabilitation of roads that form the regional trunk road network for uniform performance of this network. It can be stated that through harmonisation of specifications for road and bridge works optimum balance between road construction costs, properties of availability materials, environmental considerations, construction method, and previous experience of the performance of materials under consideration can be achieved during materials selection and design for roadworks.

In developing the proposed harmonization of specifications for road works across the member states we have taken account of the current specifications and the best practices. Adoption of the recommendations means that the member states will be using best practices in a consistently and the benefits of using best practices will be achieve uniformly in the region. Member states are therefore encouraged to revise their specifications within the next three to five years to ensure uniform realization of the benefits of adopting the best practices in the region.

4 HARMONISATION OF SPECIFICATIONS FOR ROADWORKS

4.1 Introduction

4.1.1 General

The Standard Specifications for Road and Bridge Works are the specifications developed and adopted or followed by EAC Partner States for highway construction and improvement contracts. They are incorporated into the written agreement (Contract) between the Contracting Agency and the Contractor, except where the Contract indicates that a particular specification has been amended or replaced with a special provision to resolve project-specific issues. The decision to amend or replace any standard specification with a special provision is made during the design process and is based upon the sound engineering judgment of the project designer. They are to be used only as a guideline during the design process, and once they have been incorporated into a Contract, however, they become the legal and enforceable part of the Contract. Generally, they are meant to outline the general requirements and legal agreements applicable to all highway construction and improvements as well as provisions relating to:

- General aspects which cover definitions and terms, general requirements and provisions, contractor's establishment on site and general obligations, engineer's accommodation and attendance upon engineer and his site personnel, accommodation of traffic, overhaul and environmental protection and waste disposal
- Drainage
- Earthworks and pavement layers and seals
- Ancillary road works
- Structures
- Tolerances, testing and quality control
- Improvements
- Construction methods
- Equipment
- Materials (requirements, test methods, tests on completed works and construction control tests)

The main focus of this chapter, however, is to make recommendations about harmonisation of the latter aspect of the Standard Specifications for the EAC region.

The chapter therefore reviews and contrasts existing Standard Specifications for road works within the EAC member countries makes recommendations for the EAC.

4.1.2 Study Objectives

The overall objective of the assignment is to make it possible to have a reliable, efficient and safe road transport services. This chapter gives recommendations about the specifications for road and bridge works for the EAC region. The chapter therefore discusses established specifications for road and bridge works within the EAC member countries as well as applicable SADC and other international standards, and makes recommendations for the EAC.

4.2 Specifications for Road and Bridge Works Adopted by EAC Partner States

Specifications for Burundi and Rwanda are based on the French standards while Kenya, Tanzania and Uganda are using their own specifications. Table 1 provides a summary of specifications that are used by EAC Partner States.

Table 1: Specifications for Road and Bridge Works in EAC Partner States

Country	Specification	Year	Remarks
Burundi	French (AFNOR) specifications		-
Kenya	Standard Specifications for Road and Bridge Construction	1986	New standards and specifications are currently under preparation
Rwanda	French (AFNOR) specifications		-
Tanzania (Mainland and Zanzibar)	Standard Specification for Road works	2000	-
Uganda	General Specifications for Road and Bridge Works	2005	-

A review of the specifications listed in Table 1 revealed a number of areas of commonality as well as areas unique to particular countries. A critical examination of the specifications followed by Kenya, Tanzania and Uganda revealed that the majority of materials tests specifications and procedures in the materials specifications are governed by strict procedures that, in the main, have been originally derived from British (BS), American (ASTM/ AASHTO) Standards. In most cases they have been incorporated into existing EAC Partner States standards.

Generally, there are significant problems associated with harmonising and developing general guidelines for pavement construction materials where multi-standard systems operate in a region. These problems may be seen as falling within two overlapping areas of concern. Firstly, there are problems concerned with differing test procedures. Secondly, and more fundamentally, there are difficulties associated with extrapolating test procedures and methods of analysis from a temperate environment where the original testing procedures were developed for to one places like the EAC region where tropical climates and road environments dominate are different from the former region. Table 2 summarises some common problems.

Table 2: Problems Posed by Application of Conflicting Multi-Standards

Potential problem	Typical example
Correlation between similar tests from different standards that may have apparently slight but significant different aspects	The maximum sand size in ASTM/AASHTO particle size distribution tests is defined as 4 m as opposed to the BS Standard 2 mm.
Correlation between similarly named test with differing procedures	Comparison between liquid limit test performed with drop cone and those undertaken using the Cassegrande apparatus

On the other hand, in the longer term, the development and adoption of European Standards (EN Series) will pose additional problems with respect to EAC Partner States specifications linked to British or other European National Standards as these become redundant. The documents listed in Table 1 are further discussed in the following section.

4.3 Potential Areas for Harmonisation and Improvements

4.3.1 Earthworks

Table 3 presents earthworks material type, use and compaction requirements as specified in design manuals for respective EAC Partner States. G15, G7, G3 and geotexile and/or rock fill (DR) materials are the most common materials for earthworks. The manuals for Kenya and Uganda recommended use of rock fill materials in swampy areas while this measure is not covered in the manual for Tanzania. Further, Tanzania and Uganda recommend the use of geotextile materials. As shown in Table 3, Tanzania and Uganda have common compaction requirements for earthwork materials.

Table 3: Earthworks Compaction Requirements

Layer and typical material type specified	**Minimum dry density, nominal value**		
	Kenya	Tanzania	Uganda
Hard material Soft material - waterlogged and swampy material	95% MDD, 100% MDD for 300 mm in cuttings/embankments		
Upper improved subgrade layer, nominally G15 material or better, for layers less than 150 mm below the formation level		95% of BS-Heavy	95% of BS-Heavy
Lower improved sub grade layer, nominally G7 material or better, for layers from 150 mm to 300 mm below the formation level		93% of BS-Heavy	93% of BS-Heavy
Fill, nominally G3 material or better, for layers more than 300 mm below the formation level		90% of BS-Heavy	90% of BS-Heavy
Fill or improved sub grade layers using rock fill (**DR**)		Compaction method specification shall apply	Compaction method specification shall apply
Roadbed compaction to 150 mm depth after clearing, grubbing and removal of topsoil or other unsuitable material, where the roadbed level is:			
Less than 150 mm below the formation level		97% of BS Heavy	97% of BS Heavy
150 mm to 300 mm below the formation level		95% of BS Heavy	95% of BS Heavy
300 mm to 600 mm below the formation level		93% of BS Heavy	93% of BS Heavy
More than 600 mm below the formation level		100% of BS Light	100% of BS Light

Table 4: Earthworks Material type, compaction requirements and test methods

Country	Material Type	Requirements				
		CBR (%)	**CBR Swell (%)**	**PI (%)**	**Maximum particle size**	**Maximum layer thickness**
Tanzania (Mainland and Zanzibar)	G15	In wet/moderate zones min. 15 after 4 days soaking and min. 15 at OMC and also minimum 7 after 4 days soaking in dry zones	Max. 1.5	25	2/3 of compacted layer thickness	250 mm compacted thickness placed in one operation
	G7	In wet/moderate zones min. 7 after 4 days soaking and min. is 7 at OMC and also min. 3 after 4 days soaking in dry zones	Max. 2	30	2/3 of compacted layer thickness	250 mm compacted thickness placed in one operation
	G3	3 after 4 days soaking, measured at 90% of MDD of BS-Heavy compaction	Max. 2	-	2/3 of compacted layer thickness	250 mm compacted thickness placed in one operation
	DR				Two thirds of compacted layer	1 m placed in one operation
Uganda	G15	Minimum 15 after 4 days soaking	Max. 1.5	25	1/2 of compacted layer thickness but not > 50 mm	250 mm compacted thickness placed in one operation
	G7	Minimum 7 after 4 days soaking	Max. 2	30		
	G3	3 after 4 days soaking, measured at 90% of MDD of BS-Heavy compaction	Max. 2	-		
	DR				Two thirds of compacted layer	1 m placed in one operation

Further, it can be seen from Table 4 that the design manuals for Tanzania and Uganda set threshold values of CBR, CBR swell, PI, maximum particle size and maximum layer thickness as requirements for earthworks while the manual for Kenya specifies only compaction requirements (Table 3) for earthworks. Compacted layer thickness requirement is the same in the manuals for Tanzania and Uganda, i.e. 250 mm placed in one operation while specifications for Kenya recommended two compacted layers of 150 mm thickness each.

The maximum particle size specified in Tanzania manual is 2/3 of compacted layer while it is ½ of compacted layer (but not > 50 mm) for Uganda manual. The manual for Tanzania specifies CBR values according to the climatic zones while this aspect is not considered in the design manual for Uganda. On the other hand, the design manual for Kenya did not include requirements for CBR values and PI.

Recommendation

EAC Partner States adopt materials requirements shown in Table 5 for fill and improved sub grade layers for earthworks.

Table 5: Requirements for fill and improved sub grade layers

<table>
<tr><th rowspan="2">Material Type</th><th colspan="5">Requirements</th></tr>
<tr><th>CBR (%)</th><th>CBR Swell (%)</th><th>PI (%)</th><th>Maximum particle size</th><th>Maximum layer thickness</th></tr>
<tr><td>G15</td><td>Minimum 15 after 4 days soaking</td><td>Max. 1.5</td><td>25</td><td rowspan="3">1/2 of compacted layer thickness but not > 50 mm</td><td rowspan="3">250 mm compacted thickness placed in one operation</td></tr>
<tr><td>G7</td><td>Minimum 7 after 4 days soaking</td><td>Max. 2</td><td>30</td></tr>
<tr><td>G3</td><td>3 after 4 days soaking, measured at 90% of MDD of BS-Heavy compaction</td><td>Max. 2</td><td>-</td></tr>
<tr><td>DR</td><td></td><td></td><td></td><td>Two thirds of compacted layer</td><td>1 m placed in one operation</td></tr>
</table>

4.3.2 Pavement Layers of Natural Gravel Materials

As covered in Chapter 5, the general specifications for Tanzania and Uganda cover the requirements for GW, G80, G60, G45, G30, G25 materials. Such materials codes are missing in the general specifications for Kenya. Table 6 summarises the requirements for unbound natural gravel materials as specified in the current manuals of EAC Partner States. It can be seen from Table 6 that the material properties for which the minimum and maximum requirements are set include grading, CBR, Atterberg limits, shrinkage limits, particle strength, and compaction. In comparison to the specifications for Kenya, the current specifications for Tanzania and Uganda cover a wide range of these materials properties. Both specifications for Tanzania and Uganda provide the same grading envelope for G80 materials. Additionally, Uganda specifications give grading envelope for G60 materials as well.

Table 6: Minimum requirements for pavement layers of natural gravel material in EAC Partner States

Country	**Material Type**	**Requirements**													
		Compaction	CBR	CBR swell (%)	Max. LL	% passing 37.5 mm sieve	Shrinkage product, SP	Grading coefficient (GC0	Max. PI	Max. LS	TFV	Plasticity modulus (%)	LAA (%)	ACV (%)	Uniformity coefficient (%)
Burundi	Follows French Standards														
Kenya	GW	95% MDD	Min 20						s/specs						
	Subbase	95% MDD	30						15			Max 250			Min 5
	Base	95% MDD	80						15			Max 250	Max 50	Max 35	
Rwanda	Followed French Standards and now AASHTO guide														
Tanzania (Mainland and Zanzibar)	GW	95% of BS-Heavy	Min 25/25[1]			Min. 95	120 - 400	16 - 34							
	G80[a]	98% of BS-Heavy	Min 80/80&60[2]	0.5	30/40[3] (35/45[4])				8/14[3] (10/16[4])	4/7[3] (5/8[4])	80kN[5] 60%[6]				
	G60[a]	98% of BS-Heavy	Min 60/60&45[2]	1.0	35/45[3] (40/45[4])				10/16[3] (12/18[4])	5/6[3] (6/9[4])	50kN[5] 60%[6]				
	G45[b]	95% of BS-Heavy	Min 45/45&25[2]	0.5	40/45[3] (45/50[4])				14/18[3] (16/20[4])	7/9[3] (8/10[4])					
	G25[c]	95% of BS-Heavy	Min 25/25&15[2]	1.0	45/50[3] (45/55[4])				16/20[3] (18/24[4])	8/10[3] (9/12[4])					
Uganda	GW	95% of BS-Heavy	Min 25[d]		-	Min. 95	120 - 400	16 - 34							
	G80[a2]	98% of BS-Heavy	Min 80 [d]	0.5	30[3]/35[4]				8[3]/10[4]	4[3]/5[4]	80kN[5] 60%[6]				
	G60[a2]	98% of BS-Heavy	Min 60[d]	1.0	35[3]/40[4]				10[3]/12[4]	5[3]/6[4]	50kN[5] 60%[6]				
	G45[b2]	95% of BS-Heavy	Min 45 [d]	0.5	40[3]/45[4]				14[3]/16[4]	7[3]/8[4]					
	G30[d2]	95% of BS-Heavy	Min 30 [d]	1.0	45[3]/45[4]				16[3]/18[4]	8[3]/9[4]					

[a] The largest particles d_{max} is 2/3 of compacted layer thickness and minimum grading modulus (GM) is 2.0
[a2] d_{max} is ½ of compacted layer thickness but not > 50 mm and GM is 2.0
[b] Minimum grading modulus (GM) is 1.5
[b2] GM is 1.5
[c] Minimum grading modulus (GM) is 1.2
[c2] GM is 1.2
[d] After 4 days of soaking
[1] Wet or moderate after 4 days soaking/Dry climatic zones at OMC; [2] Wet or moderate after 4 days soaking /Dry climatic zones at OMC and also after 4 days soaking
[3] General requirements; [4] Calcrete or other pedogenic materials
[5] Minimum TFV dry; [6] Ratio soaked to dry value of TFV

The CBR and Atterberg limits requirements for Tanzania are given with respect to climatic zones while specifications for Kenya and Uganda do not address the climatic zone aspect. The largest particle size recommended by the specifications for Tanzania is 2/3 of compacted layer thickness for G80 and G60 while in Uganda it is ½ of compacted layer thickness but it is recommended that this should not be greater than 50 mm. The requirements for G30 for Uganda differ from those of G25 for Tanzania in terms of CBR value and Atterberg limits. Since the specifications for Uganda are the most updated and stringent ones, it is proposed to drop G25 material from the specifications for Tanzania. The specifications for Kenya, which are the oldest compared to others, provide different materials requirements for subbase, base course GW materials as compared to the same material requirements for Tanzania and Uganda.

Recommendation

EAC Partner States adopt materials requirements shown in Tables 7 and 8 for pavement layers of natural gravels.

Table 7: Materials requirements for gravel wearing course, base and subbase layers

Material Type	**Requirements**									
	Compaction	CBR	CBR swell (%)	Max. LL	% passing 37.5 mm sieve	Shrinkage product, SP	Grading coefficient (GC0	Max. PI	Max. LS	TFV
GW	95% of BS-Heavy	Min 25[d]			Min. 95	120 - 400	16 - 34			
G80[a2]	98% of BS-Heavy	Min 80 [d]	0.5	30[3]/35[4]				8[3]/10[4]	4[3]/5[4]	80kN[5]
G60[a2]	98% of BS-Heavy	Min 60[d]	1.0	35[3]/40[4]				10[3]/12[4]	5[3]/6[4]	60%[6]
G45[b2]	95% of BS-Heavy	Min 45 [d]	0.5	40[3]/45[4]				14[3]/16[4]	7[3]/8[4]	50kN[5]
G30[d2]	95% of BS-Heavy	Min 30 [d]	1.0	45[3]/45[4]				16[3]/18[4]	8[3]/9[4]	60%[6]

[a2] d_{max} is ½ of compacted layer thickness but not > 50 mm and GM is 2.0
[b2] GM is 1.5
[c2] GM is 1.2
[d] After 4 days of soaking
3 General requirements; **4** Calcrete or other pedogenic materials
5 Minimum TFV dry; **6** Ratio soaked to dry value of TFV

Table 8: G80 and G60 materials grading requirements

Grading, BS 1377: Part 2		
Sieve size (mm)	G80 grading limits (% passing sieve)	G80 grading limits (% passing sieve)
63	100	100
37.5	80 – 100	80 – 100
20	60 – 95	60 -95
5	30 – 65	30 – 65
2	20 – 50	20 – 50
0.425	10 – 30	10 – 30
0.075	5 -15	5 - 15

4.3.3 Stabilisation

The stabilising agents, requirements for materials in cemented layers and maximum time for the cementation/modification process reported in the specifications for EAC Partner States are as shown in Tables 9, 10, and 11, respectively.

Table 9: Stabilising agents

Stabilising Agent
Hydraulic road binder
Cement
Blast-furnace cement
Other chemical stabilising agents
Milled-blast furnace slag

Table 10: Construction limitations

Stabilising agent and material class	Maximum time for completion after stabilising agent comes in contact with the material to be stabilised
Cement, C2, C1.5, C1.0, C0.7, CM	4 hours
Lime, C2, C1.5, C1.0, C0.7	8 hours
Lime, CM	48 hours

The maximum particle size in the Tanzania manual is 2/3 of compacted layer thickness while it is ½ of compacted layer thickness but not >50 mm in Uganda.

The specifications for Uganda include material classes C1.5 and C0.7 which are neither in the specifications for Tanzania manual nor for Kenya specifications. The former specifications also give material properties such as minimum CBR (%) for lime or liquid ionic stabilised material, organic content and sulphate content before stabilisation while plasticity index requirement after stabilisation is only included in specifications for Tanzania.

On the other hand, the 1986 specifications for Kenya do not cover requirements for materials in cemented layers, selection of stabiliser and the maximum time for completion after stabilising agent comes in contact with the material to be stabilised. The manual recommends thickness limitations for which the compacted thickness of base layer should not be less than 3 times the maximum particle size of the material and compacted thickness of any subbase layer not to be less than twice the maximum particle size of the material. The minimum density for all lime and cement treated materials is 95% MDD (AASHTO T180) and moisture content at the time of compaction is recommended to be between 95% and 105% of Optimum Moisture Content (AASHTO T180). It should be noted that Kenya followed AASHTO guided which recommends soaking for 4 days while Tanzania and Uganda follow BS standards that recommend soaking for 4 hours.

Table 11: Requirements for materials in cemented layers

	Tanzania (Mainland and Zanzibar			Uganda			
Material properties	**Material class**						
	C2	**C1**	**CM**	**C1.5**	**C1.0**	**C0.7**	**CM**
Min CBR (%) for lime or liquid ionic stabilised material				80	60	45	30
After stabilisation							
Min UCS (MPa)	2	1	0.5	1.5	1	0.7	0.5
Plasticity Index (%)	Maximum 8						
Before Stabilisation							
Soaked CBR (%) at 95% MDD of BS-Heavy	Min 30	Min 20	No requirement	Min 30	Min 20	Min 20	No requirement
Plasticity Index (%)	Max 20	Max 25	Max 35	Max 20	Max 25	Max 25	Max 35
Aggregate strength TFV_{dry}	Min 50 kN	No requirement	No requirement	Min 50 kN	No requirement	No requirement	No requirement
Grading Modulus (GM)	Min 1.5	Min 1.2	No requirement	Min 1.5	Min 1.2	Min 1.0	No requirement
Organic Content				Max 0.5%	Max1%	Max 1%	Max 25
Sulphate (SO3) content				Max 0.25%			
Max. particle size	2/3 of compacted layer thickness			½ of compacted layer thickness but not > 50 mm			

Recommendation

EAC Partner States adopt materials requirements shown in Table 12 for cemented layers.

Table 12: Materials requirements for cemented layers

Material properties	**C2**	**C1.5**	**C1.0**	**C0.7**	**CM**
Min CBR (%) for lime or liquid ionic stabilised material		80	60	45	30
After stabilisation					
Min UCS (MPa)	2.0	1.5	1	0.7	0.5
Before stabilisation					
Soaked CBR (%) at 95% MDD of BS-Heavy	Min 30	Min 30	Min 20	Min 20	No requirement
Plasticity Index (%)	Max 20	Max 20	Max 25	Max 25	Max 35
Aggregate strength TFV_{dry}	Min 50 kN	Min 50 kN	No requirement	No requirement	No requirement
Grading Modulus (GM)	Min 1.5	Min 1.5	Min 1.2	Min 1.0	No requirement
Organic Content	Max 0.5%	Max 0.5%	Max1%	Max 1%	Max 25
Sulphate (SO3) content	Max 0.25%				
Max. particle size	½ of compacted layer thickness but not > 50 mm				

Selection of stabiliser

When a stabilising agent is selected the following factors need to be considered:

- The physical composition and properties of the material, for example liquid limit, plasticity index (PI), clay content, and initial consumption of lime.
- The purpose of stabilisation, for example modification or cementation and whether rapid or slow increase in strength is required.
- The relative availability and cost of the different stabilising agents.

Manuals for Tanzania and Uganda provide same guidelines to select a stabilising agent.

We therefore *recommend use of the following guidelines to select a stabiliser.*

% passing the 0.075 mm sieve BS 1377-2	*Plasticity index (%) BS 1377: Part 2*	*Best suited stabiliser*
Less than 25%	*PI is less than 6 or PI x (% passing 0.075 mm) is less than 60*	*Cement only*
	6 - 10	*Cement preferred*
	More than 10	*Cement and/or lime*
More than 25%	*Less than 10*	*Cement preferred*
	10 – 20	*Cement and/lime*
	More than 20	*Lime preferred*

4.3.4 Crushed aggregate Base Course

The 1986 specifications for Kenya recommend requirements of particle strength (LAA & ACV), cleanliness (SSS), particle shape (FI), and CR for crushed stone classes A, B, and C materials depending on where the materials are to be used (subbase or base course). Specifications for Tanzania and Uganda report requirements of Atterberg limits, grading, particle strength and shape and compaction for crushed stone or quarried rock (CRS) materials and crushed, fresh, quarried rock (CRR) materials. Tables 13 and 14 present a summary of crushed aggregate base course materials requirements specified in Tanzania and Uganda specifications. It can be seen that particle strength and shape requirements are same while there is slight difference in Atterberg limits specifications whereby liquid limit is dropped from Uganda specifications. Further, specifications for Uganda give more stringent CRR materials compaction requirements than those for Tanzania. Both manuals recommend maximum compacted thickness of any layer of crushed aggregate base course compacted in one process to be 200 mm.

For crushed rock to be viable as a pavement material, it must be available, workable, and give satisfactory field performance at the lowest possible cost. The final in-place cost should take into account supply and cartage costs, cost of repair to roads damaged during cartage, spreading and compaction costs and preparation for surfacing costs. An example of this is the comparison between a 20 mm and 40 mm nominal sized crushed rock. Although both of sufficient strength, the 40 mm crushed rock is likely to be less costly to supply and delivered to the roadbed than 20 mm crushed rock. However, after taking into account the difficulty in spreading and compacting 40 mm crushed rock and the need to avoid segregation, 20 mm crushed is far easier and therefore much cheaper to spread and compact making it a far more economical material to use. Further, crushed rock must be able to be easily spread and compacted on the roadbed to the required specified standards of shape, level, and density without undue breakdown of particles. Crushed rock produced from softer source rock makes fines during compaction and this should be taken into account when specifying the supply of the product to ensure excessive fines are not produced during final compaction. Excessive fines generated during compaction and during the life of the pavement will result in loss of strength leading to pavement distress which eventually will be exhibited in the condition and life of the sprayed seal surfacing. Crushed rock produced from granitic sources can produce compaction difficulties arising from the generation of coarse and harsh fine material during the compaction process. Such material is subsequently difficult to prepare for sealing.

Table 13: Requirements for crushed aggregate base course materials

Country	Material Type	Requirements								Test Methods
		Compaction	CBR	LL	Max. PI	Max. LS	TFV	Ratio TFV soaked to dry	Max. Flakiness Index	
Kenya	Subbase course Base course	<98% <96%	50 80	-	In s/specs 15	-	-		25-35	AASHTO T180
Tanzania (Mainland and Zanzibar)	CRS CRR	100% of BS-Heavy 88% of the aggregate's apparent density		35 30		4 3	110kN 110kN	60% 75%	35% 35%	CML 1.2, 1.4, 1.9, 2.2, 2.4, 2.7/BS 1377 Part 2 & 4, BS 812: Part 2, BS 812:Section 105.1:1989 and BS 812: Part 111
Uganda	CRS CRR	98% of BS-Heavy 102% of BS-Heavy			6 Non-plastic	4 3	110kN 110kN	60% 75%	35% 35%	BS 1377:Part 2, BS 812: Part 105 & BS 812 Part 111

Table 14: Grading limits for crushed aggregate base course materials for Tanzania and Uganda

Sieve size (mm)	Grading limits (% passing sieve)			
	CRS		CRR	
	Coarse type	Fine type	Coarse type	Fine type
50	100			
37.5	90 – 100	100	100	
28	75 – 95	90 – 100	87 – 97	100
20	60 – 90	65 – 95	75 – 90	87 – 97
10	40 – 75	40 – 70	52 – 68	62 – 77
5	29 – 65	29 – 52	38 – 55	44 – 62
2	20 – 45	20 – 40	23 – 40	27 – 45
1.18	17 – 40	15 – 33	18 – 33	22 – 38
0.425	12 – 31	10 – 24	11 – 24	13 – 27
0.075	5 – 12	4 – 12	4 – 12	5 - 12

Recommendation

EAC Partner States adopt requirements shown in Table 15 for crushed aggregate base course materials.

Table 15 requirements for crushed aggregate base course materials

Material Type	Requirements					
	Compaction	Max. PI	Max. LS	TFV	Ratio TFV soaked to dry	Max. Flakiness Index
CRS	98% of BS-Heavy	6	4	110kN	60%	35%
CRR	102% of BS-Heavy	Non-plastic	3	110kN	75%	35%

Sieve size (mm)	Grading limits (% passing sieve)			
	CRS		CRR	
	Coarse type	Fine type	Coarse type	Fine type
50	100			
37.5	90 – 100	100	100	
28	75 – 95	90 – 100	87 – 97	100
20	60 – 90	65 – 95	75 – 90	87 – 97
10	40 – 75	40 – 70	52 – 68	62 – 77
5	29 – 65	29 – 52	38 – 55	44 – 62
2	20 – 45	20 – 40	23 – 40	27 – 45
1.18	17 – 40	15 – 33	18 – 33	22 – 38
0.425	12 – 31	10 – 24	11 – 24	13 – 27
0.075	5 – 12	4 – 12	4 - 12	5 - 12

4.3.5 Bituminous Layers and Seals

4.3.5.1 Prime and Curing Membrane

Tables 16 and 17 present a summary of requirements for materials for priming and aggregates for blinding, respectively. As shown in the tables, Tanzania follows AASHTO standards whereas Uganda uses BS EN series of compliancy standards.

Table 16: Requirements for materials for priming

Country	Material Type	Compliance standards
Burundi		
Kenya	MC-30 cut-back bitumen MC-70 cut-back bitumen	- -
Rwanda		
Tanzania (Mainland and Zanzibar)	MC-30 cut-back bitumen MC-70 cut-back bitumen Invert bitumen emulsion	AASHTO designation M82-75 AASHTO designation M82-75 SABS 1260
Uganda	MC-30 cut-back bitumen MC-70 cut-back bitumen Bitumen emulsion	BS EN 12591:2000 BS EN 12591:2000 BS 432 or SABS 308

It can be noted from Table 17 that a requirement of 100% passing the 6.3 mm sieve and not more than 10% passing the 2.36 mm sieve makes the specifications for aggregates for blinding for Uganda stricter than those for Tanzania. Other than the above noted divergences, the rest of the requirements adopted by Tanzania and Uganda for priming and aggregate materials are similar.

Table 17: Prime and Aggregate Materials Requirements

Country	Material Type	Requirements			
		Aggregate for blinding	Application Rate	Temperature (^0C)	
				Storage	Spraying
Burundi					
Kenya	MC-30 cut-back bitumen MC-70 cut-back bitumen	-	-	-	-
Rwanda					
Tanzania (Mainland and Zanzibar)	MC-30 cut-back bitumen MC-70 cut-back bitumen Invert bitumen emulsion	Crushed rock or river sand (min. 85% pass. 6.3 mm and no particles larger than 10 mm)	0.4 – 2.7 l/m^2	60/40[1] 80/50[1] ambient	45-60 60-80 Ambient above 10^0C
Uganda	MC-30 cut-back bitumen MC-70 cut-back bitumen Invert bitumen emulsion	Crushed rock or river sand (100% pass. 6.3 mm and should not have more than 10% passing the 2.36 mm sieve)	0.4 – 2.7 l/m^2 and 1.0 l/m^2 for bidding purposes	60/40[1] 80/50[1] ambient	45-60 60-80 Ambient above 10^0C

[1] Up to 24 hrs/Over 24 hrs

Recommendation

EAC Partner States adopt material requirements shown in Table 18 for binder and aggregates for priming and blinding the primed surface.

Table 18**:** Requirements for priming and aggregate for blinding

Material Type	Requirements				
	Compliance standards	Aggregate for blinding	Application Rate	Temperature (^{0}C)	
				Storage	Spraying
MC-30 cut-back bitumen MC-70 cut-back bitumen Invert bitumen emulsion	BS EN 12591:2000 BS EN 12591:2000 BS 432 or SABS 308	Crushed rock or river sand (100% pass. 6.3 mm and should not have more than 10% passing the 2.36 mm sieve)	0.4 – 2.7 l/m^2 and 1.0 l/m^2 for bidding purposes	60/40[1] 80/50[1] ambient	45-60 60-80 Ambient above 10^{0}C

[1] Up to 24 hrs/Over 24 hrs

4.3.5.2 Bituminous Base Course and Asphalt Concrete Surfacing

This section discusses types of bituminous base course and asphalt concrete surfacings. Table 19 shows types of bituminous base course and asphalt concrete surfacings reported in the existing specifications for EAC Partner States. It can be noted that there is a range of similarities and divergences in terms of the type of bituminous base course and asphalt concrete surfacings. The next sections discuss theses similarities and differences with a view to come up with harmonisation regimes.

Table 19: Types of Bituminous Base Course and Asphalt Concrete Surfacing

Types	Country		
	Kenya	Tanzania (Mainland and Zanzibar)	Uganda
Asphalt concrete surfacing	Type I Type II	AC20 AC14 AC10	AC20 AC14 AC10 19.0, 12.5, 9.5 Nominal mixes
Hot bituminous	DBM40 DBM30	DBM40 DBM30 LAMBS	DBM40 DBM30 37.5, 25.0 Nominal mixes
Cold mixed bituminous		FBMIX BEMIX	FBMIX BEMIX
Penetration Macadam		PM80 PM60 PM30	

a) Bituminous *binders*

Table 20 presents various bituminous binders for bituminous base course and asphalt concrete surfacing along with the compliance standards as reported in the existing specifications for the EAC Partner States. It can be seen that bituminous binders have to comply with AASHTO standards in Tanzania while Uganda follows BS standards as well as American standard (SP-1) for performance graded asphalt binder specification and testing.

Table 20: Specifications for bituminous binders

Country	Type of binder	Specifications
Kenya	-	-
Tanzania (Mainland and Zanzibar)	Penetration grade bitumen Cutback bitumen Bitumen emulsions, anionic Bitumen emulsions, cationic	AASHTO designation M20-70 AASHTO designation M82-75 AASHTO designation M140-88 AASHTO designation M208-96
Uganda	Penetration grade bitumen Performance Grade asphalt Cutback bitumen Bitumen emulsions, anionic Bitumen emulsions, cationic	BS EN 12591:2000 SP-1[a] BS EN 12591:2000 BS 434:1984 BS 434:1984

[a] Performance graded asphalt binder specification and testing, Super-pave Series No. 1 –Asphalt Institute

b) Foamed bitumen

The manuals for Tanzania and Uganda specify additional requirements for foamed bitumen in addition to the general requirements presented in Table 20. They state that foamed bitumen shall meet the following requirements when tested using a 10 litre cylindrical bucket:

- The ratio between the volume of the bitumen in a foamed state and in an un-foamed state should be minimum 15.
- At least one of the following requirements should be met:
  - the time from the foam is ejected into atmospheric pressure until the volume has decreased to half its maximum volume should be minimum 15 seconds.
  - the ratio between volume of bitumen in a foamed state and in an un-foamed state should be minimum 7.5 after 15 seconds have elapsed since the foam was ejected into atmospheric pressure.

Recommendation

EAC Partner States adopt specifications for bituminous binders shown in Table 21. In additional to these requirements, foam bitumen should meet the following requirements when tested using a 10 litre cylindrical bucket:

- The ratio between the volume of the bitumen in a foamed state and in an un-foamed state should be minimum 15.
- At least one of the following requirements should be met:
  - the time from the foam is ejected into atmospheric pressure until the volume has decreased to half its maximum volume should be minimum 15 seconds.

	– the ratio between volume of bitumen in a foamed state and in an un-foamed state should be minimum 7.5 after 15 seconds have elapsed since the foam was ejected into atmospheric pressure.

Table 21 Recommended compliance standards for bituminous binders

Type of binder	Specifications
Penetration grade bitumen	BS EN 12591:2000
Performance Grade asphalt	SP-1[a]
Cutback bitumen	BS EN 12591:2000
Bitumen emulsions, anionic	BS 434:1984
Bitumen emulsions, cationic	BS 434:1984

c) Aggregates

The aggregates employed for all bituminous base course and asphalt concrete surfacing layers when tested are required to conform to the requirements shown in Table 22. The table, however, shows some requirements for some specific asphaltic pavement layers as well. It can be noted from the table that specifications for Uganda manual supersedes specifications for Tanzania in terms of the tests for particle shape, aggregate strength and absorption as well as inclusion of new test of cleanliness, soundness and coating and stripping. It should also be noted that specifications for Kenya require course aggregate retained on a 6.3 mm sieve to consist of crushed stone free from clay, silt, organic matter and other deleterious substances and the aggregate should comply with the requirements shown in Table 23 for asphalt concrete and dense bitumen macadam.

Table 22: Specifications for aggregate for asphaltic pavement layers

Property	Tanzania		Uganda	
	Requirement	Test Method	Requirement[1]	Test Method
Cleanliness			A minimum value of 45 of sand equivalent of the fine aggregate Plasticity index be less than 4%	AASHTO T176 BS 1377-2
Particle shape	The flakiness index of the course aggregates be less than 35% for penetration macadam base PM 60 & PM 30 courses	CML Method 2.4	The flakiness index of the course aggregates be less than 25%	BS 812-105: 1989
Aggregate strength	TFV be less than 110 kN[2] (or 80 kN[3]) and wet/dry ratio not be less than 75%	CML Method 2.7	TFV should be not less than 110 kN and wet/dry ratio not less than 75%	BS 812-111:1990
Absorption	Water absorption of course aggregates not exceed 2%[4] (or 3%[5]) by mass	CML Method 3.9	Water absorption of course aggregates not exceed 2% by mass	BS 812-2:1995
Soundness			The sodium soundness be less than 12%	AASHTO T104
Coating and stripping test			The coating and stripping of the coarse aggregates be greater than 95%	AASHTO T182
CBR soak 4 days[3]	CBR value be not less than 30% before admixture of bitumen	CML Method 1.11		
Plasticity index	Plasticity index before admixture of bitumen shall not be more than: − FBMIX material: 14% − BEMIX material: 8%			

[1]Aggregates employed for all asphaltic pavement layers are required to conform to the listed requirements
[2]Applies to asphalt concrete, hot mixed bitumen base course, and penetration macadam base
[3]Applies to cold mix bitumen base course
[4]Applies to asphalt concrete only
[5]Applies to hot mixed bitumen base course only

Table 23: Requirements for course aggregate retained on a 6.3 mm sieve for asphalt concrete and dense bitumen macadam

Property		Dense bitumen macadam	Asphalt concrete		
			Aggregate Class		
			a	b	c
LAA	Max	35	30	35	40
ACV	Max	28	25	28	30
SSS	Max	12	12	12	12
FI	Max	25	20	20	25

For comparison purpose, the LAA, ACV, SSS and FI tests for aggregate are entirely adopted in the specifications for Tanzania and Uganda as shown in Table 22. It should be noted that the purposes of LAA and ACV tests have been replaced by the TFV test and its requirements.

The other very important requirement is grading of aggregates and this will be treated separately in the subsequent review and discussion of each type of bituminous base course and asphalt concrete surfacing layers.

Recommendation

EAC Partner States adopt specifications for bituminous binders shown in Table 24.

Table 24 Recommended specifications for bituminous base courses and asphalt concrete surfacing layers

Property	Requirement	Test Method
Cleanliness	A minimum value of 45 of sand equivalent of the fine aggregate Plasticity index be less than 4%	AASHTO T176 BS 1377-2
Particle shape	The flakiness index of the course aggregates be less than 25%	BS 812-105: 1989
Aggregate strength	TFV should be not less than 110 kN and wet/dry ratio not less than 75%	BS 812-111:1990
Absorption	Water absorption of course aggregates not exceed 2% by mass	BS 812-2:1995
Soundness	The sodium soundness be less than 12%	AASHTO T104
Coating and stripping test	The coating and stripping of the coarse aggregates be greater than 95%	AASHTO T182

d) Mix design requirements for Asphalt Concrete Surfacing Courses

Generally, the properties and performance of asphalt concrete mixtures are influenced by the choice and properties of materials to be used in the mix and the employed mix design method as well. Table 25 shows the general requirements for bitumen, aggregate and bitumen content (for bidding purpose) for asphalt concrete. It should also be noted that specifications for Uganda include Performance Grade (PG) Asphalt for asphalt concrete surfacing layers as may be specified in the Special Specifications. In principle, PG stipulates a temperature window within which the bitumen meets certain criteria. The PG grading required in any one region will depend on the climate and the seasonal temperature variations, as well as the intensity of the traffic.

Table 25: Material requirements for asphalt concrete

Country	Asphalt Concrete	Type of bitumen	Aggregate (%)	Bitumen content (%) for bidding purposes
Kenya	Type I & II	80/100 and 180/200 Pen Grade for altitudes in excess of 2,500 m		
Tanzania (Mainland and Zanzibar)	AC20 AC14 AC10	60/70 or 40/50 Pen Grade	95.0 94.5 94.0	5.0 5.5 6.0
Uganda	AC20 AC14 AC10	60/70 or 40/50 Pen Grade	95.0 94.5 94.0	5.0 5.5 6.0

The physical characteristics of the four types of bitumen that have found wide application in the EAC region can be summarised as shown in Table 26 whereby 180/200 Pen Grade is shown to be the most softer and 40/50 the hardest bitumen. The bitumen used for production of asphalt concrete, where high road surface temperatures prevail as it is in the EAC region, is usually pen 40–50 or pen 60–70.

Table 26: Penetration grade characteristic

Grade	Physical characteristic
180/200	Soft
80/100	↓
60/70	↓
40/50	Hard

Further, our review showed that existing specifications stipulate same grading requirements for asphalt concrete employing Marshall Mix design as shown in Table 27. For the case of Kenya, asphalt concrete Type I is of high stability consisting of course aggregate which are entirely crushed while the crushing ratio of aggregate for Type II (flexible) is not less than 60%. Some differences occur between sieve sizes adopted by the specifications. Kenya follows ASTM sieve sizes standards while Tanzania and Uganda follow BS sieve size standards. Tanzania and Uganda have already adopted common asphalt concrete mixes (AC20, AC14, & AC10) as depicted in Tables 25 and 27.

In line with a provision for inclusion of PG asphalt, specifications for Uganda provide grading limits for combined aggregate for asphalt concrete surfacing layers employing Super-pave mix design method. The grading requirements are proposed for adoption by other EAC Partner States, as indicated in our specific recommendation.

Table 27: Grading requirements for Marshall mix design

Sieve size (mm)	Kenya								Tanzania (Mainland & Zanzibar) and Uganda		
	Type I						Type II		Asphalt concrete continuously graded		
	Wearing course			Binder course			Wearing course		AC20	AC14	AC10
	0/14	0/10	0/6	0/20	0/14	0/10	0/14	0/10			
28	-	-	-	100	-	-	-	-	100	-	-
20	100	-	-	90-100	100	-	100	-	80-100	100	-
14	90-100	100	-	75-95	90-100	100	90-100	100	60-80	85-100	100
10	70-90	90-100	100	60-82	70-90	90-100	70-95	90-100	50-70	72-94	85-100
6.3	55-75	60-82	90-100	47-68	52-75	60-82	55-85	62-90			
5.0									36-56	52-72	55-72
4.0	45-63	47-67	75-95	37-57	40-60	45-65	46-75	50-80			
2.36									28-44	37-55	38-57
2.0	33-48	33-50	50-70	25-43	30-45	30-47	35-60	35-65			
1.18									20-34	26-41	27-42
1.0	23-38	23-38	33-50	18-32	20-35	20-35	25-45	25-50			
0.600									15-27	16-28	18-32
0.425	14-25	14-25	20-33	11-22	12-24	12-24	14-32	14-33			
0.300	12-22	12-22	16-28	9-17	10-20	10-20	11-27	11-27	10-20	12-20	13-23
0.150	8-16	8-16	10-20	5-12	6-14	6-14	6-17	6-17	5-13	8-15	9-16
0.075	5-10	5-10	6-12	3-7	4-8	4-8	3-9	3-8	2-6	4-10	4-10
Aggregate									95%	94.5%	94%
Bitumen content ASTM 2172-88									5%	5.5%	6%

The design requirements for asphalt concrete surfacing employing Marshall Mix design method are given in Table 28.

Table 28: Design requirements for asphalt concrete surfacing employing Marshall Mix design

Property of mixture and laboratory test method	**Kenya**			**Tanzania**	**Uganda**
	Type I (wearing course)	**Type II (binder course)**	**Type III (wearing course)**	**AC20, AC14 & AC10**	**AC20, AC14 & AC10**
Marshall flow (mm)	2 - 4	2 - 4	3 - 5	2 - 4	2 - 4
Marshall stability at refusal compaction (N), all severely loaded areas	Min. 9000[1]	Min. 7000[1]	4000 - 9000[1]	Min. 9000	Min. 9000
Marshall stability (2×75 blows) (N) – Traffic loads – $\succ 10 \times 10^6$ esal				8000 - 18000	8000 - 18000
Marshall stability (2×75 blows) (N) – Traffic loads – 1 - 10 $\times 10^6$ esal				7000 - 15000	7000 - 15000
Marshall stability (2×75 blows) (N) – Traffic loads – $\prec 1 \times 10^6$ esal				4000 - 10000	6000 - 10000
Air voids (%)	3 - 5	3 - 7	3 - 5	3 - 6	3 - 5
Voids in mineral aggregate (%) – AC20				Min. 14	Min. 14
Voids in mineral aggregate (%) – AC14				Min. 15	Min. 15
Voids in mineral aggregate (%) – AC10				Min. 16	Min. 16
Voids filled with bitumen (%) – Traffic loads – $\succ 10 \times 10^6$ esal					65 – 75
Voids filled with bitumen (%) – Traffic loads – 1 - 10 $\times 10^6$ esal					65 – 78
Voids filled with bitumen (%) – Traffic loads – $\prec 1 \times 10^6$ esal					70 - 80
Requirement after refusal laboratory compaction to BS 594 (severely loaded areas only)				Min. air voids 3%	Min. air voids 3%
Indirect tensile strength (kPa) AASHTO T 283				Min. 800 tested at 25^{0}C	Min. 800 tested at 25^{0}C
Indirect wet tensile strength (kPa) AASHTO T 283					80% of dry strength
Immersion index (%)				3 - 6	

[1] 50 blows and does not indicate whether is for severely loaded areas

It can be seen from Table 28 that Tanzania and Uganda have adopted almost similar specifications for asphalt concrete surfacing Marshall Mix design with the exception that the immersion index test is replaced by indirect wet tensile strength test and new requirements for voids filled with bitumen are introduced in the specifications for Uganda.

Furthermore, the specifications for Uganda provide requirements for gyratory compaction efforts as well as asphalt concrete mix design when the Super-pave Mix Design Method is specified in the Special Specifications. The requirements for gyratory compaction and asphalt concrete surfacing employing Super-pave Mix Design are also included hereunder as specific recommendation for EAC Partner States.

Recommendation

In view of the foregoing review, comparison and discussion about asphalt concrete surfacings mix design, EAC Partner States should:

1. Develop Performance-based asphalt grades
2. Adopt requirements shown in Table A1 in the Appendix for aggregate grading for Marshall Mix Design
3. Adopt design requirements shown in Table A2 in the Appendix for Marshall Mix design.
4. Adopt grading limits for combined aggregates shown in Table A3 in the Appendix for asphalt concrete surfacing design employing Super-pave Mix Design.
5. Adopt requirements shown in Table A4 in the Appendix for Super-pave Mix Design Gyratory compaction
6. Adopt design requirements shown in Table A5 in the Appendix for Super-pave Mix Design.

e) Requirements for hot mixed bituminous base and binder courses

Hot mixed bituminous courses in the existing specifications for EAC Partner States are: DBM40 and DBM30 are common in Kenya, Tanzania and Uganda while LAMBS are specified for Tanzania and DBM courses (with 37.5 mm nominal size of aggregates) and binder course (with 25 mm nominal aggregate sizes) employing Super-pave Mix Design Method are specified for Uganda.

Table 29 summarises general materials requirements for hot mix bituminous courses. Unlike the case of asphalt concrete surfacings, the bitumen recommended for DBM courses are Pen Grades 40/50 and 60/70. It can also be noted from Table 29 that Tanzania and Uganda recommend thicker layers of DBM40 and DBM30 compared to DBM layer thicknesses used in Kenya.

DBM courses can be designed using Marshall Mix Design and Super-pave Mix Design Methods. A summarised comparison of aggregate gradation requirements for DBM courses employing Marshall Mix Design is given Table 30. Some differences occur between sieve sizes adopted by the specifications. Tanzania and Uganda have already adopted common asphalt base course i.e., DBM40 and DBM30 in Table 30.

As shown in Table 31, it is only specifications for Kenya and Uganda which provides for the design requirements for hot mixed asphaltic base course employing Marshall Mix design. However, specifications for Uganda are superior since they specify Marshall Stability and compaction requirements at various levels of design traffic.

Moreover, it is only the specifications for Uganda which cover asphaltic base course design using Super-pave Mix Design and this study also gives recommendation on the design requirements and grading requirements for asphaltic base courses employing Super-pave Mix Design.

Table 29: Materials requirements for hot mixed bituminous base course

Country	Hot mixed bituminous base course	Requirements			Test Method
		Type of bitumen	Bitumen content for bidding purpose (%)	Maximum compacted layer thickness (mm)	
Burundi					
Kenya	DBM40 DBM30	60/70 or 80/100 60/70 or 80/100	-	75 – 125 30 - 100	- -
Rwanda					
Tanzania (Mainland and Zanzibar)	DBM40 DBM30	60/70 or 40/50 60/70 or 40/50	4.0 4.5	80 – 200 60 - 150	CML1.7, 2.7, 3.22, 3.5, 3.9/ BS812:Part 111, BS1377:Part 2, ASTM D2172-88, method B, ASTM D5-86, ASTM C127-88
Uganda	DBM40 DBM30	60/70 or 40/50 60/70 or 40/50	4.0 4.5	80 – 200 60 - 150	BS812-111, BS812-2

Table 30: Grading requirements for Dense Bitumen Macadam employing Marshall Mix Design

Sieve size (mm)	Kenya		Tanzania (Mainland & Zanzibar) and Uganda	
	0/40 (% passing)	0/30 (% passing)	DBM40 (% passing)	DBM30 (% passing)
50	100	-	100	-
37.5	95-100	100	95-100	100
28	70-94	90-100	70-95	90-100
20	-	71-95	-	70-95
14	56-76	58-82	56-76	58-82
10			53-70	52-73
6.3	44-60	44-60		
5.0			39-56	40-56
2.0	24-40	26-40	24-40	24-40
1.18			19-35	19-35
1	20-33	20-33		
0.425			9-25	9-25
0.300	7-21	7-21	7-21	7-21
0.150	4-15	-		
0.075	2-8	2-8	2-9	2-9

Table 31: Design requirements for DBM course employing Marshall Mix design

Property of mixture and laboratory test method			Kenya		Tanzania	Uganda
			0/40	0/30		DBM 40 & DBM 30
Marshall flow (mm)			2-5	2-4		2-4
Marshall stability at refusal compaction (N), all severely loaded areas			Min 5000	Min 5000		Min 9000
Marshall stability (2×75 blows) (N)	Traffic loads	$>10 \times 10^6$ esal				Min 7000
		1.5-10 $\times 10^6$ esal				Min 6000
		$<1 \times 10^6$ esal				Min 3500
Air voids (%)			4-10	4-10		4-8
Compaction levels	Traffic loads	Severely loaded areas*				To refusal**
		$>10 \times 10^6$ esal				To refusal**
		1.5-10 $\times 10^6$ esal				2×75
		$<1 \times 10^6$ esal				2×50
Loss of stability after soaking (ASTM 1075) as % of unsoaked value			Max 35%	Max 35%		
*Severely loaded areas should be identified in the Special Specifications						
** Requirement after refusal laboratory compaction to BS 594. Minimum air voids 3%.						

[1]50 blows and does not indicate whether is for severely loaded areas

Recommendation

In view of the foregoing review, comparison and discussion about hot mixed asphalt base course design, we recommend that EAC Partner States should:

1. Adopt requirements shown in Table A6 in the Appendix for aggregate grading for Marshall Mix Design
2. Adopt design requirements shown in Table A7 in the Appendix for Marshall Mix design.
3. Adopt grading limits shown in Table A8 in the Appendix for hot mixed asphaltic base and binder courses employing Super-pave Mix Design.
4. Adopt design requirements shown in Table A5 in the Appendix for Super-pave Mix Design.

f) Requirements for penetration macadam base course

Penetration macadam base course is only specified in the specifications for Tanzania. Other EAC Partner States have not included this type of base course in their specifications. Table 32 presents a summary of the materials specifications as stipulated in the specifications for Tanzania. Likewise, the specifications provide grading limits for penetration macadam base course and this study recommends them for use in EAC Partner States as indicated in our specific recommendation hereunder.

Table 32: Requirements for Penetration macadam base course

Country	Penetration macadam type	Requirements					Test Methods
		Type of bitumen (Penetration grade)	Bitumen spray rate for bidding purposes (l/m^2)	TFV (kN)	FI	Thickness (mm)	
Tanzania (Mainland and Zanzibar)	PM80 PM60 PM30	80/100 or 60/70 80/100 or 60/70 80/100 or 60/70	3.5 3.5 2.5	110 110 110	≤35 ≤35 ≤35	125 100 50	BS 1377:Part2, BS812:Section 105.1, BS812:Part 111 & ASTM D5-86

Recommendation

The study recommends specifications shown in Table A9 in the Appendix for penetration macadam base for EAC Partner States.

g) Requirements for cold mixed bituminous base course

Unlike the penetration macadam base course, cold mixed bituminous base course has been specified in the specifications for Kenya, Tanzania and Uganda. Specifications for cold mixed bituminous base course for Tanzania and Uganda are similar.

Specifications for Kenya recommend the grading of mixture of aggregates for cold mixes be within and approximately parallel to the grading envelope specified in the special specification as well as stability has to be stated in the special specification. It is also recommended that the bituminous binder for cold mixes that has to be used immediately be MC 250, MC 800 or MC 3000 or an anionic emulsion A2 or A3 or a cationic emulsion K2 or K3, as specified in the special specification, and cold mixes to be stockpiled the recommended binders are MC 250 or MC 800 or SC 250 or SC 800 and the aggregates are recommended to be specified in the special specification.

On the other hand, Tanzania and Uganda provide similar specifications for FBMIX and BEMIX cold mixed bituminous base courses. The design requirements addressed by the specifications include grading limits, bitumen type (foamed and emulsion), nominal consumption of bitumen (l/m^3) for bidding purposes, Marshall stability tested at 40^0C, minimum E-modulus tested at 290C by indirect tensile strength method or alternative approved method, minimum and maximum moisture contents at the time of laying, and minimum compacted field density as percentage of Marshall dry density.

Recommendation

We recommend specifications shown in Table A10 in the Appendix for FBMIX and BEMIX cold mixed bituminous bases for EAC Partner States.

4.3.5.3 General Bitumen Binders Requirements for Seals

The specifications for Tanzania and Uganda give similar requirements for binder and aggregate materials common to the construction of seals as follows.

a) Conventional Bituminous binders

The manuals require bituminous binder to comply with the specifications given in Table 33 or suitable equivalent.

Table 33: Specifications for conventional bituminous binders for seals

Country	Type of binder	Specifications
Kenya	-	-
Tanzania (Mainland and Zanzibar)	Penetration grade bitumen Cutback bitumen Bitumen emulsions, anionic Bitumen emulsions, cationic Invert bitumen emulsion	AASHTO designation M20-70 AASHTO designation M82-75 AASHTO designation M140-88 AASHTO designation M208-96 SABS 1260
Uganda	Penetration grade bitumen Performance Grade asphalt Cutback bitumen Bitumen emulsions, anionic Bitumen emulsions, cationic	BS EN 12591:2000 SP-1[a] BS EN 12591:2000 BS 434:1984 BS 434:1984

b) Modified binders

Over the past years, there has been a continuing, underlying, demand for high-quality bituminous binders. As a consequence, many modifying additives have been investigated with the aim of improving the properties of the penetration grade bitumen used in road pavements. Some of these modifiers are proprietary products; others are in the public domain. Some have been commercially successful; others have not. Brennan and O'Flaherty (2002) observed that the main bitumen modifiers in current use can be divided into the following groups:

- Adhesive agents
- Thermoplastic crystalline polymers. They are also known as thermoplastic plastomers, these polymers include ethylene vinyl acetate (EVA), polyethylene (PE), polypropylene (PP), polystyrene, and polyvinylchloride (PVC). Of these EVA polymer, which results from the copolymerization of ethylene and vinyl acetate, is probably the thermoplastic copolymer that is most widely used to modify bitumen for road works.
- Thermoplastic rubbers. Also know as thermoplastic elastomers include polymers such as natural rubber, vulcanized rubber, styrene-butadiene-styrene block copolymer (SBS), styrene-ethylene-butadiene-styrene block copolymer (SEBS), styrene-isoprene-styrene block copolymer (SIS), and polybutadiene.
- Thermosetting polymers include polymers such as acrylic, epoxy, or polyurethane resins, which result from the bending of a liquid resin and a liquid hardener that react chemically with each other.
- Chemical modifiers which include sulphur and manganese.

Bitumen-Rubber

Manuals for Tanzania and Uganda require bitumen-rubber binder to comply with the following specifications:

- The recommended base bitumen is 60/70, 80/100 or 150/200 penetration grade bitumen or a blend of any two or all three grades to provide a product with the required viscosity.
- The rubber should be obtained by processing and recycling pneumatic tyres. It should be pulverised, free from fabric, steel cord and other contaminants. A maximum of 4% by mass of fine particle size calcium carbonate, or talc, may be added to the rubber crumbs to prevent the rubber particles from sticking together. At the time of use the crumbs should be free flowing and dry and comply with the requirements given in Table 34.

Table 34: Requirements for Rubber Crumbs

Sieve analysis		**Test Method**
Sieve size (mm)	Percentage passing by mass	
1.18 0.60 0.075	100 40-70 0-5	BR6T (Sabita)*)
Other requirements		
Natural rubber hydro-carbon content	30% (minimum)	BS 903 Parts B11 and B12
Fibre length	6 mm (maximum)	-
Relative density (kg/m^3)	1100-1250	BR9T (Sabita)

*Southern African Bitumen and Tar Association

Extender oils should be petroleum derived material of high aromaticity and are required to comply with specifications shown in Table 35.

Table 35: Requirements for Extender Oils

Property	Requirements
Flash point % by mass of saturated hydrocarbons % by mass of aromatic, unsaturated hydrocarbons	180^0C 25% (max) 50% (min)

The specifications also specify that diluents should be a distillate of hydrocarbon and bitumen-rubber blend, including extender oil and /or diluents should comply with the requirements given in Table 36, whereas the bitumen-rubber binder should comply with the requirements given in Table 37.

Table 36: Requirements for Bitumen-Rubber Blend

Property	Requirement
% rubber by mass of total blend	20%-24%
% extender oil by mass of total blend	6% (max)
% of dilutent by mass of total blend	7% (max)
Blending/Reaction temperature	1 70^0C – 210^0C
Reaction time	0.5 - 2 hours

Table 37: Requirements for Bitumen-Rubber Binder

Property	Requirements	Test Method (or equivalent method on approval of the Engineer)
Minimum compression recovery: • after 5 minutes • after 1 hour • after 4 hours	 70% 70% 48-55%	BR3T (Sabita)
Ring-and-ball softening point	minimum 55^0C	BS EN 12591:2000
Resilience	13%-35%	BR2T (Sabita)
Dynamic viscosity (Haake at 190^0C)	20 35 cPas	BR5T (Sabita)
Flow (mm)	20 – 75	BR4T (Sabita)

Homogeneous Cold Applied Modified Binders

If any polymer other than the elastomer polymers styrene-butadiene rubber (SBR) or styren-butadiene-styrene (SBS) is required for the manufacture of cationic modified bitumen emulsions, the manuals for Tanzania and Uganda require it to comply with the requirements given in the Special Specifications. The manuals require the Special Specifications to include details on the following:

- Type of elastomer polymer; SBR or SBS. Unless otherwise specified SBR recommended for bidding purposes.
- Grade base bitumen; 80/100 penetration grade or 150/200 penetration grade. Unless otherwise specified 80/100 penetration grade road grade is recommended for bidding purposes.
- Modified binder content; 65% or 70%. Unless otherwise specified 65% is recommended for bidding purposes.

The aforementioned components together with polymer content will dictate the attributes attainable. Unless otherwise specified, the properties of cationic modified bitumen emulsion containing SBR or SBS solids should comply with the requirements in Table 38.

Table 38: Requirements for Cationic Modified Bitumen Emulsion

Polymer modifier	Required properties, grade of base bitumen	Minimum modified binder content (%)	Minimum viscosity at 50^0C Saybolt Furol (sec.)	Maximum residue on sieving (g/100ml)	Particle charge	Sedimentation after 60 rotations
SBR	80/100 150/200	70 65	80 70	0.25 0.25	Positive Positive	Nil Nil
SBS	80/100 150/200	70 65	80 50	0.25 0.25	Positive Positive	Nil Nil
Test Method	-	ASTM D244	ASTM D244	SABS 548	SABS 548	SABS 548

A volatile solvent flux content of up to 3% mass by mass of the bitumen may be added to enhance emulsion performance with regard to prevailing climatic conditions. Any expected change to specified values should first be discussed with the Engineer prior to the addition of any such enhancer. The properties of the recovered modified bitumen using a rotary vacuum evaporation method or simple evaporation method should be required to comply with the requirements in Table 39.

Table 39: Requirements for Recovered Modified Bitumen

Polymer modifier	Required properties				Elastic recovery (%)		
	Grade of base bitumen	Minimum Softening point (^{0}C)	Minimum dynamic viscosity at 135^0C (Pa.s)	Minimum ductility at 10^0C (mm)		at 5^0C (%)	at 50^0C (%)
SBR						90 90	100 100
SBS	80/100 150/200	60 47	1.2 1.0	500 500	55 60	90 90	100 100
Test Method	-	ASTM D36	ASTM D4402	DIN 52013	DIN 52013	DIN 52013	TMH methodB11
Note: Modified binder is bitumen plus polymer.							

If there is any discrepancy in the test results on recovered modified binder, then the results on recovered binder obtained from the rotary vacuum evaporation method should be adopted.

Homogeneous Hot-applied Modified Binders

As required by the Tanzania and Uganda specifications, the requirements for any polymer other than the generic types listed in Table 40 used for the manufacture of homogeneous hot-applied modified binders should be indicated in the Special Specifications.

Table 40: Requirements for Hot-Applied Modified Binders

Genetic type of modified binder		Required properties							
		Grade of Base bitumen	Minimum Softening point (^0C)	Minimum dynamic viscosity at 135^{0}C (Pa.s)	Minimum ductility at 10 ^{0}C (mm)	Minimum elastic recovery 10^{0}C (%)	Maximum stability difference (^0C)	Minimum adhesion	
								at 5^{0}C (%)	at 50^{0}C (%)
Plastomerpolymer (EVA)		150/200	48	0.5	300	45	2	90	
Elasto-merpoly-mer	SBR	80/100	47	1.0	1000	55	2	90	100
	SBS	80/100	49	1.0	500	60	2	90	
	SBR	150/200	45	0.5	1000	55	2	90	
	SBS	150/200	47	0.5	500	60	2	90	100
Test Method	-	-	ASTM D36	ASTM D4402	DIN 52013	DIN 52013	DIN 52013	TMH 1 Method B11	

Where applicable the following details should be indicated in the Special Specifications:

- Generic type (plastomer or elastomer) and type polymer.
- Grade base bitumen (80/100 or 150/200) required.

The aforementioned components together with polymer content dictate the attributes attainable.

Recommendation

We recommend to the EAC partner states the requirements for conventional and modified bituminous binder given in Tables 33 to 40 for bituminous seals.

4.3.5.4 Surface Dressings

c) Bituminous binders

The available specifications for Kenya, Tanzania and Uganda require the bituminous binder surface dressing to consist of one of the following binders shown in Table 41, whichever is specified in the Special Specifications or the Bill of Quantities or ordered by the designer. It can be seen that the most commonly used binder for surface dressing is emulsion, cut-back bitumen, and a blend of penetration grade and cut-back bitumen.

Table 41: Recommended Binders for Surface Dressing

Kenya	Tanzania (Mainland and Zanzibar)	Uganda
Cationic emulsion such as K1-60 or K1-70	80/100 penetration-grade bitumen	80/1 00 penetration-grade bitumen
MC 3000 cut-back bitumen	150/200 penetration-grade bitumen	150/200 penetration-grade bitumen
A penetration/cut-back mixture blended to the approval of the Engineer	60%, 65% or 70% spray-grade emulsion	60%, 65% or 70% spray-grade emulsion
	MC-3000 cut-back bitumen	MC-3000 cut-back bitumen
	Modified binder	Modified binder

Recommendation

In view of the technical advantages associated with the use of emulsions, we recommend the use of emulsions in surface dressing operations in EAC region.

d) Requirements for single surface dressings

Review of the three specifications revealed that aggregates for single surface dressing should consist of clean, tough, durable fragments of crushed stone free from any deleterious matter and comply with the requirements for grading, particle shape, aggregate strength, and average least dimension (ALD). Table 42 presents a summary of requirements for aggregate for comparison purpose.

It can be seen from Table 42 that specifications for Kenya lack requirements for aggregates particle strength and average least dimension. Likewise, specifications for Tanzania do not give ALD values, although the pavement and design manual for Tanzania include methods for establishing ALD values and the way they can be used to design surface dressings. Overall, specifications for Tanzania and Uganda provide similar requirements including grading limits for single surface dressings. It should be noted that Tanzania and Uganda recommend similar grading limits for single surface dressings and double surface dressings.

Table 42: Aggregate requirements for single surface dressing

Country	Nominal Sizes	Requirements						Test Method
		Max flakiness index (FI)	TFV (kN)	Ratio TFV soaked to dry (%)	ALD	Binder hot spray rate (l/m²)	Agg. Spread rate	
Kenya	14/20	20				Specified in the special specs	Specified in the special specs	
	10/14	20						
	6/10	25						
	3/6	25						
Tanzania (Mainland and Zanzibar)	20	20	For AADT>1000:160 For AADT<1000:120	75		1.80	0.015 m^3/m^2	CML1.7, 2.4, 2.7/BS1377-2, BS812:105.1 & BS812:111
	14	25				1.50	0.012 m^3/m^2	
	10	25				1.10	0.010 m^3/m^2	
	7	30			-	0.80	0.80 m^3/m^2	
Uganda	20	20	For AADT>1000:160 For AADT<1000:120	75	11 - 15	1.80	19 kg/m^2	BS812:105.1 & BS812:111
	14	25			8 - 10	1.50	14 kg/m^2	
	10	25			5 - 6.5	1.10	11 kg/m^2	
	7	30			2 - 4	0.80	7 kg/m^2	

Recommendation

We recommend to the EAC Partner States specifications shown in Table A11 in the Appendix for Single Surface Dressing.

e) Requirements for double surface dressings

As for single surface dressing, it was found that aggregates for double surface dressings should consist of clean, tough, durable fragments of crushed stone free from any deleterious matter and comply with the requirements for grading, particle shape, aggregate strength, and average least dimension (ALD). Table 43 presents a summary of requirements for aggregate for comparison purpose.

Likewise, specifications for Kenya lack requirements for aggregates particle strength and average least dimension while specifications for Tanzania do not give ALD values, although the pavement and design manual for Tanzania include methods for establishing ALD values and the way they can be used to design surface dressings. Overall, specifications for Tanzania and Uganda provide similar requirements including grading limits for double surface dressing. It should be noted that Tanzania and Uganda recommend similar grading limits for single surface dressing and double surface dressing.

Recommendation

We recommend specifications shown in Table A12 in the Appendix for Double Surface Dressings for EAC Partner States.

Table 43: Aggregate requirements for double surface dressing

Country	Nominal Size of aggregate (mm)	Requirements						Test Method
		Max flakiness index (FI)	TFV (kN)	Ratio TFV soaked to dry (%)	ALD	Nominal rates of application		
						Binder spray rate (l/m^2)	Aggregate Spread rate	
Kenya	14/20	20				Specified in the special specs	Specified in the special specs	
	10/14	20						
	6/10	25						
	3/6	25						
Tanzania (Mainland and Zanzibar)	20	20	For AADT>1000:160 For AADT<1000:120	75		1.8 for AADT<200	0.015 m^3/m^2	CML1.7, 2.4, 2.7/BS1377-2, BS812:105.1 & BS812:111
						1.5 for AADT 200-1000		
						1.2 for AADT>1000		
	14	25				1.4 for AADT<200	0.011 m^3/m^2	
						1.1 for AADT 200-1000		
						1.0 for AADT>1000		
	10	25				1.2 for AADT<200	0.009 m^3/m^2	
						1.0 for AADT 200-1000		
						0.9 for AADT>1000		
	7	30				0.9 for AADT<200	0.007 m^3/m^2	
						0.8 for AADT200-1000		
						0.7 for AADT>1000		
Uganda	20	20	For AADT>1000:160 For AADT<1000:120	75	11-15	1.8 for AADT<200	19 kg/m^2	BS812:105.1 & BS812:111
						1.5 for AADT 200-1000		
						1.2 for AADT>1000		
	14	25			8-10	1.4for AADT<200	14 kg/m^2	
						1.1for AADT 200-1000		
						1.0 for AADT>1000		
	10	25			5-6.5	1.2 for AADT<200	11 kg/m^2	
						1.0 for AADT 200-1000		
						0.9 for AADT>1000		
	7	30			2-4	0.9 for AADT<200	7 kg/m^2	
						0.8 for AADT200-1000		
						0.7 for AADT>1000		

4.3.5.5 Otta Seals

Otta seals specifications are only reported and recommended in the manual for Tanzania. The requirements are as follows.

Materials

Binders for Otta Seals are as shown in Table 44.

Table 44: Binders for Otta Seals

Weather	Binder
Warm	150/200 penetration grade bitumen or MC 3000
Cold weather with likelihood of night temperature falling below 10^0C	MC 800 or 150/200 penetration grade

Kerosene is recommended to be used as the cutter where correction of viscosity is required or cutback bitumen is made on site from penetration grade bitumen.

On the other hand, Tables 45 and 46 present the general requirements for aggregate strength and aggregate strength when AADT is less than 100 for Otta seals, and flakiness index of not to exceed 30.

Table 45: Aggregate strength for Otta Seals, general requirements

Ten Percent Fines Value, TFV tested dry	Ten Percent Fines Value, TFV tested after 24 hours soaking (in % of TFV tested dry)
Minimum 110 kN	75%

Table 46: Aggregate strength for Otta Seals, AADT less than 100

Ten Percent Fines Value, TFV tested dry	Ten Percent Fines Value, TFV tested after 24 hours soaking (in % of TFV tested dry)
Minimum 90 kN	60%

Further, the materials recommended for sand for cover seals constitute of crusher dust, river sand or other natural sand and should be non-plastic, free from organic matter and lumps of clay. All the material should pass the 6.3 mm sieve, unless otherwise approved by the Engineer.

Rates of application of material

<u>Binder for Otta Seal</u>

All spray rates recommended by the manual refers to hot spray rates of binder including any cutters and it is recommended to fall within the range of 1.5 and 2.0 l/m^2 per layer per layer of Otta seal. For bidding purposes 1.7 l/m^2 is recommended, however, it is suggested that the actual spray rate should be established on site for the approval of the Engineer.

Aggregate for Otta Seals

The aggregate application rates for Otta Seals are recommended to be in the range 0.013 to 0.020 m^3/m^2.per layer. For bidding purposes a spread rate of 0.015 m^3/m^2 per layer is recommended, however, the actual spray rate should be established on site for the approval of the Engineer.

Binder for sand covers seal

All spray rates recommended by the manual refers to hot spray rates of binder including any cutters and it is required to fall within the range of 0.6 and 0.9 l/m^2 for sand cover seals. For bidding purposes 0.8 l/m^2 is recommended, however, the actual spray rate should be established on site for the approval of the Engineer.

Aggregate for sand cover seals

The application rates of crusher dust or sand are recommended to fall within the range of 0.010 and 0.012 $m^3/\ m^2$ for sand cover seals. For bidding purposes a spread rate of 0.011 m^3/m^2 is recommended, however, the actual spray rate should be established on site for the approval of the Engineer.

Recommendation

Otta Seal is commonly applied to low volume roads worldwide, the requirements given above should be applied within the EAC partner states.

4.3.5.6 Sand Seals and Slurry

Binders

The specifications for Kenya, Tanzania and Uganda require bituminous binder for sand and slurry seals to consist of one of the following binders shown in the Table 47 or whichever is specified in the Special Specifications or the Bill of Quantities or ordered by the Engineer. As it can be noted from the table, the specifications of binder for sand seals and slurry with regard to its type and required properties are almost similar in the three manuals and more specifically this applies for the case of Tanzania and Uganda.

Table 47: Recommended binders for sand and slurry seals

	Kenya	Tanzania	Uganda
Sand Seal	MC 800 or MC 3000 or K1-60 cationic emulsion unless otherwise instructed by the Engineer. MC 3000 cut-back bitumen	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)
Slurry Seal	An anionic emulsion A4 (slow setting or rapid-setting) or a slow-acting cationic emulsion K3 unless otherwise instructed by the engineer	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)

Aggregate

The manuals for Tanzania and Uganda recommend crusher dust or clean river sand free from lumps of clay or any other deleterious matter as aggregate for sand seal, and it should comply with the requirements in Table 48. It is further recommended to screen or wash the sand to ensure compliance with the Specifications.

Table 48: Grading limits requirements for sand seal

Sieve size (mm)	Percentage passing sieve (% by mass)	
	Natural river sand	Crusher dust
10	100	100
5	85 – 100	85 – 100
1.18	20 – 60	20 – 80
0.425	0 – 30	-
0.300	0 – 15	-
0.150	0 - 5	0 - 30

The manuals also require the aggregate for slurry seals to be approved crusher sand obtained from a parent rock having a TFV value of not less than 110 kN or a mixture of such crusher sand and approved clean natural sand, where the mixture does not contain more than 25% of natural sand. The aggregate should be clean, tough, durable, angular in shape, and should meet the grading requirements given in Table 49.

Table 49: Grading limits requirements for slurry seal

Sieve size (mm)	Percentage passing sieve (% by mass)	
	Fine type	Coarse type
10		100
5	100	85-100
2	90-100	50-90
1.18	60-90	32-70
0.425	32-60	20-44
0.150	10-27	7-20
0.075	4-12	2-8

Recommendation

We recommend to the EAC partner states to adopt specifications shown in Tables 48 to 50 for sand and slurry seals.

Table 50: Recommended binders for sand and slurry seals

Sand Seal	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)
Slurry Seal	MC 800 cut back bitumen MC 3000 cut-back bitumen Spray-grade cationic emulsion (65% or 70% of net bitumen) Spray-grade anionic emulsion (65% or 70% of net bitumen)

4.4 Impact of Harmonisation of Specifications for Roadworks

The member countries do have specifications of their own. These proposed specifications are intended to provide the EAC members states Road Agencies with a quick reference to the use of specifications roadworks. It is intended that the provisions of these standards be a reference and not the final authority and as such the proposed specifications do not have a direct impact on the performance of existing road pavements. It is advised that member states should be making reference to the proposed specifications in the design, maintenance and rehabilitation of roads that form the regional trunk road network for uniform performance of this network. It can be stated that through harmonisation of specifications for road and bridge works optimum balance between road construction costs, properties of availability materials, environmental considerations, construction method, and previous experience of the performance of materials under consideration can be achieved during materials selection and design for roadworks.

In developing the proposed harmonization of specifications for road works across the member states we have taken account of the current specifications and the best practices. Adoption of the recommendations means that the member states will be using best practices in a consistently and the benefits of using best practices will be achieve uniformly in the region. Member states are therefore encouraged to revise their specifications within the next three to five years to ensure uniform realization of the benefits of adopting the best practices in the region.

BIBLIOGRAPHY

Republic of Kenya, Ministry of Transport and Communications (1986), Standard Specifications for Road and Bridge Works.

The Republic of Uganda, Ministry of Works, Housing and Communications (2005), General Specifications for Road and Bridge Works.

The United Republic of Tanzania, Ministry of Works (2000), Standard Specifications for Road Works.

APPENDIX 1

Table A1: Aggregate gradation requirements for asphalt concrete surfacing employing Marshall Mix Design method

Sieve size (mm)	Asphalt concrete continuously graded		
	AC20	AC14	AC10
20	80-100	100	-
14	60-80	85-100	100
10	50-70	72-94	85-100
5.0	36-56	52-72	55-72
2.36	28-44	37-55	38-57
1.18	20-34	26-41	27-42
0.600	15-27	16-28	18-32
0.300	10-20	12-20	13-23
0.150	5-13	8-15	9-16
0.075	2-6	4-10	4-10
Aggregate	95%	94.5%	94%
Bitumen content ASTM 2172-88	5%	5.5%	6%

Table A2: Marshall Mix Design requirement for asphalt concrete surfacings

<table>
<tr><th colspan="3">Property of mixture and laboratory test method</th><th>AC20, AC14 & AC10</th></tr>
<tr><td colspan="3">Marshall flow (mm)</td><td>2 - 4</td></tr>
<tr><td colspan="3">Marshall stability at refusal compaction (N), all severely loaded areas</td><td>Min. 9000</td></tr>
<tr><td rowspan="3">Marshall stability (2 × 75 blows) (N)</td><td rowspan="3">Traffic loads</td><td>$>10 \times 10^6$ esal</td><td>8000 - 18000</td></tr>
<tr><td>$1 - 10 \times 10^6$ esal</td><td>7000 - 15000</td></tr>
<tr><td>$<1 \times 10^6$ esal</td><td>6000 - 10000</td></tr>
<tr><td colspan="3">Air voids (%)</td><td>3 - 5</td></tr>
<tr><td colspan="2" rowspan="3">Voids in mineral aggregate (%)</td><td>AC20</td><td>Min. 14</td></tr>
<tr><td>AC14</td><td>Min. 15</td></tr>
<tr><td>AC10</td><td>Min. 16</td></tr>
<tr><td rowspan="3">Voids filled with bitumen (%)</td><td rowspan="3">Traffic loads</td><td>$>10 \times 10^6$ esal</td><td>65 – 75</td></tr>
<tr><td>$1 - 10 \times 10^6$ esal</td><td>65 – 78</td></tr>
<tr><td>$<1 \times 10^6$ esal</td><td>70 - 80</td></tr>
<tr><td colspan="3">Requirement after refusal laboratory compaction to BS 594 (severely loaded areas only)</td><td>Min. air voids 3%</td></tr>
<tr><td colspan="3">Indirect tensile strength (kPa) AASHTO T 283</td><td>Min. 800 tested at 25^0C</td></tr>
<tr><td colspan="3">Indirect wet tensile strength (kPa) AASHTO T 283</td><td>80% of dry strength</td></tr>
</table>

Table A**3**: Grading limits for combined aggregates for asphalt concrete surfacing courses employing Super-pave Mix Design method

Sieve size (mm)	19 mm Nominal size				12.5 mm Nominal size				9.5 mm Nominal size			
	Control points		Restricted zone*		Control points		Restricted zone*		Control points		Restricted zone*	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
25	100	-	-	-	-	-	-	-	-	-	-	-
19	90	100	-	-	100	-	-	-	-	-	-	-
12.5	-	90	-	-	90	100	-	-	100	-	-	-
9.5	-	-	-	-	-	90	-	-	90	100	-	-
4.75	-	-	-	-	-	-	-	-	-	90	-	-
2.36	23	49	34.6	34.6	28	58	39.1	39.1	32	67	47.2	47.2
1.18	-	-	22.3	28.3	-	-	25.6	31.6	-	-	31.6	37.6
0.600	-	-	16.7	20.7	-	-	19.1	23.1	-	-	23.5	27.5
0.300	-	-	13.7	13.7	-	-	15.5	15.5	-	-	18.7	18.7
0.075	2	8	-	-	2	10	-	-	2	10	-	-
	*) The Restricted Zone forms a band through which the gradation may not pass											
	Bitumen content for bidding purposes is 5.0%				Bitumen content for bidding purposes is 5.5%				Bitumen content for bidding purposes is 6.0%			

Table A4: Requirements for Super-pave Mix Design Gyratory Compaction

Design traffic (esal $\times 10^6$)	$N_{initial}$	N_{design}	$N_{maximum}$
< 0.3	6	50	75
0.3 - < 3	7	75	115
3 - < 30	8	100	160
≥ 30	9	125	205

Table A5: Design requirements for Super-pave Mix Design

Property of mixture and laboratory test method			Asphalt concrete, continuously graded (37.5, 25, 19, 12.5 & 9.5 Nominal Size)
Required relative density (% of G_{mm})	$N_{initial}$	$<0.3\times10^6$ esal	<91
		$0.3 - <3\times10^6$ esal	<90
		$3 - <30\times10^6$ esal	<89
		$\geq30\times10^6$ esal	<89
	N_{des}	All traffic loads	96
	N_{max}	All traffic loads	<98
Voids in mineral aggregate (%)	Aggregate size	37.5 mm	11.0
		25.0 mm	12.0
		19.0 mm	13.0
		12.5 mm	14.0
		9.5 mm	15.0
Voids filled with bitumen (%)		$<0.3\times10^6$ esal	70 – 80
		$0.3 - <3\times10^6$ esal	65 – 78
		$3 - <30\times10^6$ esal	65 – 75
		$\geq30\times10^6$ esal	65 – 75
Filler to binder ratio			0.6 – 1.2
Target air voids at Ndes			4%
Indirect dry tensile strength (kPa) AASHTO T 283			Minimum 800 tested at 25^0C
Indirect wet tensile strength (kPa) AASHTO T 283			80% of dry strength

Table A6: Grading requirements for Dense Bitumen Macadam employing Marshall Mix Design

Sieve size (mm)	DBM40 (% passing)	DBM30 (% passing)
50	100	-
37.5	95-100	100
28	70-95	90-100
20	-	70-95
14	56-76	58-82
10	53-70	52-73
5.0	39-56	40-56
2.0	24-40	24-40
1.18	19-35	19-35
0.425	9-25	9-25
0.300	7-21	7-21
0.075	2-9	2-9

Table A7: Design requirements for DBM course employing Marshall Mix design

Property of mixture and laboratory test method			Uganda DBM 40 & DBM 30
Marshall flow (mm)			2-4
Marshall stability at refusal compaction (N), all severely loaded areas			Min 9000
Marshall stability (2×75 blows) (N)	Traffic loads	$>10\times10^6$ esal	Min 7000
		$1.5\text{-}10\times10^6$ esal	Min 6000
		$<1\times10^6$ esal	Min 3500
Air voids (%)			4-8
Compaction levels	Traffic loads	Severely loaded areas*	To refusal**
		$>10\times10^6$ esal	To refusal**
		$1.5\text{-}10\times10^6$ esal	2×75
		$<1\times10^6$ esal	2×50
*Severely loaded areas should be identified in the Special Specifications			
**Requirement after refusal laboratory compaction to BS 594. Minimum air void 3%.			

Table A8: Grading requirements for asphaltic base course and binder course employing Super-pave Mix Design

Sieve size (mm)	37.5 mm Nominal size				25 mm Nominal size			
	Control points		Restricted zone*		Control points		Restricted zone*	
	Min	Max	Min	Max	Min	Max	Min	Max
50	100	-			-	-		
37.5	90	100			100	-		
25	-	90	-	-	90	100	-	-
19	-	-	-	-	-	90	-	-
12.5	-	-	-	-	-	-	-	-
9.5	-	-	-	-	-	-	-	-
4.75	-	-	34.7	34.7	-	-	39.5	39.5
2.36	15	41	23.3	27.3	19	45	26.8	30.8
1.18	-	-	15.5	21.5	-	-	18.1	24.1
0.600	-	-	11.7	15.7	-	-	13.6	17.6
0.300	-	-	10	10	-	-	11.4	11.4
0.075	0	6	-	-	1	7	-	-
	*) The Restricted Zone forms a band through which the gradation may not pass							
	Bitumen content for bidding purposes is 4.0%				Bitumen content for bidding purposes is 4.5%			

Table A9: Grading requirements for penetration macadam base course

Sieve size (mm)	Penetration macadam		
	PM80	PM60	PM30
Main fraction			
100	100	-	-
75	75-100	100	-
63	-	80-100	-
50	0-50	0-50	-
37.5	0-25	0-25	100
28	0-5	0-5	80-100
20	-	-	0-50
14	-	-	0-25
10	-	-	0-5
Key stone*			
50	100	-	-
37.5	85-100	100	-
28	0-50	85-100	-
20	0-25	0-50	100
14	0-5	0-25	85-100
10	-	0-5	0-55
6.3	-	-	0-25
5	-	-	0-10
Layer thickness (mm)	125	100	50
Bitumen type	Penetration grade 80/100 or 60/70		
Bitumen spray rate for bidding purposes (l/m2)	3.5	3.5	2.5

Table A10: Mix design requirements for cold mixed bituminous base course

Sieve size (mm)	**FBMIX (% passing)**	**BEMIX (% passing)**
37.5	100	100
28	80-100	80-100
20	60-95	60-95
10	42-78	35-70
5.0	30-65	25-50
2.0	20-50	18-35
0.425	10-30	10-25
0.075	5-15	5-8
Properties of mixture and laboratory test method	**FBMIX**	**BEMIX**
Bitumen type	Foamed	Emulsion
Nominal consumption of bitumen (litres/m^3 of compacted material) for bidding purpose	90	90
Marshall stability tested at 400C (N), BS EN 12591:2000	6000	4500
Marshall flow tested at 400C (mm), ASTM D 1559-89	2-4	2-4
Minimum E-modulus tested at 290C by indirect tensile strength method or alternative approved methods (MPa), ASTM D 3967	1600	1200
Minimum moisture content at the time of laying (%)	Mix design moisture less 1.5% points	
Maximum moisture content at the time of laying (%)	Mix design moisture plus 0.5% points	
Minimum compacted field density (% of Marshall dry density)	96	

Table A11: Requirements for single surface dressings

Chippings				
Sieve size (mm)	Chipping, nominal size of aggregates (% passing)			
	20 mm	14 mm	10 mm	7 mm
25	100			
20	85-100	100		
14	0-30	85-100	100	
10	0-5	0-30	85-100	100
6.3	-	0-5	0-30	85-100
5	-	-	0-5	0-30
2.36	-	-	-	0-5
Fines: 0.425	<0.5	<1.0	<1.0	<1.5
Dust: 0.075	<0.3	<0.5	<0.3	<1.0

Nominal Sizes	Max flakiness index (FI)	TFV (kN)	Ratio TFV soaked to dry (%)	ALD	Binder hot spray rate (l/m^2)	Aggregate Spread rate	Test Method
20	20	For AADT>1000:160 For AADT<1000:120	75	11 - 15	1.80	19 kg/m^2	BS812:105.1 & BS812:111
14	25			8 - 10	1.50	14 kg/m^2	
10	25			5 - 6.5	1.10	11 kg/m^2	
7	30			2 - 4	0.80	7 kg/m^2	

Table A12: Requirements for double surface dressings

Chippings				
Sieve size (mm)	**Chipping, nominal size of aggregates (% passing)**			
	20 mm	**14 mm**	**10 mm**	**7 mm**
25	100			
20	85-100	100		
14	0-30	85-100	100	
10	0-5	0-30	85-100	100
6.3	-	0-5	0-30	85-100
5	-	-	0-5	0-30
2.36	-	-	-	0-5
Fines: 0.425	<0.5	<1.0	<1.0	<1.5
Dust: 0.075	<0.3	<0.5	<0.3	<1.0

Nominal Size of aggregate (mm)	Max flakiness index (FI)	TFV (kN)	Ratio TFV soaked to dry (%)	ALD	Nominal rates of application		Test Method
					Binder spray rate (l/m^2)	Aggregate Spread rate	
20	20	For AADT>1000:160 For AADT<1000:120	75	11-15	1.8 for AADT<200	19 kg/m^2	BS812:105.1 & BS812:111
					1.5 for AADT 200-1000		
					1.2 for AADT>1000		
14	25			8-10	1.4for AADT<200	14 kg/m^2	
					1.1for AADT 200-1000		
					1.0 for AADT>1000		
10	25			5-6.5	1.2 for AADT<200	11 kg/m^2	
					1.0 for AADT 200-1000		
					0.9 for AADT>1000		
7	30			2-4	0.9 for AADT<200	7 kg/m^2	
					0.8 for AADT200-1000		
					0.7 for AADT>1000		

FINAL REPORT

ANNEX A 5

HARMONISATION OF ROAD AND BRIDGE MAINTENANCE STANDARDS

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GLOSARY AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ACS	Asphalt Concrete Surface
BMS	Bridge Maintenance System
COMESA	Common Market for Eastern and Southern Africa
DBSD	Double Bituminous Surface Dressing
DICC	Dar es Salaam International Conference Centre
DSM	Dar es Salaam
DUCA	District Urban Community Access
EAC	East African Community
FBU	Burundi Francs
HDM-4	Highway Development and Maintenance Model
HQ	Headquarters
IRI	International Roughness Index
IRP	Integrated Road Project
JICA	Japanese International Cooperation Agency
KeNHA	Kenya National Highways Authority
KRB	Kenya Roads Board
LCC	Life Cycle Cost
LOS	Level of Service
MININFRA	Ministry of Infrastructure
PMO-RALG	Prime Minister's Office Regional Administration and local Government
MCA	Multi-Criteria Analysis
MoIC	Ministry of Infrastructure and Communication
MoW	Ministry of Works
MoWT	Ministry of Works and Transport
MPI	Maintenance Priority Index
MWS	Maintenance Works Standards
OAG	Office of Auditor General
RAFU	Road Agency Formation Unit
RF	Road Fund
RFB	Road Fund Board
RICS	Road Inventory Condition Survey
RM	Routine Maintenance
RMF	Road Maintenance Fund
RMI	Road Maintenance Initiative
RMMS	Road Maintenance Management System
RMLF	Road Maintenance Fuel Levy
ROMDAS	Road Measurement Data Acquisition System
RRA	Rwanda Revenue Authority
RTDA	Rwanda Transport Development Agency
Rwf	Rwanda Francs
SBSD	Single Bituminous Surface Dressing
TANROADS	Tanzania National Roads Agency
TRA	Tanzania Revenue Authority
UNRA	Uganda National Road Agency
URF	Uganda Road Fund

USD	United States Dollars
ZRA	Zanzibar Roads Authority
ZRFB	Zanzibar Road Fund Board

EXECUTIVE SUMMARY

This part deals with the review of the existing road and bridge maintenance manuals with the aim to identify areas of harmonisation and improvement in the EAC partner states. It presents a general review of road and bridge maintenance problem as well as maintenance principles. It goes further to present the road and bridge maintenance situation and practices within East African partner states which include Burundi, Kenya, Rwanda, Tanzania and Uganda. Most importantly, the study recommends harmonisation and improvement areas and the impact of implementing the recommendations.

A review of institutional arrangement indicates that all partner states have put in place similar institutions for carrying out road and bridge maintenance. In each partner state there is a ministry that deals with road and bridge maintenance at policy and coordination level.

Each partner state has established an executive agency that is responsible for day to day management of road and bridge maintenance activities. In addition, each partner state has established a body that deals with funding of road and bridge maintenance. Differences in terms of sources of funds and distribution criteria between different classes of roads are observed from one country to another.

However, we have observed that the road maintenance policies in all partner states do not reflect road user needs in terms of defining clearly the level of service and economic benefits to be provided to road users. Furthermore, in all partner sates, there is no mention of the need to have similar level of service across all EAC partner states.

Furthermore, we observe that EAC partner states are at different levels of Road Maintenance Management System (RMMS) and Bridge Management System (BMS) development and utilisation. In addition, countries which have attempted to develop these systems use diverse principles and criteria in terms of data collection methods, maintenance standards, determination of road and bridge maintenance plans and programmes and execution of road and bridge maintenance activities. This results into different road and bridge condition and hence different level of service and economic benefits which road users get from one partner state to another.

Insofar as implementation phase is concerned, all EAC Partner states are phasing out the force account methods. However road maintenance contracts that have been introduced in most countries are based on a schedule of unit rates and measured quantities. In addition, Tanzania has introduced performance based contracts which have not performed very well so far. Furthermore, we are informed that Kenya is developing standards for the performance based contracts.

In this regard, we give recommendations for harmonization of the following issues:

Policy Framework

We propose that in all partner states the road and bridge maintenance policy should reflect the customer (road users) needs. The policy should include, inter alia, the following goals:

- Support the socio-economic goals at national and EAC level.
- Provide a minimum Level of service to the road users across EAC partner states
- Provide Safe roads
- Minimise the sum of road agency and user costs
- Minimise damage to the environment

Data collection Procedures

Data collection is very expensive. In this regard, the cost of data acquisition and keeping them current is likely to be the most expensive aspect of implementing and operating a highway management system. It is therefore essential that appropriate data design is undertaken to ensure that the data to be collected for a particular purpose within the highway management cycle is relevant, appropriate and reliable within the available resources i.e. affordability of the responsible agency.

We recommend that surface condition and structural evaluation should be done through roughness measurements using Vehicle Mounted Bump Integrator (VMBI) and Dynamic Cone Penetrometer (DCP) measurements, respectively. Previous studies have shown that these methods provide relatively accurate results at reasonable cost. In so far as traffic is concerned, it is proposed that manual classified traffic counts and axle load measurements should be carried out regularly.

Road Surface Riding Quality

The harmonization of road maintenance standards should be able to address the cross border riding quality to fulfill the functional requirement of traffic and entire travelling public. This entails the adoption of uniform and consistent intervention levels for road surface maintenance within EAC partner states.

We recommend that road surface intervention levels be based on economic analysis and aiming at keeping the road network in good condition with specified target roughness of not more than 4.0 m/km IRI. Studies have shown that beyond this roughness level the riding starts becoming uncomfortable at operating speeds of around 80 km/hr.

Pavement Structural Capacity

The amount and quality of maintenance the roads receive should also aim at keeping their load carrying capacity consistent from one road section to the other and from one country to another. This requires uniform and consistent intervention levels for structural maintenance of the roads.

We recommend that assessment of structural capacity should be based on maintaining strong pavements with specified minimum modified structural number of 3.0. This is a critical level of structural capacity for roads carrying heavy traffic.

Principles of Road Maintenance Management Systems

The harmonization process should result into provision of economic and technical efficiency across the region. We therefore recommend that the Road Maintenance Management Systems (RMMS) should be based on economic analysis (Life Cycle Cost and Benefit Analysis) aiming at minimising the sum of agency and road user cost while maintaining a certain minimum level of service to the road users. Experience has shown that economic based methods result into improved road network condition, over time, at a relatively low budget. HDM4 can be a very useful tool as support software for Road Maintenance Management Systems. In this regard, we advise Partner States to evaluate, for possible adaptation, the TANROADS Road Maintenance Management System which is at a very advanced stage.

Bridge Maintenance System

Bridges need to be properly maintained for effective and smooth movement of goods and people through the road network. However, the review carried out in this study indicates that most EAC partner states have not done much in development and utilization of Bridge Maintenance Systems.

At the moment Tanzania is relatively advanced in this aspect by developing and operational Bridge Maintenance System known as TANBRIDGEMAN.

We propose here that, rather than re-inventing the wheel, further evaluation of the Bridge Management System for Tanzania (BMST) should be done by individual EAC Partner States before the final decision of adapting the system is made.

Road Maintenance Methods (Implementation)

Many countries around the world have introduced new ways of contracting out road maintenance by using performance specified contracts. Experience has shown that these methods reduce maintenance costs and improve road conditions. Unfortunately, improper implementation of these schemes can produce adverse effects. Nonetheless, we noted that Uganda is going back to force account because of lack of local contractors' capacity to carry out maintenance works. It is our sincere hope that they will work on contractors' capacity building so that they can gradually go back to road maintenance contracts.

We recommend that Partner States should continue with a traditional method of contracting out road maintenance based on a schedule of unit rates and measured quantities. However, a gradual approach of introducing performance based contracts should be adopted, starting with short-term pilot contracts with simple performance standards such as control of potholes and cracks and cleaning of the drainage system.

Environmental Protection

As with road construction activities, road maintenance can contribute to soil erosion, disturbance of water flows, chemical pollution, traffic disruption, noise, dust, and other impacts on surrounding communities and natural life. In this regard, an appropriate level of analysis, which is project specific, should be undertaken to determine the likely effects of the maintenance works on the environment.

We recommend that bid documents and road maintenance contracts should include clauses which require contractors to prepare environmental management plan outlining mitigation measures for all environmental issues identified. Clauses for use in Road Maintenance activities should include, inter alia, work-site installations, preparation and supply of gravel materials in pit or quarry, material disposal after cleaning ditches and other drainage facilities, tree planting, minimizing occupational health and safety risks.

Consistent Monitoring and Evaluation

Effective highway management requires that policies, standards and the effectiveness of programmes are reviewed on a regular basis. This can only be possible if a feedback mechanism, which is done through monitoring and evaluation, is in place.

We recommend that Partner States should develop a comprehensive, harmonised and up to date database which would facilitate information sharing across EAC partner states. This would assist in determining whether the programme was achieved according to specified standards and, most importantly, assess the effectiveness of the programme in terms of meeting the specified level of service and economic benefits to the road users. The results of this evaluation shall form a basis for review of policies and maintenance standards.

5 HARMONISATION OF ROAD AND BRIDGE MAINTENANCE STANDARDS

5.1 Introduction

5.1.1 General

The highway network of any country is one of the major public investments designed to support the national economy. When well developed and maintained, the road network is expected to meet the national objectives for road transport. These objectives have to be achieved while minimising the Life Cycle Cost (LCC) of the facilities as well as the transport cost of goods and people.

Roads are designed to cater for both structural and functional requirements of traffic and the entire travelling public. The quality of service they provide may differ by country, agency, road class, traffic levels and pavement type. The target quality of service determines the level of condition to which a road is allowed to fall before a certain treatment is triggered. Once this level is reached, the responsible agency is required to intervene by providing an appropriate treatment. In this case, road maintenance standards, sometimes known as intervention levels, refer to pavement surface and/or structural condition at which particular treatments are carried out. In setting up the maintenance standards, both agency and road user costs have to be considered. Relatively high maintenance standards result in considerable savings in road user costs but require high maintenance budgets. Naturally, ideal maintenance standards would aim at providing a fair trade-off between agency and road user cost. This part of the study reviews road and bridge maintenance standards used in EAC partner states and identifies areas of convergence as well as divergence. On the basis of this review, potential areas for harmonisation are identified.

5.1.2 Background and Objectives of the Study

This section addresses a part of Thematic area 1 namely; Harmonisation of Standards and Specifications. The thematic area includes nine sub themes which are:

i. Harmonisation of road geometric design standards
ii. Harmonisation of road pavement and bridge design standards
iii. Harmonisation of specifications for road and bridge works
iv. Harmonisation of road and bridge maintenance standards
v. Harmonisation of road signs, traffic signals and marking
vi. Harmonisation of vehicle safety and fitness
vii. Harmonisation of driver testing and training manuals
viii. Harmonisation of vehicle dimensions and combinations
ix. Harmonisation of transportation of abnormal, awkward and hazardous loads

This part of the study deals with the review of the existing road and bridge maintenance manuals with the aim to identify areas of harmonisation and improvement in the EAC partner states. It presents a general review of road and bridge maintenance problem as well as maintenance principles. It goes further to present the road and bridge maintenance situation and practices within East African partner states which include Burundi, Kenya, Rwanda, Tanzania and Uganda. Most importantly, the chapter suggests harmonisation and improvement issues.

5.1.3 Methodology

Similar to other parts of the project, the task included a desk study of the documented general road and bridge maintenance problems and principles. The task also deals with the collection and detailed review of the existing road and bridge maintenance manuals within East African partner states. In addition, participatory approach is adopted. This approach entails the involvement of relevant experts and stakeholders by conducting structured interviews as well as experts and stakeholders workshops in all 5 countries which form the East African Community.

The experts' views were collected through expert and stakeholders workshops between 4th June and 25th July 2011 in all EAC partner states. These meetings were held as presented in Table 1.

Table 1: Expert and Stakeholders Workshops

S/N	Country	City/venue	Date	No of Participants
1	Kenya	Nairobi/680 hotel	4th July 2011	19
2	Burundi	Bujumbura/White Stone	6th July 2011	40
3	Rwanda	Kigali/Sportsview hotel	8th July 2011	21
4	Uganda	Kampala/Ridar Hotel	11th & 12th July 2011	38
5	Tanzania	DSM/DICC	15th July 2011	38
		Zanzibar/Ocean View Hotel	25th July 2011	17

In addition, the task force review meeting, which brought together delegates from all EAC Partners States, was held in Dar es Salaam from 19 to 23 September 2011. Inputs, comments and suggestions from the experts' workshops and task force meeting have been incorporated in this report.

5.2 Overview of Road and Bridge Maintenance Standards

5.2.1 Road and Bridge Maintenance Problem

It is well documented that the problem of managing highway maintenance has proved to be a particularly difficult issue for many countries around the world. A great challenge is that highway networks are usually spread over a wide geographic area, and their condition is changing continuously as a result of the effects of traffic, climate and environmental conditions. Invariably, the effect of these factors on maintenance interventions is complex, and is often difficult to predict, particularly over the longer term. It can therefore be seen that maintenance is, essentially, a management problem of getting the right people, materials and equipment, to the right place on the highway network, to carry out the right remedial or preventive work, at the right time and at the right cost. This process continues day after day and year after year.

However, over the decades highways professionals have traditionally been preoccupied with design and construction of new roads without giving due attention to adequate preservation of the investment. Basically, maintenance principles are different from normal project principles, in that projects have a clearly defined beginning and end, and require the consumption of resources to move from the start to the finish. Although maintenance consumes resources, it is an on-going activity with no start or end. As such, it is a process rather than a project. Whereas traditional engineering management uses the techniques of project management, it can be seen that highway management needs to draw on the wider resources of business management if it is to be undertaken effectively.

A new road is expensive. However, building a new road is a one-off cost. On the other hand, the cost of maintaining a road network is relatively low but has to be paid time and time again. If the maintenance is neglected it will cost five or six times as much to restore the road. Rationally, if the country is stressed for money an economically feasible cause of action would be to maintain the existing roads before investing in new ones. In addition, the road maintenance should be done today because tomorrow it will be more expensive.

The types of works to be undertaken in the highway management process can be conveniently grouped as follows:

Routine Maintenance – Maintenance works required continuously. These may comprise of activities such as grass cutting, drain cleaning, culvert and bridge cleaning paved road patching, edge repair, crack sealing, unpaved road grading shaping and pothole repairs.

Periodic Maintenance: Maintenance works carried out at intervals of several years. The activities may include resealing, overlays of less than 100 mm, fog sprays and shoulder reforming.

Emergency Maintenance: Maintenance activities required to open or repair roads, bridges and other parts of road infrastructure after a natural or unforeseen disaster.

Spot Maintenance: Maintenance works carried out on short sections (typically 1 km or less) of roads in order to ensure reasonable level of passability. The works comprise of activities such as road surface repairs, embankment repair, culvert and drainage repairs, localised road shaping and re-gravelling and construction of diversions.

5.2.2 Maintenance Management Principles

Almost all countries the world over are confronted with acute shortage of resources and genuine competing demands for the same from various development and social service programmes. Consequently, not all road sections that need maintenance or rehabilitation can be funded. Under such budget constraints, an agency responsible for road maintenance should be able to prepare road maintenance programmes based on the following questions:

- Which treatments and for which road sections have the highest pay-off?
- Which road sections can be funded under the given budget constraints?
- What are the effects of various budget levels on long term road network condition and the associated agency and user costs?
- What is the optimum budget level which would keep the road network at the desired level of serviceability?

To answer these questions, a proper Road Maintenance Management System (RMMS) which is capable of looking into the future and predict the network condition under different maintenance strategies is essential. In addition, not only that the RMMS must be able to select cost-effective maintenance programmes but also it must have the ability to optimise maintenance expenditures under budget constraints. Such tools help to decide when, where and what should be done to keep the roads in economically efficient condition.

However, it is not only the problem of road maintenance prioritisation under budget constraint that has to be solved. There should be a proper communication between road administration and decision makers. This is mainly due to the fact that decision makers want to be sure that funds allocated for the road maintenance are used properly. For proper management and adequate fulfilment of the decisions made in the road sector, the following should be in place:

- An established and up kept legal and institutional framework

- A clear definition of the role of the Ministry and the role of the Road Administration
- A policy framework for the entire road sector
- Stable funding for longer term
- Proper managerial process or procedure for road maintenance
- Improvement in the overall management of road maintenance activities
- Competence-based education and training of human resources in the road sector.

Inadequate highway maintenance is a problem for many countries. This can be caused by several factors, including:

- Insufficient funds, including a lack of foreign exchange
- Shortage of qualified staff
- Absence of machines and spare parts
- Deficient institutional arrangements
- Lacking management capability

However, the most common reason given for inadequate maintenance is the difficulty in securing the necessary funds. Although this is a problem in a large number of countries, it is also apparent that, in many situations, the funds allocated for maintenance are often poorly utilized: funds being diverted to non- maintenance activities, being used on activities with political rather than economic priority, or being used to support large labour forces which are unproductive.

Studies of highway operation in the developing and emerging countries have consistently highlighted deficient institutional arrangements and management practices as major reasons for inadequate highway maintenance. Highway authorities sometimes have too many responsibilities. Too much emphasis is often put on execution, whilst planning, control and evaluation are neglected.

5.2.3 Strategy for Effective Maintenance Management

Figure 1 and subsequent sections show and clearly present the strategy advocated for highway management.

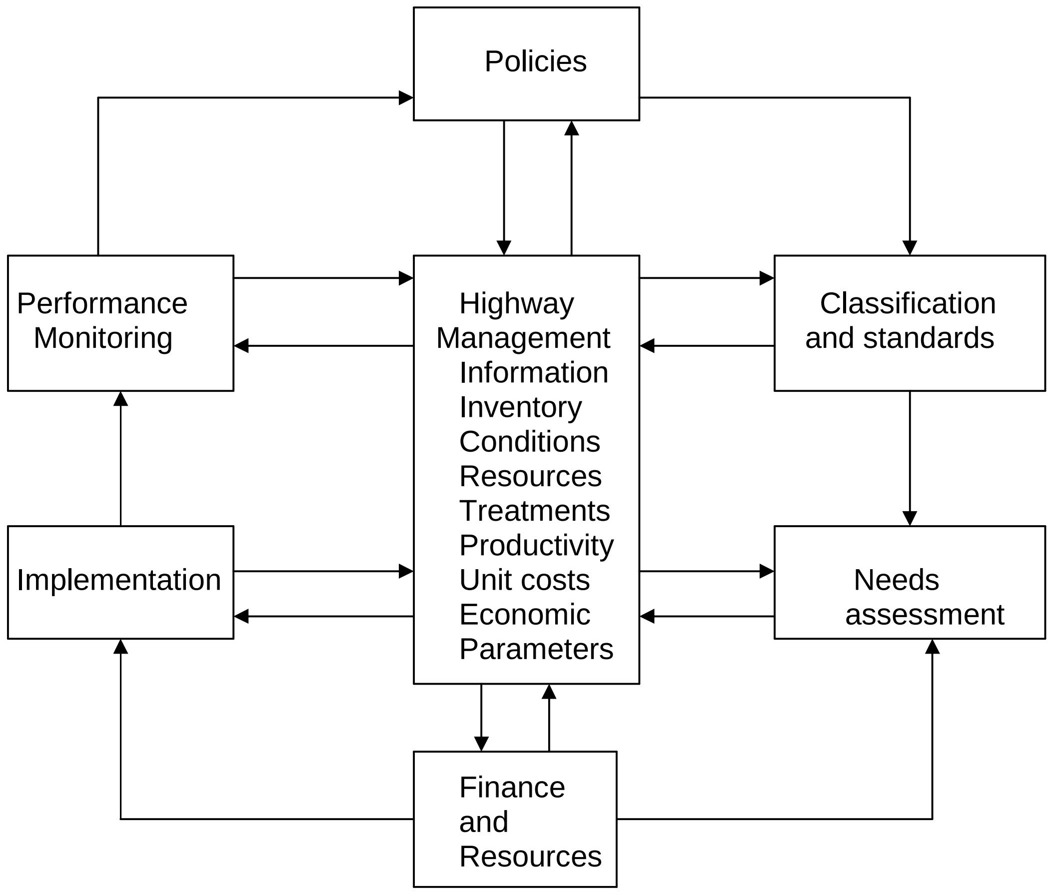

Figure 1: The Highway Management Cycle

5.2.3.1 The policy framework

Policies are a key issue in highway management. They define the levels of service which the road authority intends to provide for its customers in terms of level of comfort and economic benefits. Objectives should be designed to provide a sub-sector focus within the policy framework which has been defined. This might simply be to retain the condition of the highway network at its present level, or to upgrade the condition of footways in urban areas.

5.2.3.2 Classification and standards

Standards are the defined levels of condition or response which the highway authority would wish to meet, or exceed, in its highway management activities. For structural pavement maintenance, standards can be warning or intervention levels; for routine maintenance, standards can be the prescribed frequencies for carrying out particular activities. Physical standards will include such things as levels of deterioration, while operational standards will deal with matters like response times to emergency of carrying our cyclic operations and the assurance of satisfactory level of service to the public.

Standards and level of service will vary depending upon the nature of the highway and traffic. Highways are, therefore, allocated to categories which form a classification. There will also be variation to suit other circumstances such as terrain or environment.

The definition of standards and their application to particular highways creates a base for considering options and priorities, determining costs securing finance or monitoring achievement and value for money.

5.2.3.3 Needs assessment

The definition of standards provides the basis of assessing maintenance needs. The actual situation or conditions are compared with standards to see where there are shortfalls which must be addressed by highway works.

5.2.3.4 Finance and resources

Maintenance needs identified as part of the assessment process will normally be greater than the resources available for remedial works. A rational system of setting priorities is therefore required to allocate available resources in a systematic and equitable way, such that best value for money is obtained.

It is essential that staff involved in highway management should have some understanding and appreciation of the processes that lead to the provision of their annual financial allocation. Such wider understanding contributes to the more effective management of highways.

5.2.3.5 Implementation

Procedures are necessary for procurement, implementation and supervision of work. It is important that, having defined standards and responses, the achievement of this is subjected to scrutiny and value-for-money consideration.

5.2.3.6 Performance monitoring

A review process must form an integral part of needs based highway management. From a practical point of view, the key performance indicators are:

- Was the programme achieved?
- Were the standards achieved?
- Was the work carried out efficiently?

Clearly, the preceding discussion indicates that, a highway maintenance management information system lies at the heart of the highway management cycle and the flow of decision based upon the information system, as shown; enable the strategy to be put in place.

The need to assess physical condition, safety, level of service, and efficiency of operation of highway systems is widely recognized. In addition to knowing the characteristics of the existing system, it is becoming increasingly important to be able to predict the effects that proposed policies are likely to have in the future. Such predictive capabilities enable the decision maker to test alternative courses of action to determine which policies and strategies will be the most effective in accomplishing the desired goals the resources available.

Data are, therefore, needed to provide the basis for management decisions on such aspects as:

- Assessing current levels of highway and bridge condition
- Determining appropriate levels of investment
- Prioritizing capital improvements and investments in maintenance

- Simulating the effects on any improvements on the future condition and performance of the highway system
- Estimating the cost of improvements
- Controlling on-going expenditures

It has been argued that management information of this nature provides the quantitative basis of dialogue between technical departments and ministries of finance during the fund allocation process. It can also provide the quantitative basis of a dialogue with elected representatives, and for monitoring departmental performance and the meeting of policy objectives.

5.3 Situational Analysis

5.3.1 Institutional Arrangement in EAC Partner States

Table 2 presents the existing institutions responsible for road maintenance management in the East African Community Partner States.

Table 2: Institutions responsible for Road Maintenance Management

S/N	Country	Ministry Responsible for Major Roads	Road Maintenance Authority	Road Maintenance Funding Authority
1	Burundi	Ministry of Transport, Works & Equipment	Roads Office	National Road Fund
2	Kenya	Ministry of Roads	Kenya National Highways Authority	Kenya Road Fund Board
3	Rwanda	Ministry of Infrastructure	Rwanda Transport Development Agency	Road Maintenance Fund
4	Tanzania	Ministry of Works Ministry of Infrastructure and Communication (Zanzibar)	TANROADS Zanzibar Roads Authority (to be established soon)	Road Fund Board Zanzibar Road Fund Board
5	Uganda	Ministry of Works and Transport	Uganda National Roads Authority	Uganda Road Fund

5.3.2 Existing Road and Bridge Maintenance Manuals

As explained in the methodology section, one of the basic activities is the collection and detailed review of the existing manuals in EAC partner states. The situation is as presented in Table 3 while Table 4 presents the existing bridge maintenance manuals.

Table 3: Existing Road and Maintenance Manuals

S/N	Country Name	Road Maintenance Manual/document
1	**Burundi**	None
2	**Kenya**	Road Maintenance Manual (May 2010)
3	**Rwanda**	• Road Maintenance Strategy (2008) • Proposal for MWS for Rwanda (2011)
4	**Tanzania**	▪ Data Collection Manual (2005) ▪ Maintenance Supervision Manual ▪ Road Maintenance Handbook volumes 1 &2 ▪ Standard Specification for Road Maintenance Works (2010) ▪ Performance Agreement for Road Maintenance Works in Unguja and Pemba Islands (2011)
5	**Uganda**	Road Maintenance Management Manual (July 2010)

Table 4: Existing Bridge Maintenance Manuals

S/N	Country Name	Bridge Maintenance Manual
1	**Burundi**	None
2	**Kenya**	Bridge inspection manual
3	**Rwanda**	None
4	**Tanzania**	- Handbook for Bridge Inventory - Handbook for Bridge Inspection - Handbook for Bridge Maintenance
5	**Uganda**	None

5.3.3 Country-wise Review of Current Maintenance Practices

5.3.3.1 Road Maintenance in Burundi

The Government of Burundi established an independent Roads Office with a responsibility to carry out routine and periodic road maintenance.

In addition, the government established the National Road Fund in 2004 with a clear mandate to finance the maintenance of the Road Network in the country. The fund receives the programme of road network maintenance from the Road Agency known as Roads Office. After the approval by the Road Fund Board the maintenance programme is funded in accordance with the available funds. The main sources of the road fund are:

- Fuel levy: FBU 80/litre of petrol/diesel. This contributes about 90% of the total revenue
- Transit Charges USD 175 articulated trucks and USD 85 for other heavy trucks

- Road license according to the engine capacity of a vehicle (once a year)
- Drivers' license fees

The road maintenance funding capacity has been increasing from 25.6% of the requirements in 2004 to 62.2% in 2010.

The fund classifies the road network into National, Provincial and Community roads. Road maintenance programmes are based on requests from provinces. However, the fund distribution does not follow any fixed proportions for different road classes.

Generally, Burundi is still at early stages of developing Road and Bridge Maintenance Management systems.

5.3.3.2 Road Maintenance in Kenya

Organisation of Road Maintenance in Kenya

The Kenya Roads Board (KRB) was established through an Act of Parliament, KRB ACT No. 7, in 1999 and was given presidential assent on 6th January 2000. The Act commenced on 1st July 2000 and the Board of Directors was appointed. The act specifies the following as the mandates of the board:-

- Administer the funds derived from the Road Maintenance Levy Fund (RMLF) and any other funds that may accrue to it;
- Coordinate the development, rehabilitation and maintenance of the road network, with a view to achieving efficiency, cost effectiveness and safety;
- Coordinate the implementation of all policies relating to the development, rehabilitation and maintenance of the road network;
- Determine the allocation of financial resources from the RMLF or from any other source available to the Board required by road agencies for the development, rehabilitation and maintenance of the road network and
- Monitor the operations or activities undertaken by road agencies in the development, rehabilitation and maintenance of the Kenyan road network.

Subsequently, the Kenya Roads Bill of 2007 established the Kenya National Highway Authority (KeNHA) with the responsibility for the management, development, rehabilitation and maintenance of national roads.

KeNHA Vision

"A leading Highways Authority committed to quality, safe and adequate national trunk roads".

KeNHA Mission

"To construct and manage national trunk roads that enhance socio-economic growth and prosperity".

Country Road Network

Based on the Road Inventory and Condition Survey (RICS) which was done in 2001, the road network was established to be 160,886 km long comprising of 11,189 km of paved roads and 149,689 km of unpaved roads.

Road Maintenance Management System

The road maintenance manuals developed with assistance from JICA have been updated to incorporate changes that have taken place in recent years with the initial three manuals combined into one and reprinted. The manuals are to be used by road stakeholders, including local authorities, consultants and small to medium contractors. The manual is a standardized tool of maintaining the road network in the country. It is a practical approach to handling road maintenance at whichever level.

Efforts are being made to start developing a Road Maintenance Management System. In addition, a proper Bridge Maintenance System (BMS) has not been developed.

Road Maintenance Funding

The Road Maintenance Levy Fund Act of 1993 introduced a fuel levy which is used for funding of road maintenance. This Act empowers the Minister responsible for roads, in consultation with the Minister responsible for finance to impose on any petroleum fuels entered for home use a road maintenance levy. The resources generated from fuel levy and road transit charges are deposited into the Road Maintenance Levy Fund.

The distribution is done in the following proportions:

- 60% International and National trunk and primary roads
- 24 % secondary roads
- 16% rural roads

5.3.3.3 Road Maintenance in Rwanda

Organisation of Road Maintenance in Rwanda

Law No. 02/2010 of 20/01/2010 established an Agency in-charge of transport. The agency is known as "Rwanda Transport Development Agency".

Mission of RTDA

- To manage and control national road network with a view to achieving road safety and maintenance
- To control airport infrastructure in order to ensure their maintenance and safety
- To manage and control waterways transport infrastructure with a view to ensuring their value added
- To develop railway infrastructure in Rwanda

Even before the establishment of RTDA, Rwanda had developed a Road Maintenance Strategy as a main guiding tool in the management of road maintenance activities in the country. This strategy, which was developed in 2008, provides a framework for programming, planning and execution of maintenance activities.

Furthermore, the strategy places much emphasis on the participation of local communities in road maintenance activities. It also considers employment creation by placing adequate emphasis on labour based road engineering technology.

The strategy clearly documents that the level of expenditure directed towards road maintenance activities is expected to rise as more and more newly constructed and rehabilitated roads are placed on a maintenance programme for either routine, recurrent, periodic or emergency maintenance.

Country Road Network

The Rwanda road network is grouped into classified and unclassified roads.

According to the road maintenance strategy, the Classified Road Network in Rwanda is categorized into 2 classes of roads; National Roads and District Roads. These can be either Paved or Gravel.

On the other hand the Un-classified Road Network consists of rural feeder roads and urban roads in District Capitals and other urban centres that are not sections of the classified national roads.

Kigali City Council is charged with the responsibility of managing the Urban Roads that do not fall under the classified national road network in the City of Kigali.

Road Maintenance Management System

The strategy indicates that condition surveys of the classified road network that are planned to be carried out periodically shall form the basis of the road maintenance management system. Road condition surveys of the feeder road network are normal ongoing operations that are carried out by the district infrastructure management units and help in forecasting maintenance activities for the year ahead. A maintenance strategy to fit the expected budget is developed based on the budget available for road maintenance.

In the Maintenance programme poor roads are not maintained, (except for Emergency Maintenance), until rehabilitation has taken place. Emergency maintenance is carried out as and when required on all classified roads irrespective of their importance or current condition. Routine Maintenance is carried out on all Paved and Un-paved National Roads which are in good or fair condition.

Recurrent Maintenance is carried out on all Paved National roads which are in good condition. It will be carried out on all Unpaved National and District roads according to priority and available funds based on traffic, condition and population data. Periodic Maintenance is carried out on all Paved National roads which are in good condition and on all unpaved National and District roads according to priority and available funds based on traffic, condition and population data.

The strategy plans to carry out traffic counts on the classified road network twice a year. Each road shall be assessed once in the rainy season and once in the dry season with a 12 hour classified traffic count.

Furthermore, the strategy specifies that, for feeder roads, traffic surveys will be carried out every 2 years and will be 12 hour classified traffic counts.

Overall condition and Maintenance standards for unpaved roads

To provide an overall measure condition and maintenance standards for unpaved roads the intervention levels presented in Table 5 are specified according to the Road Maintenance Strategy.

Table 5: Intervention Levels for unpaved Roads in Rwanda

S/N	CONDITION	DESCRIPTION	INTERVENTION
2	Good	allowable vehicular speed at 40 km/hr	routine grading and spot repairs
3	Fair	allowable vehicular speed at 20-40 km/hr	immediate periodic maintenance and only after this has been carried out will they be scheduled for routine, recurrent and periodic maintenance. Emergency maintenance can be carried out before periodic maintenance if required
4	Poor	allowable vehicular speed at 20 km/hr	reconstruction and major drainage works

Overall condition and Maintenance Standards for Paved Roads

To provide an overall measure of pavement condition and maintenance standards for paved roads the intervention levels presented in Table 6 are specified according to the Road Maintenance Strategy.

Recently, RTDA has embarked on the new effort to develop a Road Maintenance Management System based on HDM4. The system specifies Maintenance Work Standards (MWS) for National paved roads and unpaved roads and District unpaved roads. Using HDM4 7 year work programme has been prepared. However, the proposed system is still under trial. The system uses maintenance treatments that suit local conditions considering long term practice in the country. The following maintenance treatments are being used in Rwanda:

- Routine maintenance
- Patching potholes
- Crack sealing
- Resealing with Single and Double Bituminous Surface Dressing
- Asphalt Overlay-50 mm
- Partial reconstruction with 50mm AC and granular base and/or recycling of existing base
- Reconstruction

Table 6: Intervention Levels for Paved Roads in Rwanda

S/N	Condition	Description	Intervention
1	Good	substantially free of defects allowable vehicular speed at least 60 km/hr	Routine maintenance, scheduled for routine, recurrent and periodic maintenance
2	Fair	allowable vehicular speed at 40 - 60 km/hr	Immediate periodic maintenance and only after this has been carried out will they be scheduled for routine, recurrent and periodic maintenance. Emergency maintenance can be carried out before periodic maintenance if required
3	Poor	allowable vehicular speed at 40 km/hr	Immediate rehabilitation or reconstruction

The maintenance work standards proposed for the national road network of Rwanda are presented in Table 7.

Table 7: Proposed Maintenance Work Standards for National Roads in Rwanda

Roughness Range (IRI)	Damaged Area %	Traffic Range MT AADT				
		< 500	**500-1000**	**1001-2000**	**2001-3000**	**>3000+**
> 3.5	≤ 30%	RM	RM	RM	RM	RM
	> 30%	RM	12 mm SBSD	12 mm SBSD	12 mm SBSD	25 mm DBSD
3.5 ≤ 5.5	ALL	12 mm SBSD	12 mm SBSD	25 mm DBSD	25 mm DBSD	50 mm overlay
5.5 ≤ 8	ALL	12 mm SBSD (SC)	25 mm DBSD (SC)	50 mm overlay	60 mm overlay	70 mm overlay
8 ≤ 10.5	ALL	25 mm DBSD (SC)	50 mm overlay	Strengthen with 60mm ACS	Strengthen with 70mm ACS	Strengthen with 70mm ACS
> 10.5	ALL	Strengthen With 40mm ACS	Strengthen With 50mm ACS	Recon with 60 mm ACS	Recon with 70 mm ACS	Recon with 70 mm ACS
>12	ALL	Recon with 40 mm ACS	Recon with 50 mm ACS			

It has to be mentioned here that this is preliminary effort to develop maintenance work standards for Rwanda. The trial runs of the proposed standards shall lead to reviews which will be based on the outcome of the economic analysis. It is recognised that necessary data for the trial run are not available. However, efforts to collect the data are underway.

Road Maintenance Funding

On the 5th of November 1998 by the law no. 14bis/98, the National Road Fund was renamed Road Maintenance Fund (RMF) and the resources were fixed. The latest changes have been documented in law no. 52bis/2006 from 12th of December 2006, determining the attributions, structure and functioning of the Road Maintenance Fund.

In accordance to the newest RMF-Law, article 3, RMF shall have the main attribution of receiving, effectively manage and distribute funds for the maintenance of public roads as determined by a Presidential Order. It has the following particular attributions:

- To collect and effectively manage funds received from sources provided for by article 19 of this law;
- To collaborate with other relevant organs in preparation of road maintenance programs which are RMF funded;
- To examine project studies and the bidding documents for road maintenance before launching tenders;
- To monitor the activities in technical terms and finance disbursed in order to ensure that activities are carried out as planned in the signed contract.

The Fund carries out payment for maintenance activities, monitoring, and technical as well as financial audits on the basis of invitations to tender, in conformity with the law regarding the awarding of public offers as well as the cost of the evaluations and programming of road maintenance activities.

RMF financial resources (recurrent budget)

Road Maintenance Fund's law defines by article 19a number (11) of types of resources:

- ♦ State budget;

- Government / donor subsidies;
- Funds from activities performed by RMF;
- Interest from investments;
- Road user charge levied on gas, oil & petrol;
- Road toll levied on foreign registered vehicles;
- Annual road toll levied on locally registered vehicles;
- Fines levied on overloaded transport vehicles;
- Compensation for damages caused to the road sector;
- Fines paid by persons who contravene the road traffic law;
- Donations & bequests.

The current revenues of the Road Fund are derived from:

(i) a petrol and diesel levy (at present FRW 62.37 Rwf per litre (€ 0.076) for both petrol and diesel), which is about 69.4 % of the revenues,

(ii) a road toll (based on COMESA regulations) charged on foreign heavy vehicles (29.6 % of fund revenues), and

(iii) a number of other sources (mainly fines paid for overloading vehicles and for contravening the traffic law). These other sources generate about 1 % of the total revenues.

Revenues from fuel levy and road toll levied on foreign registered vehicles are collected by Rwanda Revenue Authority (RRA) and transferred into the Road Maintenance Fund account held in the National Bank of Rwanda (NBR).

The distribution is done in the following proportions

- MININFRA(National Roads) 80%
- Districts (Districts roads) 0%
- Kigali City (Kigali Urban roads) 18%
- RMF for running activities 2%

Bridge Maintenance Management

RTDA is in the process of developing a Bridge Management System (BMS). It has adopted AASHTO PONTIS bridge condition rating system. However, RTDA's BMS is still at its infancy and it has not been documented.

5.3.3.4 Road Maintenance in Tanzania

Organisation of Road Maintenance in Tanzania

Under section 3(1) of the Executive Agencies Act (Cap 245) Tanzania established the Tanzania National Roads Agency (TANROADS) as an Executive Agency under the Ministry of Works, and the Agency came into operation in July, 2000. The Agency is responsible for the maintenance and development of the trunk and regional road network in Tanzania Mainland.

For Tanzania Zanzibar, a Road Authority will be established soon. Currently, the Ministry of Infrastructure and Communication is responsible for the maintenance and development of the road network in Zanzibar.

Vision of TANROADS

To have sustainable, safe and environmentally friendly all weather Trunk and Regional road network to support the social economic development of Tanzania.

Mission of TANROADS

To develop, maintain and manage the Trunk and Regional Roads Network efficiently and in a cost effective, safe and environmentally sustainable manner consistent with the Poverty Reduction Strategy and other Government policies through a competent and well motivated workforce.

Country Road Network

Out of the total classified road network in Tanzania Mainland which is estimated to be 86,472 km, the Ministry of Works through TANROADS is managing the National road network of about 33,012 km. Out of these 12,786 km are Trunk and 20,226 km are Regional roads. The remaining network of about 53,460 km comprises of Urban, District and Feeder Roads. These are managed by the Prime Minister's Office Regional Administration and Local Government (PMO-RALG).

The National Road Network

The National Road Network consists of:

- Trunk roads 12,786 km out of which 5,166 km are paved and 7,620 km are unpaved.
- Regional Roads: 20,226 km out of which 660 km are paved and 19,466 km are unpaved.

Zanzibar road network comprises a total of around 1,100 km of paved and unpaved roads.

Road Maintenance Management System

In 2001, Tanzania developed a Road Database system, known as Road Mentor. This system was implemented in 2002 in all regions as well as at the TANROADS Headquarters (HQ). From 2005 Road Mentor was developed into fully-fledged road maintenance system (RMMS) which focuses on supporting maintenance programming and works implementation in the regions and on data consolidation and planning at HQ level. The RMMS integrates a number of modules each designed to assist in specific decision–making process. The modules include:

- Routine and Recurrent Maintenance Module
- Periodic Maintenance Module
- Budget Split Module
- Contract Monitoring Module
- Budget Summaries Module
- HDM-4 Export Module

In addition, the system integrates with a Geographical Information System (GIS). The RMMS is capable of estimating the needs for routine and recurrent maintenance programmes as well as periodic maintenance and spot improvement for a period of one year. A Maintenance Priority Index (MPI) is used to prioritise Routine and Recurrent Maintenance activities. The MPI concept is based on Traffic level, importance of a road element and Maintenance Level.

The budget split module consolidates the needs analyses carried out by each region and prioritises them under a given budget constraint. On the other hand, the budget summaries module is used to generate and produce the needed information for the performance agreement between TANROADS, the Road Fund and the Ministry of Works.

If the multi-year rolling maintenance programmes are required, the HDM-4 export module is used to prepare road network files for further analysis in HDM-4. Furthermore, the contract management module is used to prepare and monitor road maintenance contracts. In the process, several manuals were developed to guide the implementation of the RMMS. These manuals include:

Data Collection Manual

The data collection manual provides a step-by-step guide to the collection of highway survey data for input into Road Mentor. The manual covers inventory survey for both paved and unpaved roads, unpaved roads condition survey and paved roads condition survey at network level.

Road Maintenance Supervision Manual

This manual was developed to guide the road maintenance contract supervision activities. The manual facilitates efficient and effective maintenance supervision by harmonizing supervision procedures and formats. These maintenance supervision activities may be carried out in-house by TANROADS Regional Offices personnel or outsourced to consultants. The manual covers the organisation of supervision, supervision means and main duties, quality control.

Furthermore, the manual specifies the forms and formats used in supervision as well as supervision checklists by maintenance activities.

Standard Specifications for Road Maintenance

The specifications provided here are read in conjunction with General Condition of Contract. The specifications clearly stipulate the ultimate criteria for acceptance or rejection of the works. However, they may be supplemented by guidelines issued by the Engineer from time to time

Road Maintenance Handbook Volumes 1 & 2

The Road Maintenance Handbook was developed with the aim to provide standard methodologies and tools for engineers, inspectors and technicians involved in road maintenance. The manual covers aspects of inspection and evaluation methods in volume I while volume 2 covers the execution methods.

Unpaved Road Network Condition Survey

Network level condition data for unpaved roads is collected during a drive over survey recording data for each one-kilometre sub-link.

The manual does not specify the frequency of surveys but it indicates the likely frequency to be between 1 and 3 years. Between 60 km and 90 km of survey can be achieved in a day, depending on survey routes and severity of defects. An operator on foot or cycle can of course, apply them, although output would be considerably reduced. The techniques used are all observational, i.e. no measurements are required, and require scores to be entered from a menu of options for each particular item. The content of the surveys takes into account the key performance factors in the provision and maintenance of unpaved roads.

Overall condition and Maintenance standards for unpaved roads

To provide an overall measure condition and maintenance standards for unpaved roads the intervention levels presented in Table 8 are specified.

Table 8: Intervention Levels for Unpaved Roads Maintenance in Tanzania

S/N	CONDITION	DESCRIPTION	INTERVENTION
1	Very Good	Shape condition of the surface in the 'as built condition. IRI less than 4 m/km	Routine Maintenance
2	Good	Positive camber or crossfall with no ponding of water, with low frequency of defects of low severity. The camber or crossfall will usually be greater than 4%.	Light grading capable of maintaining surface condition). IRI 4 – 6 m/km
3	Fair	Camber or crossfall at minimum required to shed water. Insignificant ponding of water with low frequency of defects with medium severity, or medium frequency of defects with low severity, IRI 6 to 9 m/km	Light grading capable of restoring surface condition unless extensive potholing and concave shape exists, otherwise heavy grading required to restore surface condition.
4	Poor	Camber or crossfall insufficient to shed water and water ponding in ruts or areas of concave shape up to 150 mm deep. Medium frequency of defects with low severity or high frequency of defects with medium severity, IRI 9 – 15 m/km	Reprocessing suitable under most conditions, otherwise light or heavy reshaping required
5	Very Poor	Substantial loss of camber or crossfall and water ponding in ruts or areas of concave shape in excess of 150 –300 mm. High frequency of defects with high severity, IRI greater than 15m/km	Light or heavy reshaping essential to restore shape

Paved Road Network Condition Survey

The assessment of the condition of the pavement is conducted from a specially prepared condition data collection vehicle. This data collection vehicle is operated by a driver and two specially trained observers/recorders. One observer is responsible for the automatic measurement of roughness and the recording of assessed defects using the keypad connected to the ROMDAS system. The second observer is also responsible for recording the visual assessment of the road condition and the legible recording of this information on the forms provided. The driver of the vehicle will be responsible for driving the vehicle at a fixed speed of 30 kph in the lane wheel-path. The driver is additionally and specifically responsible for the safe conduct of the survey and should be fully aware of prevailing traffic conditions at all times. The vehicle is fitted with an amber rotating beacon which will be activated when surveying.

Overall condition and Maintenance Standards for Paved Roads

To provide an overall measure of pavement condition and maintenance standards for paved roads the intervention levels presented in Table 9 are specified.

Table 9: Intervention Levels for Paved Road Maintenance in Tanzania

S/N	CONDITION	DESCRIPTION	INTERVENTION
1	Very Good	No visible defects i.e. the running surface is in the 'as built' condition, IRI less that 2.5 m/km	Routine Maintenance
2	Good	Low frequency of defects of low severity, Containing few or no potholes or patches or areas of visible cracking. IRI less than 3.5 m/km	Routine Maintenance
3	Fair	Low frequency of defects with medium severity, or medium frequency of defects of low severity. Occasional potholes and surface patches (5 per km), visible cracking and/or ruts of low severity (25mm) affect less than 10 percent of the length. IRI less than 5m/km	Localised repairs and resealing required to restore or hold surface condition
4	Poor	Medium frequency of defects with high severity, or high frequency of defects with medium severity. Frequent potholes and/or patches (5-20 per km) with extensive visible cracking and/or ruts of high severity affecting up to 20 per cent of the length, IRI less than 7 m/km.	Extensive structural patching, with or without shape correction and resealing or structural overlay required.
5	Very Poor	High frequency of defects with high severity. Extensive potholes and patches (> 20/km) and/or cracking and severe rutting affecting greater than 20 per cent of sub-link length. IRI greater than 7 m/km.	Reconstruction or extensive structural patching, shape correction and reseal; essential

Multiple Criteria Analysis Data Collection Manual

This manual provides a step-by-step guide to the collection of the data needed for the Multiple Criteria Analysis (MCA) used in Road Mentor for the ranking of periodic maintenance and needs for spot improvements. The manual describes the data specifically needed for the MCA. The data collection is carried out by combining desk studies with field interviews, as indicated for each data item to be collected. The following data needs are collected as presented and explained in Table 10.

- Social services
- Tourism
- Population
- Productivity
- Connectivity

The survey is carried out for each link (i.e. each road is recorded on separate forms). A record is made for every link. Table 10 presents the data needs for Multiple Criteria Analysis in Tanzania.

Table 10: Data Needs for Multiple Criteria Analysis in Tanzania

Parameter	**Description**	**Record**		**Likely Data Sources**
Social Services	The surveyor register the total number of social services within the *catchment Zone* (5 km on either side of the road)	Total number of services (hospitals, schools, clinics etc.)		Interview with regional and district administrative representatives and, if necessary, interviews with inhabitants along the road.
Tourism	The surveyor evaluates the road's importance in relation to tourist centres. Only the tourist centres directly related to the road link, should be included in the survey.	H	If the main purpose of the link is to provide access to tourist centres	Interview with regional and district administrative representatives.
		HI	Not used	
		ME	If the link is providing access to tourist centres but not specifically constructed to serve such centres	
		LO	Not used	
		VL	If the link is not directly providing access to tourist centres	
Population	The surveyor records the number of persons living within the *catchment zone* (5 km on either side of the road)	Total number of persons		Regional and district offices would most likely have the information.
Production Centres	The surveyor evaluates the road's importance in relation to production centres. Only the production centres directly related to the road link, should be included in the survey.	VH	If the main purpose of the link is to provide access to production centres	Interview with regional and district administrative representatives. Centres would include mines, forestry production, lager farms, factories etc.
		HI	Not used	
		ME	If the link is providing access to production centres but not specifically constructed to serve such centres	
		LO	Not used	
		VL	If the link is not directly providing access to production centres	
Network Connectivity	The surveyor evaluates the road's importance in relation to providing network	VH	The link is linking two Trunk roads (links)	Maps, combined with interview of regional offices, would provide the necessary
		HI	The link is linking a Trunk road (link)	

	connectivity		and a Regional road (link) and/or the link is the only connection between important centres and the main road network (no alternative routes)	information
		ME	The link is linking two Regional roads (links) or linking a Trunk road (link) with road (link) of a lower road class	
		LO	The link is linking a Regional road (link) with road (link) of a lower road class	
		VL	Not used	
Notes:	VH = Very High, HI = High, ME = Medium, LO = Low, VL = Very Low Catchment Zone; see section below			

It has to be mentioned here that the manual does not clearly indicate how the collected MCA data is used to prioritize the maintenance programme.

Financing of Road Maintenance

Tanzania was one of the first countries to adopt the Road Management Initiative (RMI) framework to reform the road management and maintenance systems. With a view to addressing the classified road maintenance needs, the Parliament of the Republic of Tanzania enacted the Roads (Amendment) Tolls No. 2 Act in July, 1985. The Funds were to be realized from collection of tolls and treated as part of the normal government taxes. The first generation of road sector reforms in Tanzania were incorporated into the design of the first road sector recovery program (Integrated Road Project—IRP, 1990). Two Road Funds were established under two separate declarations by the Minister of Finance—the first “Declaration to establish a special Road Fund” was made in August 1991, with the objective to finance maintenance and rehabilitation costs for the regional core network; and the second “Declaration to establish the local government Road Fund” was made in August 1992, with the objective to finance urban and district roads. However, the Funds were established as part of an administrative procedure and the declarations had no legal force or liability for compliance.

In an attempt to give the Fund some legal force and secure stable financing for road maintenance, the Parliament of the United Republic of Tanzania enacted the Roads Tolls Act in December 1998, establishing the Road Fund (RF) and the RF Board (RFB). The Act ring fenced 90 percent of the funds for road maintenance and no more than 10 percent for road development, upgrading and for local costs of compensation and counterpart funds for projects financed by donors.

Road Fund revenues is collected at source by the Tanzania Revenue Authority (TRA) and passed on to the Ministry of finance which deposits it to the respective Road Fund account.

The distribution is done in the following proportions:

- 63% TANROADS

- 7% Ministry of Infrastructure Development
- 30% PMORALG (1% HQ and 29 Local Authorities)

On the other hand the Government of Zanzibar established the Road Fund Board by The Zanzibar Roads Fund Act No. 2 of 2001 and its amendments of Act No.13 of 2008.

The said Act empowers the BOARD to enter into performance agreement with Ministry of Infrastructure and Communication (MoIC) for road maintenance activities in Unguja and Pemba Islands. In addition, the Act requires the BOARD to institute sound monitoring system of the disbursement of funds to MoIC to ensure the fund is only disbursed for the purpose and set out objectives. The monitoring system is done through technical and financial audits.

The source of revenue for Zanzibar Road Fund is the Road Development Levy imposed on fuel (petrol and diesel).

BRIDGES (Bridge Maintenance Management System)

Handbook for Bridge Inventory

This handbook provides key information about bridges in the entire road network including their location and methods of construction. The handbook generally explains a typical bridge and its components. It also describes different types of bridges and culverts. Most importantly, the handbook explains in some sufficient detail the required information to be recorded and the procedure of collecting the same. Further, the manual covers safety precautions as well as necessary human resources required for the inventory of bridges.

Handbook for Bridge Inspection

This handbook forms the basis for systematic and uniform approach to bridge inspection. The handbook reviews different elements of a bridge including the most common construction materials. It proceeds to cover various types of inspection, inspection plan and inspection programme. The manual also gives a list of damage and defects and their causes along with the assessment procedure. Material investigations and their testing methods are also covered. Finally, the manual provides the inspection record keeping methods.

Assessment of bridge condition gives priority to the defects which affect the load carrying capacity of the bridge followed by those defects affecting traffic safety. The third priority is given to those defects that affect maintenance cost and, finally, defects that affect aesthetics are considered last in the prioritization of bridge maintenance activities.

Handbook for Bridge Maintenance

This handbook covers aspects of planning, prioritisation, execution and quality control. It also deals with time and cost control of bridge maintenance activities. Activity and resource scheduling are covered. Furthermore, the handbook proceeds to provide guidelines of the work procedures and how measurements and payments are made. The handbook also includes safety precautions during repairs and maintenance. TANROADS have gone further to develop a computerised Bridge Maintenance System called TANBRIDGEMAN.

5.3.3.5 Road Maintenance in Uganda

There are four classes of roads in Uganda. These include national roads of 21,000 kms under Uganda National Roads Authority (UNRA). Under District Urban Community Access (DUCA) there are three classes of roads. District roads, which are 22,500 kms, are under the respective district authorities. Then there are urban roads in the major big towns including Kampala. These are under the respective urban authorities. The other class of roads is the community roads at community level which cover about 30,000kms.

In the 2005/06 financial year, the function of Road Maintenance was transferred from the Ministry of Works and Transport to the Road Agency Formation Unit (RAFU). RAFU, a semiautonomous body, was subsequently transformed into statutory authority namely the Uganda National Road Agency (UNRA), by an act of Parliament which became fully operational in 2008.

UNRA was established with the responsibility of managing the National Roads Network including its maintenance and development.

UNRA Mandate

The Authority is mandated to:

- Develop and maintain the national roads network totaling to about 20,000 km, managing ferries linking the national roads network and controlling axle overloading.
- Render advisory services to Government and for related matters concerning National Roads Network, among others.

UNRA Vision

To operate a safe, efficient and well-developed national roads network

UNRA Mission

To develop and maintain a national roads network that is responsive to the economic development needs of Uganda, to the safety of all road users, and to the environmental sustainability of the national roads corridors.

UNRA Goals

- Optimize the quality, timeliness and cost effectiveness of road works.
- Guarantee all year round safe and efficient movement of people and goods throughout the country.

Road Maintenance Management System

Uganda developed Road Maintenance Management Guidelines in 1993. These provide guidelines of organising and packaging the works, supervising monitoring and measuring of activities, payments and accounting, work method statements, labour, intensive and mechanised based maintenance for different tasks of maintenance

A Road Maintenance Management Manual was developed in 2004 and subsequently reviewed in 2010. This manual focuses on management issues for road maintenance measures. The manual introduces routine maintenance prioritisation system using a road condition rating system. The Level of Service (LOS) concept using factors like traffic volume, road user costs and maintenance costs has been introduced.

However, the audit which was carried out in 2010 by the Office of Auditor General (OAG) in Uganda established that UNRA did not have a documented strategic plan for periodic maintenance activities. The audit documents that some roads whose life span could have been prolonged through periodic maintenance are deteriorating to the levels that require major rehabilitation to make them motorable.

In addition, the audit found that:

- Some stations do not have surveillance reports which establish causes of road failures.

- The grass had grown beyond tolerable height at various locations on the all 2300 km roads inspected.
- A number of roads inspected have edge of mat failure
- Not all culverts on the 35 roads are suitably maintained
- Several sections of the gravel roads were in poor conditions with terrible rutting and gullies along the drive way
- Not all the national roads handling commercial vehicles have permanent or mobile weighbridges.
- Only two of the four weighbridges are offloading excess load on identified vehicles as statutorily required.
- The fines and penalties being given to offenders are neither consistently applied nor deterrent.

Bridge Maintenance Management

The audit also established that of the 217 bridges UNRA had no information on the status of 109 bridges (i.e.50% of the bridges). It was established that 58 (27%) bridges are being maintained while 50 (23 %) bridges are not being maintained.

Currently, the Ministry of Works and Transport is in the process of procurement for a consultant to develop a Bridge Maintenance System.

Road Maintenance Financing

The Uganda Road Fund (URF) was established by an Act of Parliament to finance the routine and periodic maintenance of public roads, facilitate delivery of road maintenance services and other related matters.

Under the Act, the purpose of the URF is to finance maintenance of all the roads mentioned earlier which amounts to 80,000 km. The source of funding is road user charges as stipulated under section 21 of the URF Act. These include fuel levies, international transit fees collected from foreign vehicles entering the country, road licences, axle load fines, bridge tolls and road tolls, and weight distance charges. This applies to the heavy vehicles operating internally. The other source of funding is under the Traffic and Road Safety Act which the traffic police are collecting as express penalties.

Preliminary findings

Review of institutional arrangement indicates that all partner states have put in place similar institutions for carrying out road and bridge maintenance. In each partner state there is a ministry that deals with road and bridge maintenance at policy and coordination level.

Each partner state has established an executive agency that is responsible for day to day management of road and bridge maintenance activities. A slight difference can be observed concerning the responsibilities of the established agencies. While agencies established in 4 partner states are responsible for roads only, the Rwanda Transport Development Authority (RTDA) also deals with other modes of transport.

In addition, each partner state has established a body that deals with funding of road and bridge maintenance. Differences in terms of sources of funds and distribution criteria between different classes of roads are observed from one country to another.

However, it has been observed that the road maintenance policies in all partner states do not reflect road user needs in terms of defining clearly the level of service and economic benefits to be provided to road users. Furthermore, in all partner sates, there is no mention of the need to have similar level of service across all EAC partner states.

Detailed review of institutional arrangements in the EAC partner states is presented in Chapter 7 of this project.

It has been observed that EAC partner states are at different levels of Road Maintenance Management System (RMMS) and Bridge Management System (BMS) development and utilisation. In addition, countries which have attempted to develop these systems use diverse principles and criteria in terms of data collection methods, maintenance standards, determination of road and bridge maintenance plans and programmes and execution of road and bridge maintenance activities. This results into different road and bridge condition and hence different level of service and economic benefits which road users get from one partner state to another.

Insofar as implementation phase is concerned, all EAC Partner states are phasing out the force account methods. However road maintenance contracts that have been introduced in most countries are based on a schedule of unit rates and measured quantities. In addition, Tanzania has introduced performance based contracts which have not performed very well so far. Furthermore, we are informed that Kenya is developing standards for the performance based contracts.

5.4 Potential Areas for Harmonisation and Improvement

The review has identified the following harmonization and improvement issues:

5.4.1 Policy Framework

A harmonised policy framework shall set uniform objectives in terms of the level of service which the road users should receive across the EAC partner states. This may include specification of level of comfort and economic benefits.

Recommendation 1*: We propose that in all partner states the road and bridge maintenance policy should reflect the customer (road users) needs. The policy should include, inter alia, the following goals:*

- *Support the socio-economic goals at national and EAC level.*
- *Provide a minimum Level of service to the road users across EAC partner states*
- *Provide Safe roads*
- *Minimise the sum of road agency and user costs*
- *Minimise damage to the environment*

5.4.2 Data collection Procedures

It has already been noted that a highway maintenance management information system lies at the heart of the management cycle. However, data collection is very expensive. In this regard, the cost of data acquisition and keeping them current is likely to be the most expensive aspect of implementing and operating a highway management system. It is therefore essential that appropriate data design is undertaken to ensure that the data to be collected for a particular purpose within the highway management cycle is relevant, appropriate and reliable within the available resources i.e. affordability of the responsible agency. Three guiding principles should be considered when deciding which data to collect. These principles are:

- Collect only the data needed;
- Collect data at the lowest level of detail sufficient to make appropriate decisions; and,
- Collect data only when they are needed.

The proposed data collection regime is presented in Table 11.

Table 11: Proposed Data Collection Regime for Road Maintenance

S/N	Assessment/ Purpose	Data Item	Frequency	Responsibility
1	Road Inventory	Road Network Physical Elements	Once-off exercise (updated when there are changes-verification after 5 years)	Executive Agency
2	Rideability (Serviceability)	Roughness	Annually	Executive Agency
3	Functional Evaluation	Surface Distress	Annually	Executive Agency
4	Structural Capacity	Mechanical Properties	Every 3 years	Executive Agency
5	Usage (Traffic)	Traffic Volume	Annually	Executive Agency
		Traffic Loading	Every 3 years	Executive Agency

Recommendation 2*: We recommend that surface condition and structural evaluation should be done through roughness measurements using Vehicle Mounted Bump Integrator (VMBI) and Dynamic Cone Penetrometer (DCP) measurements, respectively. Previous studies have shown that these methods provide relatively accurate results at reasonable cost. In so far as traffic is concerned, it is proposed that manual classified traffic counts and axle load measurements should be carried out regularly.*

5.4.3 Road Surface Riding Quality

The harmonization of road maintenance standards should be able to address the cross border riding quality to fulfill the functional requirement of traffic and entire travelling public. This entails the adoption of uniform and consistent intervention levels for road surface maintenance within EAC partner states.

Recommendation 3*: We recommend that road surface intervention levels be based on economic analysis and aiming at keeping the road network in good condition with specified target roughness of not more than 4.0 m/km IRI. Studies have shown that beyond this roughness level the riding starts becoming uncomfortable at operating speeds of around 80 km/hr.*

5.4.4 Pavement Structural Capacity

It is a fact that roads deteriorate with time. The harmonization of road design standards will result in having roads with same initial load carrying capacity across borders. However, the amount and quality of maintenance the roads receive should also aim at keeping their load carrying capacity consistent from one road section to the other and from one country to another. This requires uniform and consistent intervention levels for structural maintenance of the roads.

Recommendation 4*: We propose that assessment of structural capacity should be based on maintaining strong pavements with specified* ***minimum*** *modified structural number of 3.0. This is a critical level of structural capacity for roads carrying heavy traffic.*

5.4.5 Principles of Road Maintenance Management Systems

The harmonization process should result into provision of economic and technical efficiency across the region. It is therefore essential that the basis for decision making on investment choices should be consistent and uniform across all EAC countries. It is suggested here that each partner state should develop or adapt a Road Maintenance Management System (RMMS) which must be able to select cost-effective maintenance programmes and optimise maintenance expenditures under budget constraints.

***Recommendation 5**: We recommend that the Road Maintenance Management Systems (RMMS) should be based on economic analysis (Life Cycle Cost and Benefit Analysis) aiming at minimising the sum of agency and road user cost while maintaining a certain minimum level of service to the road users. Experience has shown that economic based methods result into improved road network condition, over time, at a relatively low budget. HDM4 can be a very useful tool as support software for Road Maintenance Management Systems.* In this regard, we advise Partner States to evaluate, for possible adaptation, the TANROADS Road Maintenance Management System which is at a very advanced stage.

5.4.6 Bridge Maintenance System

Bridges are part of road transport infrastructure. They therefore need to be properly maintained for effective and smooth movement of goods and people through the road network. However, the review carried out in this study indicates that most EAC partner states have not done much in development and utilization of Bridge Maintenance Systems. At the moment Tanzania is relatively advanced in this aspect. It has developed comprehensive bridge inventory, inspection and maintenance procedures. In prioritization of bridge maintenance activities, the highest priority is given to the defects that affect bridge load carrying capacity followed by defects that affect traffic safety. The defects that affect maintenance cost are given third priority and finally those defects that affect aesthetics are considered last.

5.4.7 Bridge Maintenance Programme

The maintenance programme should be designed to include prevention of deterioration and damage, prompt detection of deficiencies and early accomplishment of maintenance and repairs to prevent interruptions of operations or limitations/restriction of bridge use. The bridge maintenance programme consists of the following elements:

a) **Inspection**: Continuous, rigorous inspections are necessary for effective maintenance programme.
b) **Maintenance**: This is the recurrent day-to-day periodic or scheduled work that is required to preserve or restore a bridge to such a condition that it can be effectively utilized for its designed purpose
    (i) Routine maintenance: includes adjusting bearings, complete repainting, repairing potholes, filling cracks and sealing concrete.
    (ii) Major maintenance approaches rehabilitation in that it might include the replacement of bearings, readjustment of forces such as cables, replacement of joints, fatigue crack repair, water way adjustment and other specialized activities not performed very often.

5.4.8 Data Collection for Bridge Maintenance

Effective support of bridge management decision making requires timely and quality data about bridge condition. The Data collection regime bridge maintenance is presented in Table 12.

Table 12: Proposed Data Collection Regime for Bridge Maintenance

S/N	Assessment/ Purpose		Data Item	Frequency	Responsibility
1	Bridge Inventory		Historical and physical information of each bridge	Once-off exercise (updated when there are changes- verification after 5 years)	Executive Agency
2	Inspection	Informal	Observation of any damage, and obvious abnormalities such as impact damage to superstructures, bridge supports or parapets, flood damage or insecure expansion plates, erosion, scour, change of river course, formation of islands in the riverbed etc.	Not fixed	Executive Agency
		General	Visual control and assessment of all bridge elements with no material investigation. It involves thorough check of all bridge elements, e.g. approaches, potholes and related damage, settlement of the pavement, erosion, excessive vegetation, any obstructions or missing signs, cracks, spalling deflections etc.	Annually	Executive Agency
		Major	Thorough visual inspection of the entire structure supplemented by detailed measurements and material investigations. These may include both destructive and non-destructive tests such as rebound hammer test, covermeter tests, taking core samples, steel samples where possible, soil samples, stone samples etc for laboratory testing	Once in 3 years	Executive Agency
		Special	Visual inspection combined with measurements, material investigations and determination of the remaining carrying capacity of the bridge	Not fixed – done to investigate damage discovered during major inspection that needs further investigation	Executive Agency

5.4.9 Damage Evaluation

5.4.9.1 Degree of Damage

The degree of damage is used to quantify specific damage by indicating the extent and severity of the damage. The Bridge Management System for Tanzania (BMST) gives four characters to indicate different levels of damage as follows:

1 Minor damage or defects that might require any remedial action within the next 10 years
2 Average or slight damage or defects that require remedial action within 3 – 10 years.
3 Serious damage or defects that remedial action within 1 – 3 years
4 Critical damage or defects that require immediate remedial action within 0 -1 year

The above characters enable the inspector to indicate the seriousness of the damage observed and the timing for their repair based on experience and professional judgement.

5.4.9.2 Consequences of Damage

These indicate the impact if the damage is not repaired in time. Again, the Bridge Management System for Tanzania considers four effects as follows:

C: Damage or defect that affects carrying capacity
T: Damage or defect that affects traffic safety
M: Damage or defect that affects maintenance cost
E: Damage or defect that affects environment/aesthetics

Assessment of damage is done by combining the degree and consequence of the damage and may result into final assessment as C1, C2, C3 or C4 for damages that affect the carrying capacity (C) of the bridge. Similarly, for damages that affect traffic safety (T) the assessment would be T1, T2, T3 or T4. On the other hand, the final assessment may be M1, M2, M3 or M4 and E1, E2, E3 or E4 for damages that influence maintenance (M) and environment (E), respectively.

Additionally, TANROADS has developed a computerized Bridge Maintenance System known as TANBRIDGEMAN.

Recommendation 6*: We propose here that,* rather than re-inventing the wheel, further evaluation of the Bridge Management System for Tanzania (BMST) should be done by individual EAC Partner States before the final decision of adapting the system is made.

5.4.10 Road Maintenance Methods (Implementation)

Generally, there are two methods of carrying out road maintenance. These are force account (in-house) and contracting out to the private sector. All EAC Partner States are phasing out the force account methods. However, in most countries, road maintenance contracts that have been introduced are based on a schedule of unit rates and measured quantities. Nonetheless, we noted that Uganda is going back to force account because of lack of local contractors' capacity to carry out maintenance works. It is our sincere hope that they will work on contractors' capacity building so that they can gradually go back to road maintenance contracts.

Many countries around the world have introduced new ways of contracting out road maintenance by using performance specified contracts. Experience has shown that these methods reduce maintenance costs and improve road conditions. Unfortunately, improper implementation of these schemes can produce adverse effects. In this regard, countries should allow ample time to go through a learning curve through pilot short term performance contracts before embarking fully on long term contracts. This would provide the opportunity for road administration to gain sufficient experience and improve their ability to prepare and monitor such contracts. In addition, sufficient time should be allowed for the local contractors to acquire sufficient capacity and qualifications to manage these new road maintenance contracts. Tanzania has introduced performance based contracts which have not performed very well so far. In addition, we are informed that Kenya is developing standards for the performance based contracts.

Recommendation 7*: We recommend that Partner States should continue with a traditional method of contracting out road maintenance based on a schedule of unit rates and measured quantities. However, a gradual approach of introducing performance based contracts should be adopted, starting with short-term pilot contracts with simple performance standards such as control of potholes and cracks and cleaning of the drainage system.*

5.4.11 Environmental Protection

As with road construction activities, road maintenance can contribute to soil erosion, disturbance of water flows, chemical pollution, traffic disruption, noise, dust, and other impacts on surrounding communities and natural life. In this regard, an appropriate level of analysis, which is project specific, should be undertaken to determine the likely effects of the maintenance works on the environment. Mitigation measures should be outlined for all environmental issues identified. This requirement should be included in the bid documents so that potential bidders are aware and can submit realistic and properly costed proposals.

Recommendation 8: *We recommend that bid documents and road maintenance contracts should include clauses* which require contractors to prepare environmental management plan outlining mitigation measures for all environmental issues identified. *Clauses for use in Road Maintenance activities should include, inter alia, work-site installations, preparation and supply of gravel materials in pit or quarry, material disposal after cleaning ditches and other drainage facilities, tree planting, minimizing occupational health and safety risks.*

5.4.12 Consistent Monitoring and Evaluation

Effective highway management requires that policies, standards and the effectiveness of programmes are reviewed on a regular basis. This can only be possible if a feedback mechanism done through monitoring and evaluation is in place.

Recommendation 9*: We recommend that each Partner State should develop a comprehensive,* harmonised and up to date database which would facilitate information sharing across EAC partner states*. This would assist in determining whether the programme was achieved according to specified standards and, most importantly, assess the effectiveness of the programme in terms of meeting the specified level of service and economic benefits to the road users. The results of this evaluation shall form a basis for review of policies and maintenance standards.*

5.5 Conclusions and Impacts of Harmonization and Improvements

5.5.1 Conclusions

In this chapter we have presented our findings concerning harmonisation of road and bridge maintenance standards. This was done after detailed review of existing road and bridge maintenance manuals and current practices in EAC partner states.

Furthermore, the analysis was based on best practice and the strategy advocated for effective road and bridge maintenance management. The strategy comprises a management cycle, within which there is a requirement for management information system. Components considered in this cycle include setting up a policy framework which clearly defines the aims and objectives of highway management. The policy framework is followed by defining standards for intervention on which the assessment of maintenance needs is based. Thereafter, the allocation of available resources is made in a systematic and equitable way in order to obtain the best value for money. This calls for the use of proper road and bridge maintenance managements systems. The implementation in terms of proper procurement procedures and supervision of works is another important component of the maintenance management cycle.

Finally, performance monitoring and evaluation to assess the achievements is essential for subsequent revision and improvement of the entire system.

Additionally, it is worth mentioning here that, in the course of this study, a participatory approach was adopted. In this regard, comments and contributions from experts and stakeholders workshops held in Nairobi, Bujumbura, Kigali, Kampala, Dar es Salaam and Zanzibar provided necessary inputs to this document. In addition, the task force review meeting provided valuable inputs towards this draft final report. Consequently, resulting potential areas for harmonisation and improvement along with corresponding recommendations have been presented. Of particular significance, rather than re-inventing the wheel, Partner States have been advised to carry out further evaluation of TANROADS Road Maintenance Management System (RMMS) and Bridge Management System for Tanzania (BMST) for possible improvement and adaptation. Our review revealed that these two systems are operational at a very advanced stage.

5.5.2 Impact of Harmonisation of Road and Bridge Maintenance Standards

As discussed earlier, road and bridge maintenance policies in all member states do not reflect road user needs in terms of defining clearly the level of service and economic benefits to be provided to road users. In this regard, the first step towards implementing the recommendations is to re-define the policy framework. This would require member states to commit themselves and spend time and resources to go through the process of reviewing the road and bridge maintenance policies.

In addition, member states have been advised to carry out detailed evaluation for subsequent improvement and adaptation of computerised road and bridge maintenance management systems. However, experience of implementing computerised systems suggests that not only are more staff required, but the required staff need skills that are not traditionally found within the implementing road agencies. In this case, road agencies in member states shall have to allocate more resources for staff recruitment, training and retraining. Other extra costs required for the implementation of the given recommendations include those for collection and updating of relevant data accompanied with procurement of required equipment. This might impact on other activities which are competing for the same resources.

Despite the above, benefits such as improved asset management, improved contract and cost control as well as acquisition of better management information are expected as a result of implementing the given recommendations. If implemented, the recommendations suggested will result into uniform and consistent level of service and economic benefits across EAC partner states.

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FINAL REPORT

ANNEX A 6

HARMONISATION OF ROAD TRAFFIC SIGNS

TABLE OF CONTENTS

LIST OF TABLES

GLOSSARY AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ATSSA	The American Traffic Safety Association
FFTSS	Full Fibreglass Traffic Signs System
GTS	Guide to Traffic Signing – Issued by MoID, Tanzania
ITE	Institute of Transportation Engineers
KTSR	The Kenya Traffic Act Traffic Signs Regulations
MUTCD	Manual on Uniform Traffic Control Devices
RTSM	SADC – Road Traffic Signs Manual
SADC	Southern African Development Community
TCD	Traffic Control Devices

EXECUTIVE SUMMARY

This chapter reports on the areas of harmonization and improvement of traffic signs in the EAC Partner States. Traffic signs include road signs, traffic signals (traffic lights), and road markings. They are used by road authorities to communicate with road users with view to regulate their behaviour, to guide, warn or inform them for their own safety and convenience on the public road networks.

International conventions provide a framework for traffic signs and each Partner State has adopted a set of traffic signs to meet their current needs through their traffic law and/or traffic signs manual. Traffic signs regulations and manuals keep evolving to meet the needs of the travelling public. The harmonization of traffic signs in the EAC is essential to the safety and convenience of the public travelling on the road networks whether for trade, tourism, education or other purposes.

The main recommendation of this chapter is to integrate the existing road traffic signs and road marking schedules of the member states and to adopt all-white pavement marking system. To overcome the problem of vandalism we recommend adoption of Full Fibreglass Traffic Signs System which has been demonstrated to be cheaper, durable, costs less in maintenance and have zero re-sale value in best practice counties. We recommend that each Partner State develop and publish its road traffic signs manual to facilitate the harmonization of the more technical aspects of road signing as provided in this chapter.

A prudent implementation of approved changes over a ten year period is recommended under budgetary constraints. The harmonized signing and design principles should be applied on all new and rehabilitation projects and in the course of maintenance of road signs. No special budget will be required if this approach is adopted. However, it is desirable to mobilize funds and implement posting speed limits on all paved rural roads and apply an all-white pavement marking to accelerate achievement of road safety benefits to quickly match practice with the provisions of the law.

Road Signs

Generally, there exists no big difference in the use of road signs in the Partner States. To ensure greater uniformity in the installation of road signs the following are recommended.

i. The guidelines on mounting, placement and sign plate sizes as a function of road design speed should be provided in the partner states road signs manuals. A harmonized regime is proposed in this chapter.
ii. Partner States regulatory and warning signs schedules are generally similar. Integrated regulatory and warning signs schedules (both permanent and temporary signs) are recommended for adoption by the Partner States.
iii. Guidance and information signs schedules in use across the Partner States tend to differ in colour code. A consistent colour code and a schedule for guidance and information signs are recommended in this chapter for adoption by the Partner States.
iv. Road authorities in Partner States should adopt management practices that will ensure consistent application and maintenance of road signs at all relevant locations at all times. In particular we recommend the adoption of Full Fibreglass Traffic Signs System (FFTSS) to overcome the persistent problem of vandalism of road signs made of metallic materials that have a ready market as scrap metal.

Road Markings

The use of pavement markings across the Partner States is generally similar with the exception of the use of solid yellow marked lines to separate traffic travelling in opposite directions in Kenya and Uganda as opposed to the use of solid white line in Burundi, Rwanda and Tanzania. In Tanzania a solid yellow line marking is used to mark the edge of the travelled carriageway. It is recommended to adopt all-white pavement marking system. This provides for greater uniformity and has economical and technical advantages since whole life cost of white line is cheaper and has better retro-reflectivity under similar circumstances to yellow lines. Since there are circumstances which may require installation of road studs for better visibility, Partner States are encouraged to adopt their use. In particular, road studs should be used throughout the regional routes to ensure durable and all weather visibility of centrelines.

Traffic Signals

There are no major differences in the regulations for operation of traffic signals in the Partner States The meaning of the lights and sequencing are generally consistent and road users can quickly adapt to the slight differences that exist. However, de-legalizing of traffic signals based on only the red and green light (two lights system) allowed under the laws of Burundi and Rwanda but not practiced is desirable. The installation of two sets of signal heads per approach, one set on the near side of the approach and the other set at the far side of the approach is desirable especially on arterial road junctions. It is of utmost importance that junctions with similar traffic conditions be controlled in a uniform manner in the interest of consistency and developing respect for traffic signs. Signals should therefore be installed where an engineering study confirms that their installation is warranted. The process should be guided by competent professional on the basis of current best practices preferably the guidance provided in SADC - Road Traffic Signs Manual. The practice of traffic police overriding control of traffic by light signals should be strongly discouraged as it is serious health hazard.

Traffic Signs for School Zones and for Pedestrians with Special Needs

The practice of developing school route plan, defining school zones and the necessary traffic signing around schools both in urban and rural areas is recommended for adoption by partner states as means of providing for safety and convenience of school children when walking to school within school zones. The needs of physically challenged road users should also be provided through installation of appropriate warning signs when warranted and road design. Installation of blinking lights at pedestrian crossings on roads with heavy traffic and on hazardous road locations is recommended.

Signing for Traffic Calming

The need for traffic calming across the Partner States is being met without the benefit of a systematic approach involving both the professionals and the public. Sometimes the traffic signing is not adequate and the measures are not visible to the road users. Harmonization of the planning, signing and design of traffic calming measures is recommended.

6 HARMONISATION OF ROAD TRAFFIC SIGNS

6.1 Introduction

6.1.1 Background

This chapter addresses the harmonisation of road signs, traffic signals and markings. For the purpose of this chapter traffic signs include road signs, traffic signals and markings; sometimes referred to as traffic control devices (TCD). The chapter presents a review of the legal traffic signs and recommended practices in the Partner States and identifies areas requiring harmonization.

6.1.2 Objective

The objective of this chapter is to make recommendations for the "Harmonisation of Road Signs, Traffic Signals and Markings" as spelt out in the ToR. A brief review of the situation in the partner states and justification of the recommendations are given where it was considered necessary.

6.1.3 Methods

In order to arrive at recommendations that are acceptable to the Partner States the following approach was adopted:

i. First a review of existing and proposed legislation and manuals within Partner States was carried out and compared with the best practices including the SADC Road traffic Signs Manual.

ii. On the basis of the findings of the review areas that needed harmonization or modernization were identified.

iii. These proposals were presented to experts and stakeholders in workshops carried out in all Partner States and Zanzibar and were extensively commented upon.

iv. The proposals were revised to take into account the comments from the experts and stakeholders from the Partner States and Zanzibar.

v. The findings were presented to the EAC Task Force which convened in Dar es Salaam during September 2011. The Task Force discussed and gave further comments and suggestions.

vi. The comments and suggestions were used to improve the chapter which was submitted as part of the draft final report.

vii. The Chapter was again presented to the meeting of partner state representatives which convened in Mwanza in May 2012. The recommendations from that meeting have been used to enrich the final report.

6.1.4 Outputs

The final report is the outcome of the process outlined above. The recommendations are presented for road signs, traffic signals and pavement markings. The issues of pedestrians with special needs including school children and the physically challenged as well as traffic calming and implementation framework were considered to be very important and accordingly recommendations are included in this report.

6.2 Road Signs

The following subsections summarize legal requirements for road signs guidance offered in current manuals and/or laws and recommend the way forward where harmonization is required.

6.2.1 Placement and Mounting of Road Signs

The following general advice on sign mounting and positioning is recommended though it might not be possible to follow the advice exactly due to site constraints:

- Check that the signs are clearly visible from the appropriate distance
- Check that there is no confusion about which road the sign refers to
- Check that signs do not obstruct the view of drivers, especially at junctions
- Check that the signs are not placed where they could be struck by vehicles

Placement

Signs on two-lane roads should be placed on the side of the road on which the traffic it is intended to control is using except on sharp bends where signs placement on the other side will make it more visible. On dual carriageway roads warning and regulatory signs should be installed in pairs, one on the left-hand and another on the median. The approach speed and the required visibility of traffic sign is related as shown in Table 1.

Table 1: Relationship between design speed and the required sign visibility distance

Approach speed (km/h)	Visibility distance to the sign (m)
≤ 60 km/h	60
80 km/h	80
≥ 100 km/hr	100 – 150

Source: Guide to Traffic Signs (GTS) Tanzania

The GTS further recommended that, if two signs are to be placed at the same place they should be spaced at least V metres apart (where V is the approach speed in km/h) to avoid obscurity and information overload.

In Rwanda (driving in the right) the requirement is to place signs on the right side of the road. The warning (danger) signs must be place 150 m to 200 m from the dangerous area. An additional panel should be used to indicate the distance to the dangerous location if it is not practical to place the sign at 150 to 200 m from it. This is a good practice.

Mounting: Heights and side Clearances

The recommended standard mounting height is 2100 mm from the lowest edge of the sign plate to the road surface. The law in Rwanda specifies a minimum height of 1.50 m and not more than 2.10 m above the ground (Article 92). When a clearance greater than 2.1 m from the ground is required, for example when a sign is placed within the path of pedestrian or cycle path, a clearance of 2.5 m is recommended.

Multiple Signs / Secondary Message Signs

The SADC-RTSM and the GTS recommend that a secondary sign to be mounted beneath the primary sign to which it refers, and when two or more warning signs are mounted on the same sign pole the sign at the top should refer to the nearest of the hazards. When appropriate it is better to place the secondary message signs on the same sign plate as the primary sign (stronger plate, less chance for vandalism).

Angle of the Sign Plate

The recommended sign plate angle in the RTSM is 930 to the direction of travel in order to avoid mirror-like glare when the sign is illuminated by vehicle headlights.

Harmonized placement and mounting requirements can be summarized as follows:

i. To adopt standard height of the bottom border of a sign at 2.10 m above the ground and to allow greater heights (say up to 2.5 m) if it is necessary to achieve visibility of the sign or when greater clearance (as when pedestrians and cyclists are likely to pass beneath the sign) is required.
ii. Mounting of secondary sign below the primary sign be accepted where it is not a standard practice.
iii. Adopt mounting angle (93^0) to the direction of travel.
iv. Required visibility of the sign to be provided (in metres) to be at least the design speed of the road in km/hr.

6.2.2 Regulatory Signs

Use and Classification

Regulatory signs are used to control the actions of road users in the interest of safety and efficient use of road space. Regulatory signs are classified as follows:

Group	**Function**
Control	Exercise control over the right of way of traffic
Command	Instruct drivers what to do
Prohibition	Instruct drivers what they must **not** do
Reservation	Reserve road space for specific vehicle types or road users

Temporary Signs

The colours of the Control Group of signs such as STOP sign should not change when the sign is used on a temporary basis, but the background of command, prohibition and reservation group sign when are used for temporary restrictions such as at roadworks should be yellow. The colour code for temporary regulatory signs in use can be summarized as follows:

	Command Group	**Prohibition Group**	**Reservation Group**
Border	Black	Red	Black
Background	Yellow	Yellow	Yellow
Symbol	Black	Black	Black

End of a Restriction

The GTS and the Traffic Act Regulations (Kenya) superimpose four black diagonal bars on a grey version of the restriction sign. This is the case also for the Uganda and Burundi (proposed) Highway Codes and Rwanda's law.

Placement and Sizes

In general regulatory sign should be sited on the side of the road the traffic is using at or near the point where the instruction applies. The Table below gives the recommended sizes for standard regulatory signs given in the Kenya Traffic Act (Regulations, Revised edition 2009) and the GTS.

Table 2: Recommended sizes for standard regulatory signs: Kenya and Tanzania

Approach Speed	GTS		KTSR	
	Circular sign (diameter) (mm)	Rectangular sign (H x W) (mm)	Circular sign (diameter) (mm)	Rectangular sign (H x W) (mm)
≤ 60 km/h	600 (900 on 3-lane roads and dual carriageways)	600 x 450	Should be not less than 600	Should be not less than 100 x 300
80 km/h	900	900 x 675		
≥ 100 km/h	1200	1200 x 900		
Overhead signs	1600	1600 x 1200		

The general principle of using dimensions corresponding with the design speed should be adopted. The guidance given in the GTS given above is therefore recommended for adoption across the Partner States.

The GTS recommends reduced signs for the following circumstances:

- Signs on traffic signal heads – 300 mm diameter
- Keep Left signs on traffic bollards – 300 mm diameter
- Parking signs – 450 mm diameter and 450 x 340 mm

Table 3: Recommended dimensions for regulatory road signs

Approach Speed	Recommended dimensions	
	Circular sign (diameter) (mm)	Rectangular sign (H x W) (mm)
≤ 60 km/h	600 (900 on 3-lane roads and dual carriageways)	600 x 450
80 km/h	900	900 x 675
≥ 100 km/h	1200	1200 x 900
Overhead signs	1600	1600 x 1200
Exceptions		
	Circular sign (diameter in mm)	Rectangular sign (H x W) (mm)
Signs on signal heads	300	
Keep-left signs on traffic bollards	300	
Parking signs	450	450 x 350

Code and Schedule of Regulatory Signs

The following (Tables 4 and 5) shape and colour code for permanent and temporary regulatory signs is recommended.

Table 4: Shape and colour code: permanent regulatory signs

Group	Shape	Border	Background	Symbol
Control	Varies	Red	Varies	Usually white
Command	Circular or Rectangular	White	Blue	White
Prohibit	Circular	Red	White	Black
Reservation	Rectangular	White	Blue	White
End of Restriction	Circular	Gray	White	Gray

Table 5: Shape and colour code: temporary regulatory signs

Group	Shape	Border	Background	Symbol
Control	Same as permanent sign	Same as permanent sign		
Command		Black	**YELLOW**	**BLACK**
Prohibit		Red		
Reservation		Black		
End of Restriction		Same as permanent sign		

Regulatory signs schedule

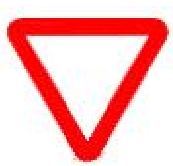
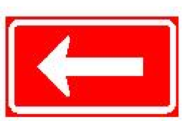
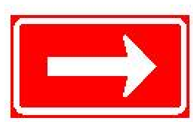
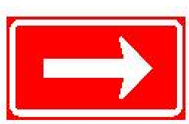
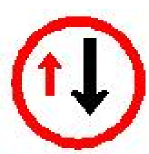
Recommended regulatory signs schedule is presented in Table 6 under the following subsections:

- **a)** Control signs
- **b)** Command signs
- **c)** Prohibition signs
- **d)** Reservation signs
- **e)** Supplementary signs
- **f)** Regulatory signs to be abandoned

We recommend the adoption of the SADC-RTSM numbering system to make it easy for users to make reference to the manual when further information or guidance is required. The numbering system is used throughout this chapter with minor extension when signs proposed for use in EAC are not included in the manual.

Table 6: Recommended regulatory signs schedule

(a) Control signs

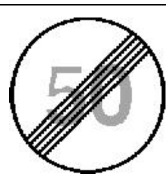
SN	Sign	Brief Description	SN	Sign	Brief Description
R1		Stop	R1.5		Stop/Go Control
R2		Yield / Give Way	R3		No Entry
R4.1		One –Way (Left)	R4.2		One –Way (Right)
R4.3		One –Way (Straight On)	R6		Give Way to Oncoming Traffic

(b) Command Signs

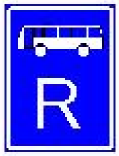
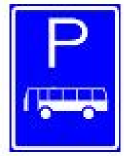
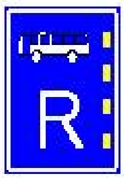
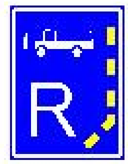
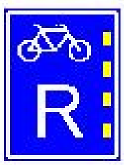
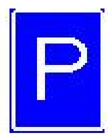
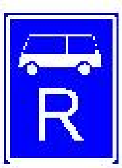
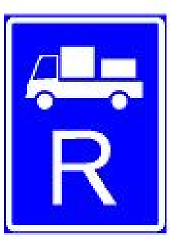
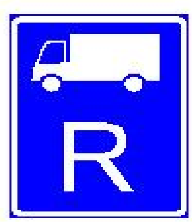
SN	*Sign*	**Brief Description**	*SN*	*Sign*	*Brief Description*
R101.		Minimum speed			
R103		Keep left	R104		Keep right
R105		Proceed left only	R1O6		Proceed right only
R107		Proceed straight only	R107.1		Proceed straight or right only
R107.2		Proceed straight or left only	R108		Turn left ahead
R109		Turn right Ahead	R110		Pedestrians only
R111		Cyclists only	**R121**		Buses Only
R131.1		Horses and riders only	**R137**		Keep left on the round about

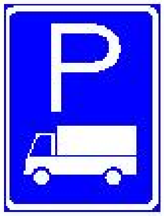
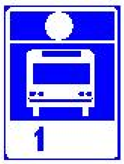
(c) Prohibition signs

SN	***Sign***	**Brief Description**	***SN***	***Sign***	***Brief Description***
R201		Speed Limit	R202		Mass Limit
R203		Axle Load Limit (single axle)	R204		Height Limit
R205		Length Limit	R206		No Excessive Noise
R209 .		No Left Turn Ahead	R210		No Right Turn Ahead
R211		No Left Turn	R212		No Right Turn
R213		No U – turn	R214		No – Overtaking for all vehicles
R215		Goods vehicles not allowed to overtake	R216		No parking
R217		No stopping	R217		No stopping during the indicated hours

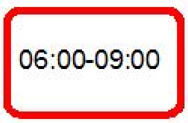
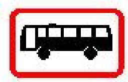
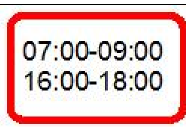
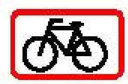
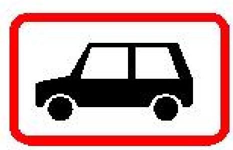
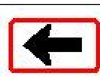
R218		No Entry for Pedestrians	R219		No Entry for Cyclists
R220		No Cyclists and Pedestrians	R222		No Entry for Motorcycles
R223		No entry for motor cars	R224		No Taxis
R227		No entry for Buses	R229		No entry for Goods Vehicles
R236		No entry for agricultural vehicles	R237		No entry for animal drawn Vehicles
R239		Width Limit	R243.		No entry for handcarts
R245		Axle load limit – tandem / group of axles)	R214-600		End of (no overtaking) Restriction - Example
R201		End of Speed Limit Restriction			

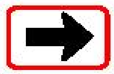
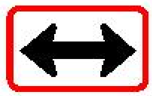
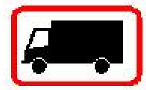
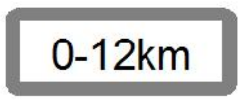
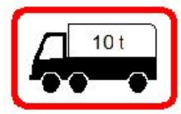
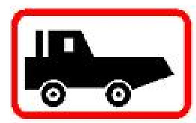
(d) Reservation signs

SN	***Sign***	**Brief Description**	***SN***	***Sign***	***Brief Description***
R301		Area reserved for Buses	R301-P		Bus Parking Area
R302		Bus Lane	R303		Start of Bus Lane
R304		Bicycle Lane	R305-P		Parking area
R306		Limited Duration Parking	R309-P		Parking Area for Taxis
R310		Area Reserved for Minibuses	R310-P		Parking Area for Minibuses
R312		Area Reserved for Use by Delivery Vehicles	R313		Area Reserved for Use by Goods Vehicles

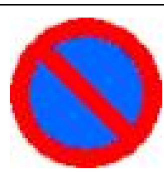
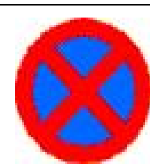
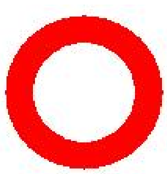
R313-P		Goods Vehicle Parking	R323-P		Disabled Persons Vehicle Parking
R325		Bus Stop (for one bus)	R360		Pedestrian (zebra) crossing

(e) Supplementary signs for use with regulatory signs

(R)501		One Time Period	(R)560		Bus (message on primary sign applied to buses)
(R)502		Two Time Periods	(R)561		Bicycle
(R)503		Two Time Periods	(R)562		Motorcycle
(R)506		Maximum stay during one time period	(R)563		Motor Car
(R)520		Reserved movement Left	(R)564		Taxi

(R)521		Reserved movement Right	(R)567		Delivery Vehicle
(R)522		Reserved movement in both direction	(R)568		Goods Vehicle
(R)535		Distance over which the limit applies	(R)569		Goods Vehicle Over Indicated Gross Vehicle Mass
			(R)570		Construction Vehicle

(f) Regulatory signs to be abandoned

SN	Sign	Brief Description	SN	Sign	Brief Description
1		No Parking Replaced by sign R216	2.		No Stopping Replaced by sign no R217
3.		Road Closed to all Vehicles in both Directions Replace with R3 and "road closed" supplementary plate	4.	All signs not conforming with the approved location, colour and shape code	• Relocate signs on inappropriate locations to recommended locations • Replace all signs that do not conform with the approved code

6.2.3 Warning Signs

Use and Classification

Warning signs are used to alert drivers to danger or potential danger ahead. Two groups of warning signs are in use:

Group	Description
Advance Warning Signs	Road Layout
	Movement
	Symbol
Hazard Marker Signs	Curve and object markers

Supplementary plates may be used underneath the primary sign.

Temporary Signs

The use of temporary warning signs is a common practice. For example the GTS provides for the use of warning signs in temporary situations. When they are used in such situations they should have yellow backgrounds instead of white background. The following signs have been documented in the manual for use in temporary situations:

Description	Background
Road works	Yellow
Loose stones	Yellow
Edge drop	Yellow
"Stop/Go" Control Ahead	Yellow
Road Crash	Yellow

Placement and Sizes

The following Table gives the sitting distances and sizes of standard warning signs for different traffic speeds as recommended in the GTS (based on SADC-RTSM). The Kenya Traffic Act Traffic Signs Regulations - KTSR) requires a warning signs length to be not less than 700 mm. The placement distances and plate sizes are recommended for adoption by the Partner States.

Table 7: Warning sign dimensions and distance from hazard location

Approach speed	Distance of sign from hazard (m)[1]	Advance warning signs – side length (mm)	Hazard marker signs[3] Width height			
			W401 W402	W405 W406	W407 W408	W409[2] W410
≤ 60 km/h	100	900	200 800	400 400	1200 400	2400 400
80 km/h	160	1200	250 1000	600 600	1800 600	3600 600
≥ 100 km/h	240	1500	300 1200	800 800	2400 800	4800 800

[1] Does not apply to hazard markers. [2] When used underneath a direction sign the sign should be adjusted to equal the length of the direction sign. [3]See the warning signs schedule.

Schedule of Warning Signs

Analysis of warning signs in use across the Partner States revealed the need to integrate them to form a common schedule presented in Table 8. The recommended colour and shape code can be summarized as follows:

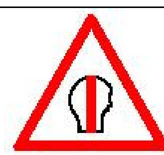
Shape and colour code: warning signs

- The shape of warning sign is triangular, the apex pointing up
- The border is red
- The background
    - White for permanent sign
    - Yellow for a temporary sign
- The symbol is black

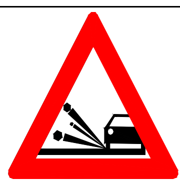
Table 8: Proposed Warning Signs Schedule

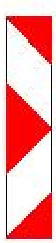
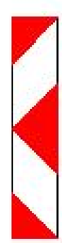
S/No.	Sign	Description	S/No.	Sign	Description
W102		Crossroad on Priority Road	**W104**		T-junction
W105		Skew T - Junction (Right)	**W106**		Skew T- Junction (Left)
W107		Side Road Junction (Left)	**W108**		Side Road Junction (Right)
W109		Staggered Junctions (Right-Left)	**W110**		Staggered Junctions (Left-Right)
W111		Sharp Junction (Half left)	**W112**		Sharp Junction (Left)
W113		Sharp Junction (Half Right)	**W114**		Sharp Junction (Right)

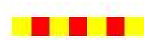
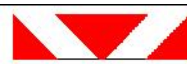
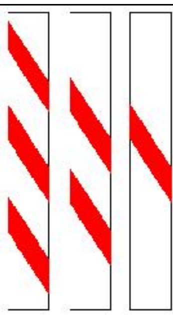
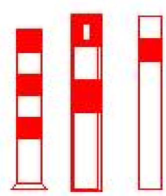
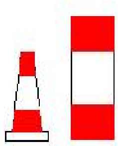
W115		Y-Junction	**W116**		End of Dual Roadway (To Right)
W117		End of Dual Roadway (Straight on)	**W118**		Start of Dual Roadway (Straight on)
W119		Start of Dual Roadway (To Left)	**W201**		Roundabout (For countries driving on the right the arrows point the other way)
W202		Gentle Curve (Right)	**W203**		Gentle Curve (Left)
W204		Sharp Curve (Right)	**W205**		Sharp Curve (Left)
W206		Hairpin Bend (Right)	**W207**		Hairpin Bend (Left)
W208		Winding Road (Right – Left)	**W209**		Winding Road (Left – Right)

W210		Combined Curves (Right – Left)	**W211**		Combined Curves (Left – Right)
W212.		Two – Way Traffic Ahead	**W213**		Two – Way Traffic Crossroad
W301		Traffic Signals Ahead	**W302**		Traffic Control "Stop" Ahead
W303		Traffic Control "Yield" Ahead	**W306**		Pedestrian Crossing
W306.1		Physically impairment or crossing ahead	**W306.2**		Blind pedestrians crossing ahead
W306.3		Pedestrians with Serious visual impairment crossing ahead	**W307**		Pedestrians ahead
W307.1		Deaf Pedestrians ahead	**W307.2**		Pedestrians with mental health conditions ahead

W308		Children			
W309		Cyclists	**W310**		Domestic Animals
W313		Wild Animals	**W318**		Railway Crossing
W319		Tunnel	**W320**		Height Restricted
W321		Length Restricted	**W322**		Steep Descent
W323		Steep Ascent	**W326**		Narrow Bridge
W327		One Vehicle Width Structure	**W328**		Road Narrows Both Sides

W329		Road Narrows From Right Side	**W330**		Road Narrows From Left Side
W331		Uneven Roadway	**W332**		Speed Humps
W333		Slippery Road .	**W334**		Falling Rocks (From Right)
W335		Falling Rocks (From Left)	**TW336**	 **TW 336**	Road Works
TW338		Loose Stones	**W339**		General Warning
TW340	 **TW 339**	Edge Drop	**TW343**		Stop/go Control Ahead

W348		Jetty Edge or River Bank or ferry berth	**W349**		Crosswinds
W350		Drift	**W351**		Low Flying Aircraft
W352		Agricultural vehicles	**TW353**	 **TW 353**	Road Crash
W355		Traffic Queue	**W360**		Width Restriction
W 365		Opening Bridge			
Series 400: Hazard markers; Use yellow background for temporary situation					
W 401		Danger Plate / Delineator Plate (Left)	W 402		Danger Plate / Delineator Plate (Right)

W403		Railway Crossing	**W404**		Railway Crossing (more than one track)
W405		Sharp Curve Chevron (single) (To the Right)	**W406**		Sharp Curve Chevron (single) (To the Left)
W407		Sharp Curve Chevron (Triple) (To the Right)	**W408**		Sharp Curve Chevron (Triple) (To the Left)
W409		T-junction Chevron	**TW411**		Barricade
W 413		Pass Either Side	**W 415**		Overhead Danger Plate
W416		Distance to hazard: Install marker with decreasing number of stripes as distance to hazard decreases	**D3**		Delineators
TD4		Traffic cones and drums			

6.2.4 Guidance Signs

Use and Classification

Guidance signs give road users information on how to find their way to their destination. The application of guidance signs is an underdeveloped area across the Partner States. Visitors have a hard time finding their way to their destination as the guidance offered through road signs is rather inadequate. We therefore recommend the guidance offered in the SADC - RTSM for adoption and adequate budget for providing guidance to road users.

The SADC - RTSM classified guidance signs into five groups listed below.

Group	Description	Proposed sign number prefix
Location	Place names, river names	GL
Direction	Direction signs before and at junctions	GD
Tourism	Direction to tourist attractions, services and facilities	GF
Location direction	Direction signs for minor, local destinations in urban areas	GDL
Diagrammatic	Signs warning of a change in road layout ahead (e.g. start and finish of climbing lane)	GS

Direction Signs – Types and Placement

The SADC-RTSM elaborates on guidance signs as shown below:

Types of Signs	Description
Advance direction signs (GD1, GD5, GD8)	They are used to give route information to drivers approaching a junction
Direction signs (GD2, GD4)	They are used to give route information at the junction and often point along the route referred to
Confirmation signs (GD3)	These list the destinations ahead, and are placed about 100 m after important junctions, and at intervals along the road.

Recommended placement of advance direction signs (GD1, GD5, GD8) in the GTS are as shown below:

Approach speed	Distance of sign from junction (m)
≤ 60 km/h	50 - 90
80 km/h	90 - 150
≥ 100 km/h	150 - 220

Sizes and design principles

To avoid confusion limit the number of destinations to four per sign. In one direction limit the number of destinations to two with the nearest one at the top. The size of the sign depends on the amount of text. Typically letter sizes depend on the approach speed as shown below.

Approach speed	**Typical situation**	**Capital letter height (mm)**	**Lower case letter height (mm)**
< 50 km/h	Very minor roads	112	80
50 km/h	Local urban and rural roads	140	100
60 km/h	Urban and rural 2-lane roads and urban dual carriageways	175	125
80 km/h	Good standard rural main roads	210	150
≥ 100 km/h	High standard rural main roads, and all signs mounted overhead	280	200

Shape and colour code for guidance signs

For guidance signs the code is summarized in the text box below.

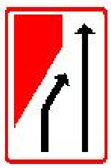
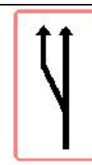
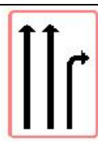
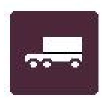
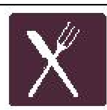
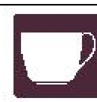
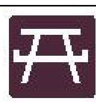
Shape and colour code: guidance signs

- Shape - rectangular
- Size – Depends on amount of text , and lettering size (design speed)

The Colour Code

Group	**Description**	**Border**	**Background**	**Symbol/letter**
Location	Place names	Blue	White	Black
Direction	Direction signs	White	Green	White
Tourism / services	Tourist attractions, Services and facilities	White	Gray	White
Local direction	Destination within urban areas	Gray	White	Black
Facilities	Facility type	No border	Gray	White
Diagrammatic	Warning of change in road layout	Red	White	Black

Table 9: Guidance signs basic schedule (examples)

S/No.	Sign	Description	S/No.	Sign	Description
GD		Direction signs before and at junctions	**GDL**		Direction signs for minor, local destinations in urban areas
GF		Directions to tourist attractions, services and facilities	**GL**		Place names, river names
GS 101		Left-hand lane ends	**GS 205**		Lane added on left hand side
GS 805		Lane pre-selection sign	**GFS B1-2**		Police symbol
GFS B1-3		Hospital symbol (with name)	**GFS B1-4.**		First aid Post
GFS B2-1		Filling station, workshop	**GFS B3-**		Caravans / other vehicle classes when applicable: Rest or/and service area
GFS B4-1		Restaurant	**GFS B4-2**		Refreshments
GFS B4-3		Take-away	**GFS B4-4**		Café / Rural shop
GFS B5-1		Parking	**GFS B5-2**		Toilets
GFS B5-7		Picnic site	**GFS B5-8**		Tourist information

GFS B5-9		Facility for the handicapped	**GFS B6-3**		Rest and service area

6.2.5 Information Signs

Use and classification

Information sign may take the form of a supplementary plate that provides additional information to that given on the primary sign or to indicate the presence of an information center or other facility.

Sizes and design principles

It is recommended to adopt the SADC-RTSM manual:

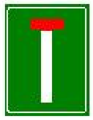
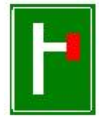
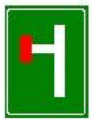
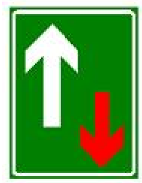
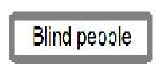
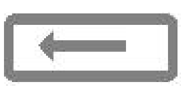
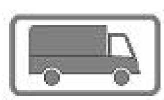
Supplementary signs	**Sizes/colour**
Lettering, symbol and border of supplementary plates	Black
Background	White
Width of Supplementary plates	Should match that of the primary sign
Capital letter height size	140 mm
Lower case letter height	100 mm
Cul-de-sac	600 mm (h) by 450 mm (w)
When the supplementary plate is to be used with a small sign such as a 600 mm diameter regulatory sign the text size may be reduced to 112 mm/80 mm	

Shape and colour code: information signs

The text box summarizes the shape and colour code.

- Shape: Rectangular
- Border colour : White for the main signs, black for the supplementary signs
- Background: Green when it is a primary sign and white if providing supplementary information to a primary sign
- Size: supplementary information sign must have the same width as the primary sign

Table 10: Proposed information signs basic schedule (examples)

S/No.	Sign	Description	S/No.	Sign	Description
IN4		No Through Road (Cul-de-sac)	**IN5**		No Through Road (Cul-de-sac) – Right Side
IN6		No Through Road (Cul-de-sac) –Left Side	**IN12**		Information Centre
IN16		Bus Stop Ahead	**IN20**		Oncoming vehicles are required to give way
IN11.1		Supplementary. Plate-Advisory speed	**IN11.2**		Supplementary. Plate-Distance “For”
IN11.3		Supplementary Plate - Distance “To”	**IN11.4**		Supplementary. Plate-Text Message
IN11.502 (**11.503**)		Arrow left (Arrow right)	**IN11.568**		Supplementary. Plate Vehicle Class

6.2.6 Signing at Road Works

The appropriate signing of road works is required in the laws of the Partner States. Generally regulatory and warning signs are adopted for road works by using backgrounds recommended for temporary signs. Advance warning signs and appropriate regulatory signs should be used. Markers may also be used as appropriate. Appropriate signs and markers are included in the regulatory and warning signs schedules.

Maintenance manuals and road works specifications should give detailed guidance and instructions to those supervising the works and provide mechanisms to ensure compliance.

6.2.7 Traffic Sign Materials and Manufacture

Current practice and guidelines

Across the Partner States there exists signs that are not equally visible day and night i.e. the sign plate is not made from retroreflective sheeting. Current guidelines to remedy the situation are presented, for example, in the GTS (Tanzania):

Sign Face Materials

All traffic sign, permanent and temporary, should be fully retroreflectorized by making the sign face from retroreflective sheeting. Symbols or letters coloured black are recommended to be made from black non-reflective overlay film.

Sign Plate

Use of aluminium alloy sheet is recommended due to its resistance to corrosion. Since aluminium alloy sheets are expensive cheaper alternatives such as galvanized steel, plastic-coated steel or steel treated with primer. The back of the sign plate be painted grey, and the sign manufacturer’s name and the date of manufacture be permanently marked on it.

Sign Support Frame and Fastenings

The sign plate should be fixed to a back support frame made of angle iron or aluminium sections. All frames, brackets, clips, rivets, nuts, bolts and washers are corrosion-proofed if not made of aluminium or stainless steel. The frames and brackets are recommended to be painted grey.

Sign Post

Sign posts are recommended to be made of galvanized steel tube set in concrete foundations.

However, due to the problem of vandalism we recommend changing the traffic signing system to full fibreglass traffic signs system. Some elaboration is provided below and further information can be found in the publications and other information resulting from research and development of the FFTSS in best practice countries. Should the partner states approve the recommendation, road authorities may wish to consult such publications to inform the preparations of their specifications.

Road Sign Specifications

Partner States include specifications for road signs in the specifications for road works. Specifications include requirements on manufacture and installation of road signs. Both standard and special specifications for particular projects should ensure that the required standards of visibility are met and the materials used have zero re-sale value.

Recommendations to Overcome Vandalism of Road Signs

Throughout the Partner States the problem of vandalism is a significant one given the ready market for the metallic materials used to construct road signs. Prosecution of the offenders, use of reinforced concrete instead of galvanized steel posts, perforating the steel plates to make them less useful, welding parts instead of merely bolting and placing signs higher up to make the less accessible are some of the measures that have been deployed to manage the problem. These measures have achieved varying degree of success but have not overcome the problem.

We recommend the use of road sign materials with zero re-sale value in order to overcome the problem of road signs vandalism. An approach adopted by best practice countries is the use of retroreflectorized plastics especially fibreglass sign posts and plates. The use of reinforced concrete – practiced in some Partner States – for the sign posts is to be discouraged since the post increases the probability of injury should a vehicle collide with it. The use of fibre glass for the posts have safety advantages since such posts yield on collision resulting in lower severity injury for the vehicle occupants.

6.2.8 Road Signs Management

In addition to the considerations for design, placement and materials for road signs there are additional issues that should be addressed by road authorities to ensure uniformity in control of traffic:

I. To make sure that every location that needs a traffic signs is provide with an appropriate sign. Road authorities across the Partner States should adopt management practices that will ensure consistent application and maintenance of road signs at all relevant locations at all times.
II. To ensure that where a sign is no longer needed it is removed,
III. To take precautions against vandalism of road signs, and
IV. To ensure that signs are properly maintained and are visible.

6.3 Pavement Markings

6.3.1 Purpose and Classification

Road markings are used to control, warn, or guide road users and may be classified as follows:

Road Markings	Types
Longitudinal Lines	Center lines, edge lines
Transverse lines	Stop lines, and give way lines
Others	Arrows, Symbols, Patterns and Words

6.3.2 Design Principles, Colour Code and Dimensions and Spacing of lines

Design Principles

Road markings should be considered in detail at the design stage of new or improved roads and junctions.

Requirements for road markings include:

- Good visibility by day and night
- Good skid resistance
- Durability

- Clarity of message
- Symbols and words should be elongated in the direction of traffic by a factor of three times whilst retaining the original width.

Meaning assigned to centre-line and lane markings on pavement surface:

- Broken longitudinal lines are permissive (overtaking / lane change permitted)
- Continuous solid longitudinal lines are restrictive (prohibit overtaking / lane-change)
- Double continuous solid longitudinal lines indicate maximum levels of restriction
- The width of the line is an indication of the degree of emphasis attached to the marking

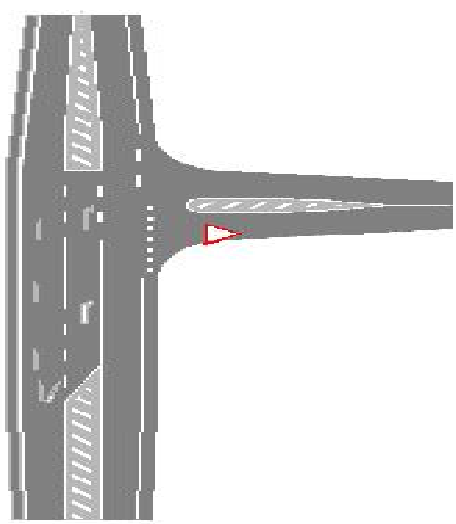
The no overtaking lines imposes a mandatory requirement that drivers keep the line to their right (driving on the left) unless when turning into or out of a side access. The lines are provided at places with limited sight distances or other hazardous conditions. On vertical and horizontal curves with sight distances less than those indicated below, no overtaking lines should be applied.

Traffic speed	Sight distance (m)
50 km/h	150
60 km/h	180
80 km/h	250
100 km/h	300
120 km/h	400

The recommended minimum length of overtaking line on vertical and horizontal curves is 150 m and the absolute recommended minimum distance between successive overtaking lines is 120 m. when used on the approach to junctions a minimum length of 24 m is recommended and may be increased to 60 m or more at higher traffic speeds. No overtaking lines are called “no crossing lines” when two of these lines are used one for each direction of traffic.

Marking at Junctions

Guidance on application of marking principles on un-signalized and signalized junctions is offered in RTSM – SADC. We recommend adoption of principles to achieve improvement of traffic operations at junctions across the Partner States. A typical example is given below.



Typical markings at a junction

Colour Code

Historically, different colours have been used in pavement markings to convey a specific message. For example, solid line in yellow colour is used in USA to separate traffic moving in opposite directions. This is also the practice in Kenya and Uganda. Tanzania and Rwanda use white line to indicate the centreline of the road. Solid line whether white or yellow on the centreline indicates that overtaking is prohibited. The yellow colour is used to mark the edge of the carriageway as is the practice in Tanzania. Solid yellow line indicates the pavement edge and to prohibit parking on the shoulder for a section so marked. The use of white and yellow lines varies across the partner states and need to be harmonized. For the reasons given below we recommend the adoption of all-white pavement marking.

Blue and red colour are also used for example red colour is used to indicate maximum degree of prohibition for example to prohibit blocking of fire engines. This is not common across the Partner States.

Dimensions and Spacing of Lines

Dimensions of lines and their spacing depend on the design speed of the road. Table 11 provides harmonized recommendations for the dimensions and spacing of the pavement markings.

Table 11: Recommended dimensions and spacing for pavement markings

SADC-RTSM Marking Number Ref.	Description	Dimension (mm)			
		Rural (speed> 60 km/h)		Urban (speed ≤ 60 km/h)	
		Width	Line gap	Width	Line gap
RTM1	Stop Line	500		300	
RTM2	Give Way Line	300	600-300	200	600-300
RTM4	Pedestrian (Zebra) Crossing	3000 4000	600-600	3000 4000	600-600
RM1	No Overtaking Line	150 100		150 100	
RM2	No Crossing Lines	2 x 150 (170 apart) 2 x 100 (120 apart)		2 x 150 (170 apart) 2 x 100 (120 apart)	
RM3	Channelizing Line	200 150 100		200 150 100	
RM4.1	Left Edge Line	150 100		100	
RM4.2	Right Edge Line	150 100		100	
RM5	Painted Island	Edge line: 150 100 Bar: 300 200	Bar width to space: 1:2	Edge line: 150 100 Bar: 200 150	Bar width to space: 1:2
RM6	Parking Bays	100		100	
RM9	Exclusive Use Lane Line	N/A		150	
RM10	Box Junction	N/A		Border: 200 Diagonals: 150	
RM11	Zigzag Zone Line	100	2000-150	100	2000-150
RM13	No Parking Line (24hr) No Parking Line (selective times)	100 100	 4000-2000	100 100	 4000-2000
WM2	Continuity Line	300 200	2000-2000 2000-4000	200	1500-1500 1500-3000
WM3	Dividing Line	150 100	4000-8000	150 100	3000-6000
WM12	Rumble Strips	4 x 150 (400 apart)		4 x 150 (400 apart)	
GM1	Lane Line	150 100	2000-4000	150 100	1500-3000
GM2	Guide Line	100	500-1500	100	500-1500
GM8	Kerb face Marking	Black: 600 1000 White: 600 1000		Black: 600 1000 White: 600 1000	

Table 11 (Continued)

SADC-RTSM Marking Number Ref.	Description	Dimension (mm)	
		Rural (speed> 60 km/h)	Urban (speed ≤ 60 km/h)
		Length	Length
RM8	Mandatory Direction Arrows	4000	4000 2500
RM17	Exclusive use lane symbol	N/A	1600 (cycle) 400 (bus)
WM1	Railway Crossing Ahead	7500 Line width: 400	4000 Line width: 200
WM5	Give Way Control Ahead	4000	2500
WM6	Lane Reduction Arrow	6000	4000
WM7	Lane Direction Arrow	4000	4000 2500
WM8	No Overtaking Line Ahead	4000	3000
WM10	Road Hump	1500 Square: 500x500	1500 Square: 500x500
WM11	End of Exclusive Use Lane Arrow	N/A	WM11.1: 7200 WM11.2: 6000
GM3	Bifurcation Arrow	5000	4000
GM6	Cycle Facility	N/A	1600
GM7	Word Marking	4000	4000 2500
GM9	Speed Limit	7500 symbol width: 1500	4300 Symbol width: 1500

6.3.3 Rationale for an All-White Pavement Marking

We have recommended that all-white pavement marking system be adopted by partner states. This is the practice in many industrialized countries especially in Europe. This section outlines economic and technical reasons for the recommendation.

From the historical perspective, according Hawkins et al (2002), MUTCD (USA) required the use of the yellow markings to separate traffic travelling in opposite directions in 1971. However, USA has been considering the option of using all-white pavement marking system used in most industrialized countries. The technical and economic advantages of using white pavement markings over the yellow line identified in the report by Hawkins et al (2002) and relevant to our situation included:

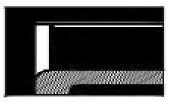
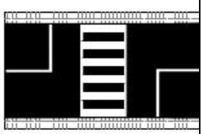
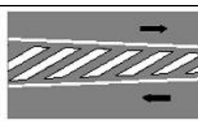
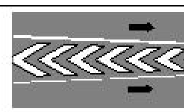
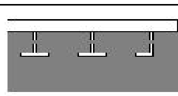
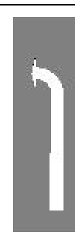
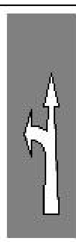
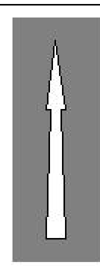
i. All other factors being equal, white markings have higher retro-reflectivity than yellow markings. Practically, better retroreflectivity at night translates to significant prevention of crashes.
ii. Some of the pigments used in yellow markings are difficult to recognize as yellow in night-time conditions.
iii. White markings are cheaper than yellow markings especially considering the higher service life.

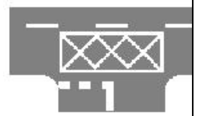
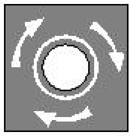
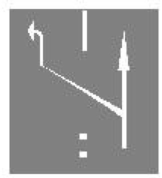
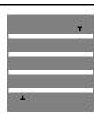
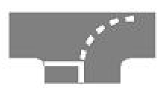
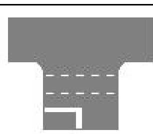
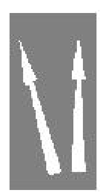
Given that drivers across the Partner States understand the use of solid line to indicate the mandatory requirement not to cross over to the other lane the solid white line can conveniently replace the use of yellow line to separate traffic travelling in opposite directions. The implementation may be done as part of maintenance at the end of the service life of existing markings and so no special budget will be required to effect the change apart from the awareness campaign to inform drivers of the change. On the other hand because of legalities it is preferred to implement the changes immediately across the partner states and budgetary implications and time frame are given in sections 6.9 and 6.10.

6.3.4 Pavement Marking Schedule

As mentioned before the variation of practices of pavement markings is not significant apart from the colour coding which shall be harmonized by the proposal to adopt the all-white pavement marking system. We present in Table 12 our recommendation for a basic schedule of an all-white pavement markings.

Table 12: Pavement Markings Schedule (ALL WHITE)

SN	Sign	Description	SN	Sign	Description
Regulatory markings					
RTM1		Stop Line	**RTM2**		Give Way Line.
RTM4		Pedestrian Crossing (zebra) markings.	**RM1**		No Overtaking Lin
RM2		No Crossing Lines.	**RM3**		Channelizing Line for lanes in the same direction (do not cross except in emergency)
RM4.1		Left Edge Line/Right Edge Line – one way roads only	**RM5.1**		Channelizing Island
RM5.2		Channelizing Island: Do not drive over the line except in emergency	**RM5.3**		Channelizing Island: Do not drive over the line except in emergency
RM6		Parking Bays	**RM8.1**		Mandatory Direction Arrow left (Right – RM8.5)
RM8.2		Mandatory Direction arrow - ahead and left (right RM8.4)	**RM8.3**		Mandatory Direction arrow (ahead)

RM9		Exclusive Use Lane Line Plus Word Marking	**RM10**		Box Junction - Do not enter unless the exit is clear
RM15		Roundabout Mandatory Direction Arrows	**RM17.1**		Cycle Facility
Warning markings: WM7.1, 7.2, 7.3, 7.4 and 7.5 shall be marked together with the word "Ahead"					
WM2		Continuity Line between through lanes and turning lanes	**WM3**		Dividing Line
WM7.1		Lane Direction Arrow left Ahead (Right: WM7.5)	**WM7.3**		Lane Direction Arrow (ahead)
WM7.2		Lane Direction Arrow ahead and left (WM7.4 ahead and right)	**WM11.1**		End of exclusive use lane – left lane (right lane WM11.2
WM12		Rumble strips			
Guidance markings					
GM1		Lane Line	**GM2.1**		Turning Guide Line
GM2.2		Pedestrian Guide Line	**GM3.1**		Bifurcation arrow - left (GM3.2 right)
GM7		Word marking: Examples include • Stop • Bus • Taxi • Slow Down	**GM9**		Speed Limit Marking

		• School • Ahead			

6.3.5 Pavement Marking Materials

General guidance:

- **Road Paint:** Use paint which dry quickly, durable and have good skid resistance.
- **Thermoplastic:** Hot-applied thermoplastic is the preferred marking materials.
- **Reflectorization: R**oad markings materials should be reflectorized by the addition of reflecting glass beads.
- **Specifications:** Road works specifications in the Partner States should specify materials that will ensure durability especially for road with heavy traffic, visibility and good skid resistance.

6.3.6 Road studs

Use of road studs

Some Partner States have clear regulation on the use of road studs. Because of their cost, road studs are generally used under the following conditions:

- Frequent mist, fog or rain (making it difficult to see the road markings)
- Poor visibility due to glare from headlights of oncoming vehicles
- Difficult alignment (e.g. roads with many bends, some of which may be hard to see)
- Roadside hazards (e.g. reduced carriageway width or limited clearance to obstacles)
- Other hazardous sites.

It is advisable to use road studs where they may reduce hazard to travellers. Due to the importance attached to the regional route network and the critical role of adequate delineation in road safety it is recommended to use road studs throughout to ensure permanent delineation of the network. The following guidance recommended.

Colour and Spacing

Recommended colour and spacing:

- RED – for Prohibition
- YELLOW – for Warning
- WHITE – for Guidance

The following table shows the recommended spacing of road studs on continuous lines and dividing line:

Lines	**Spacing**
No Overtaking Lines	Rural areas: 24 m Urban areas: 18 m
Edge Lines	Rural areas: 24 m and 50 mm away from the line Urban areas: 18 m and 50 mm away from the line
Diving Line	Every second gap

We recommend that road studs be aligned with the pavement markings and be placed between lines for parallel lines and should never be use on transverse markings since they could undermine vehicle's stopping ability and are hazardous to two-wheeled vehicles.

Specifications

Road works specification should specify the standards to be adopted. It should be noted that if the type of bonding adopted can be undermined by pavement markings then they should not be installed on top of road markings.

6.4 Traffic Signals

Traffic signals are used in the major urban centres across the Partner States and requirements are provided in the respective legislation and manuals. Deficiencies and recommendations for improvement and harmonization are outlined below.

6.4.1 Purpose, Meaning and Use

The purpose of traffic signals is to provide positive control of vehicular and pedestrians at warranted road junctions, pedestrian crossings, multilane roadways and railway crossings. The assignment of right of way is by green light illumination while the right of way is denied by a red signal. An amber (yellow) illumination is shown as a transition from green time to red time. The meanings of the traffic signal indications under the three colour system described above and for pedestrian signals are given below. A two light system is allowed in the laws of Burundi and Rwanda: the amber is replaced by simultaneous illumination of both the red and green light. However, this system is not in use and it should be de-legalized.

It is highly desirable to ensure uniform usage of traffic signals so that similar traffic conditions are controlled in a consistent manner. This encourages compliance and achieves higher safety level. The basis for installation or removal of traffic signals in the member states and their placement is therefore proposed below.

6.4.2 Basis for Installation or Removal of Traffic Signal

Traffic signal control should only be installed when an engineering study of traffic and roadway conditions at the intersections confirms that a signal control is warranted. The decision is based on careful analysis of traffic operations and needs of pedestrians and cyclists at many signalized and un-signalized road junctions combined with engineering judgement which resulted in a series of warrants defining minimum conditions under which signalization might be justified. To ensure uniformity, signalized intersections which no longer meet the minimum conditions should have their signals removed.

It is important for junctions with similar conditions within the EAC to be controlled in the same way for the interest of uniformity. The requirements for design and installation of traffic signals set out above should be adopted by the Partner States.

6.4.3 Design Principles

Generally signals are used to control traffic at warranted junctions in built-up areas on roads with speed limit of 50 km/hr. When used on roads with higher speed limits (up to 70 km/hr and rarely 80 km/hr) advance warning signs are used, visibility should be excellent, all right turning movements should be fully protected and at least one signal on each high speed approach should be mounted overhead. Table 13 presents good practice guidance on the mounting and positioning of signals at junctions, pedestrian crossings and railway crossings should be followed by all Partner States.

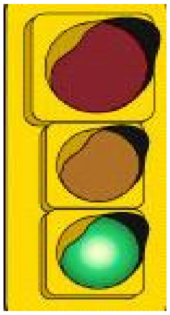
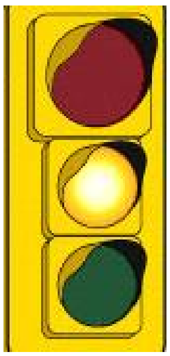
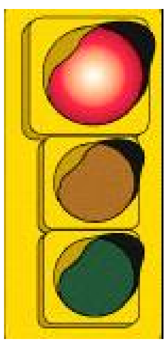
Table 13: Recommended guide to installation of traffic signals

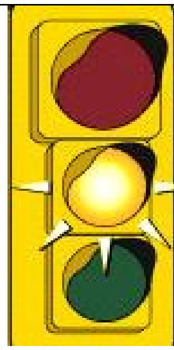
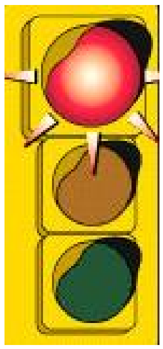
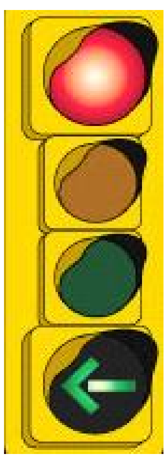
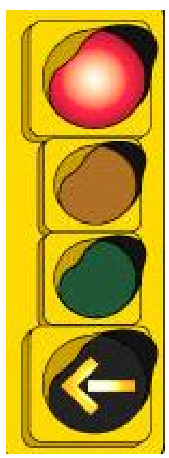
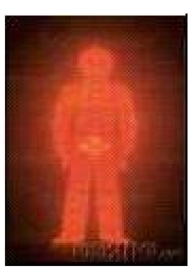
Location of the signal	Description
Signals at junctions	Signal head should be mounted on a post at the side of the road. The clearance of the signal heads from the road surface should be 2.3 m to 3m.
	The signals should be mounted vertically with the red signal at the top, yellow at the middle and green at the bottom with the lenses protected from direct sunlight by hoods. When mounted horizontally the same order should be followed from left to right.
	Signal head should not be within 500 mm of the kerb face
	Signal lens diameter should not be less than 200 millimetres and no more than 300 millimetres.
	The unit casing should be painted black or grey and the post should be painted grey.
	Overhead signals clearance height above the road surface not be less than 5.2 m.
	Install two signal heads per approach, one on the near side of the junction and another on the far side.
	The distance between the centres of the lenses not to be more than 400 millimetres apart.
	Sequence of the lights i. Red ii. Red and amber/yellow (optional) iii. Green iv. amber
	Signal heads should be visible from at least 80 m for 50 km/hr approaches and 110 m if the traffic speed is more than 50 km/h
Pedestrian signals	The signals should be arranged vertically with the red standing man signal at the top and the green walking man at the bottom and illuminated separately.
	The signal lens diameter should not be less than 200 millimetres and no more than 300 millimetres
	Signal head clearance should not be less than 2.1 m and not more than 2.6 m.
Signals at Railway Crossing	The signal lens diameter should at least be 200 mm
	Signal should be positioned within 5 - 10 m of the nearest rail line
	Signal should be mounted at the left hand side of the road or above the carriageway
Remaining red time	The remaining time before signal change from red to green of green change to amber should be indicated on a separate green signal head at intersections that can benefit from the innovation, that is, intersections with cycle length greater than 100 seconds or where it is likely that the number of crashes can be reduced by the installation.

6.4.4 Traffic signal indications in use and their meaning

Table 14 presents traffic signal indications and their meaning as per international convention and practice across the partner states.

Table 14: Traffic signal indications in use and their meaning

S/No.	Indication	Meaning
1.		**Steady Green signal** Traffic, except pedestrians, facing a CIRCULAR GREEN (S1) or GREEN ARROW signal indication (S2, S3, S4) may proceed straight through or turn right or left except as such movement is modified by lane-use signs (for example S1B sign for bus use lane), turn prohibition signs, lane markings, or roadway design.
2.		**Steady Yellow signal** Traffic, except pedestrians, facing a steady CIRCULAR YELLOW or YELLOW ARROW signal indication is thereby warned that the related green movement is being terminated or that a red signal indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
3.		**Steady Red signal** Vehicular traffic facing a steady CIRCULAR RED signal indication alone shall stop at a clearly marked stop line, but if there is no stop line, traffic shall stop before entering the crosswalk on the near side of the intersection; or if there is no crosswalk, then before entering the intersection, and shall remain stopped until a signal indication to proceed is shown.
4.		**Flashing signal** Flashing Yellow - When a YELLOW LENS is illuminated with rapid intermittent flashes, vehicular traffic may proceed through the intersection or past such signal indication only with caution. Flashing red - When a red lens is illuminated with rapid intermittent flashes, vehicular traffic shall stop at a clearly-marked stop line; but if there is no stop line, traffic shall stop before entering the crosswalk on the near side of the

	 	intersection; or if there is no crosswalk, at the point nearest the intersecting roadway where the driver has a view of approaching traffic on the intersecting roadway before entering the intersection.
5.	 	**Steady Arrows** • When a left-turn green arrow is shown with a red light, you may turn left from the left-turn lane.(S3) • When a right-turn green arrow is shown with a red light, you may turn right from the right-turn lane (S2) • When a straight-through (green-arrow-up) green arrow is shown with a red light, you may enter the intersection from the straight-through lane (S4) After the Right/left-turn or up green arrow, a yellow arrow may appear. This means the green light is about to appear for traffic in both directions. Do not start your left turn. Stop if you can do so safely; otherwise, complete your manoeuvre with caution.
6.		**Steady Red Pedestrian Figure** This is displayed to warn pedestrians not to step out onto the road and should wait by the kerb.
7.		**Green Walking Pedestrian Figure**

		This is displayed to pedestrians to indicate to them that after checking that it is safe, they may cross the road.
8.		**Flashing Red Pedestrian Figure** This is displayed to warn pedestrians not to step out onto the road and should wait by the kerb, but should finish crossing when already on the road.

6.5 Traffic Signs for School Zones, Pedestrians Crossings, Physically Challenged Pedestrians and for Hazardous Road Locations

6.5.1 General

School going children constitute significant proportion of the population of any country and require specific attention for schools in urban centres or in rural areas where school route crosses a major road. It is important that similar traffic situations are treated in the same manner for the interest of uniformity. Generally school crossing are not addressing very specifically in the existing guides or regulations but their needs are assumed to be met within the existing framework of road signing.

We recommend that for interest of uniformity schools crossing be managed in similar manner within the partner states. Schools, traffic police and respective road agency should develop a school route plan consisting of a map showing streets, the school, existing traffic signs, school routes and crossings. The types of school traffic signs to be used should depend on the volume and speed of traffic, street width and the age and number of children using the crossing. The following sections describe road signs, pavement markings, traffic signals, supervised crossing and grade separation that might be used in school zones. In addition, the use of calming measures should be carefully considered because driver compliance to school zone regulatory signs as well as other signage is generally poor.

The practice of developing school route plan, defining school zones and the necessary traffic signing around schools both in urban and rural areas is recommended for adoption by partner states as means of providing for safety and convenience of school children when walking within school zones.

6.5.2 Road Signs

As for other signs, school signs shall be place where they will convey their message most effectively without restricting sight distance or being in danger of being hit by errant vehicles and generally comply with requirements for regulatory or warning signs.

The following school signs are proposed:

i. School advance warning sign: the children ahead warning sign (W308) with a supplementary information plate reading "SCHOOL"
ii. School bus stop ahead
iii. Reduced speed school zone ahead information sign
iv. School speed limit (30 km/hr)
v. End of school zone

The signs signing should comply with the regulatory, warning and information signs framework.

6.5.3 Pavement Markings

The standard pavement markings can be used effectively in the context of a school zone. Word message may be used to complement mandatory signs or as information. In particular, the word "SCHOOL" should be written across a lane(s) between the stop line and the zebra lines of a crossing within a school zone (see GM7 in Table 12).

6.5.4 Traffic Signals at School Zones

Where adequate gaps in traffic stream in a school zone are few the resulting delay may tempt scholars to endanger themselves by attempting to cross using insufficient gaps. Under such circumstances it is appropriate to consider installation of a pedestrian signal. The signal may be installed if an engineering study confirms that the situation meets the requirements stated in the warrant for installation of school crossing signal according to the MUTCD or other best practice guide.

6.5.5 Supervised Crossing

Supervised crossing is needed where gaps are not sufficient and need to be created by an adult who is mandated to stop vehicular traffic to allow pedestrians to cross. The following practice is recommended:

i. Uniform of the guard: high-visibility retro-reflective safety clothing marked **STW (School Traffic Warden)** on the back.
ii. Use of **STOP** paddle as a hand signalling device.
iii. Selection of school traffic warden shall ensure that people of at least average intelligence, with sense of responsibility for safety of school children and are respectable are assigned the responsibility of supervising a school crossing.
iv. The school shall have primary responsibility for the appointment of a school traffic warden but shall consult with the traffic police and the respective road authority.
v. The hours that a school crossing needs to be supervised shall be agreed between the school, traffic police and the respective road authority.

6.5.6 Grade Separated Crossings

A grade separated facility may be appropriate means of physically separating school children from vehicular traffic. Generally pedestrians prefer to cross at grade and grade separated facility should be considered when road design type or traffic volume and speed is demonstrated by an engineering study to favour a grade separation. An over pass is easier to maintain and supervise than an under pass and therefore should be preferred unless topography is particularly favourable for an underpass. Grade separated facility may require the use of barriers to prevent pedestrians from crossing at grade.

6.5.7 Pedestrian Crossing on Roads with heavy Traffic

This is a common problem where traffic volume is high since drivers do not respect and give way to pedestrians wishing to use a pedestrian crossing. This results in considerable delay to pedestrians who may attempt to use inadequate gaps in traffic and as a result may be involved in a crash. In many cases the use of pelican crossings are warranted but not installed. It is recommended to install a overhead sign with a lighting which blinks when a pedestrian need to cross. The blinking sign should include a requirement for drivers to yield right-of-way. When preparing the warrants and specifications for blinking lights above pedestrian crossings road authorities my wish to consult best practices.

6.5.8 Physically Challenged Pedestrians

Physically challenged road users include those with poor vision, those who cannot hear and those with impaired mobility capabilities. There is a need to warn other road users regarding their presence on a particular location so they can exercise extra caution. We recommend including appropriate signs the warning sign schedule namely warning sign numbers W306.1, W306.2, W306.3, W307.1, and W307.2. The provision of audible sound at signalized intersections should be used to alert those who cannot see the pedestrian signal when it is time to cross. Road design should include tactile surfaces to alert pedestrians with poor vision regarding the presence of a crossing facility. Proper design should allow for ramps to facilitate use by those in wheel chairs at pedestrian crossings. The design and signage of our roads must provide for universal access.

6.5.9 Hazardous Road Locations

Many locations known to have high frequency of traffic crashes have not been properly treated and their safety can be improved by installation of warning signs. The effectiveness of the warning signs can be improved by providing a blinking light at night. The precise nature of the hazard may be indicated in advance and must be marked by a hazard marker at the location. Tight curves should be marked with lighted delineators to guide the drivers at night.

6.5.10 Use of Appropriate Technology

Technology is advancing very rapidly and road authorities should be on the look-out in order to take advantage of research and development from best practice countries. The application of lighting in the cases mentioned in this section may be accomplished by solar power. This is very appropriate given the unreliability of electric power supply facing all the partner states. The innovation that allows the indication of remaining red (or green) time should be adopted.

6.5.11 Summary of Recommendations

In this section we summarize the recommendations addressing the needs of pedestrians and hazardous locations.

For school zones:

i. The practice of developing school route plan, defining school zones and the necessary traffic signing around schools both in urban and rural areas is recommended for adoption by Partner states as means of providing for safety and convenience of school children when walking to school within school zones. The road signing should always include a 30 km/hr speed limit and a “SCHOOL” secondary sign with appropriate traffic calming measures.

For physically challenged pedestrians and drivers:

ii. The proposed warning signs schedule includes signs to signify the presence of physically challenged road users (challenged vision, hearing & mentally retarded).

iii. The design of the facilities to take into account the specific needs of the physically challenged to ensure universal access of the road space.

iv. The positioning of red, yellow and amber (vertically or horizontally) to always be consistent to help those with colour blindness.

For crossings where pedestrians suffer delay due to drivers’ failure to yield right of way

v. Provide an overhead sign informing drivers of the presence of a pedestrian wishing to cross and requiring them to yield the right-of-way. The sign should be provided with light which blinks in the presence of a pedestrian wishing to cross.

For hazardous road locations

vi. All hazardous road locations should be provided with appropriate hazard markers. On the approaches to the hazardous location road authorities should install appropriate warning sign including blinking hazard lights.

vii. Tight curves should be provided with reflective or lighted markers to improve visibility during the night.

Use of appropriate technology

viii. Partner states are encouraged to use solar power to overcome current problems of variable availability of electric power supply. Use of solar power is particularly suited to supply energy to the proposed blinking lights and lighted delineators.

6.6 Signing for Traffic Calming

6.6.1 Introduction

The principle goal of traffic calming is to improve traffic safety and the environment by moderating traffic behaviour through physical and legislative measures aimed at reducing vehicle speeds and/or traffic volumes whilst giving due regard to mobility and accessibility requirements. It has to be borne in mind that the road signs and markings are used when implementing traffic calming measures and these signs generally fall into the regulatory, warning and information signs.

There is a great need to harmonize the applications of traffic calming in the partner states in order to do away with the haphazard applications in favour of a systematic approach that ensures uniformity and the achievement of the intended purpose with due regard to the needs of all road users and the general public.

6.6.2 Types of Traffic Calming Measures

The types are categorized in terms of measures located at intersections, those located between intersections and area-wide measures.

Measures located at intersections

Traffic calming measures applied at intersections are primarily aimed at reducing speed into the road or limiting or removing access movements. These measures are:

- Mini-roundabouts;
- Raised intersections;
- Intersection diverters;
- Street closures;
- Intersection narrowing.

Measures located between intersections

The objective of traffic calming measures applied between intersections is to reduce vehicle speeds. The commonly used calming measures include:

- Speed humps/tables;
- Chicanes, pinch points or chokers;
- Rumble strips;
- Carriageway narrowing;
- Pedestrian crossing table;

Area-wide measures

Area-wide measures include the following applications of traffic calming:

- The Woonerf – this means “shared space” which is a concept rather than a specific traffic calming measure.
- More traditional general treatment such as one way systems. The system is applied to eliminate through traffic movements, reduce traffic conflict and to reduce vehicle speeds.

6.6.3 Planning for Traffic Calming

Traffic Calming Objectives and Planning Overview

In achieving the goal of traffic calming measures as defined above, the objective is not to exclude traffic but rather to manage and moderate its behaviour by ensuring no significant reduction of accessibility and mobility of pedestrians. To address the problem associated with the introduction of traffic calming measures, there is a need to identify planning aspects which need to be considered during the desk study, field investigations and consultations.

Traffic Calming Study Requirements

To evaluate and conclude whether the requests from residential groups for implementation of traffic calming measures are justified or unjustified, there is a need to ensure that a proper and sufficiently comprehensive traffic/transport study is undertaken.

Public Involvement

To have a good point for implementation of traffic calming measures, the involvement of public is necessary as are the residents who know or have knowledge of their area, in terms of its traffic issues and bring concepts of problem area to the study team. In identifying who should be included in the public involvement, the following proposed list may assist in the identification of potential role players:

- Neighbourhood resident groups/associations;
- Adjacent resident groups/associations;
- Local Authority within which the neighbourhood is situated;
- Adjacent Local Authorities;
- National Road Authority when applicable;
- Representation from Emergency Services;
- Representation from the business sector, commerce and industry etc. when applicable;
- Representation from the private transport sector and from the authorities, operators and users of other transport modes if applicable.

Study Requirements

Before introducing traffic calming measures, studies must be undertaken which will address and where applicable, collect reliable information on:

- The precise concerns of residents (through traffic, speed, noise, accidents etc);
- The magnitude of the problem;
- The origins and destinations of the external traffic;
- An examination and analysis of the roads at the “correct” location;
- Travel times through the residential neighbourhood;
- The identification of other routes/roads which have the potential to be impacted should traffic calming measures be introduced;
- The implications on residents should traffic calming measures be introduced;
- The implication beyond the bounds of one neighbourhood;
- Emergency vehicle access requirements;
- Public transport implications.

6.6.4 Traffic Calming Sign Applications

Road traffic signs related to traffic calming (for example speed hump, speed limits etc) are prescribed in the Road Traffic Act. Additional information signs to raise awareness of road users may be necessary. Table 16 presents a checklist for use when the need for implementation of roundabouts, speed humps, chicanes/pinch points and raised intersections is under consideration:

Table 15: Traffic Calming Measures and Checklists

Type of Traffic Calming Measure	Checklists
Mini-roundabout	- Is any mini-roundabout in the area difficult to see, either by day or by night? - Is the use of retroreflective road studs advisable?
Speed Humps	- Does street lighting exist in the area? - Is the speed hump an isolated one or part of an area traffic calming treatment? - Is the speed hump part of a traffic calming treatment involving a range of different traffic calming measures? - Is the speed hump a flat top hump which is used as a pedestrian crossing?
Chicanes or Pinch Points	- Are peak hour traffic volumes under 600 vehicles per hour? - Is the area adequately illuminated? - Is the road used by public transport vehicles or a significant percentage of heavy goods vehicles? - Is operation to be one-way or two-way? - Are chicanes to be landscaped? - Are diagrammatic signs required to assist awareness?
Raised Intersections	- Is the junction signalized or not? - Are additional measures necessary to control pedestrians? - Can the vertical height of the plateau be made clearly visible to drivers?

From the answers to the questions the need for additional signing can be determined. From the point of view of traffic signing it is important that the road users be made aware of the measures and the measures be made visible to the road users at a sufficient distance to allow proper and safe response.

There is also a need to standardize the design of the physical devices used for traffic calming. The current practice by road authorities to apply varying designs including excessive heights of road humps should be abolished immediately.

6.7 Road Signs, Signals and Pavement Markings for Freeways

The EAC economies are among the fastest growing in Africa and soon some of the regional routes will have enough traffic to justify upgrading to freeway roadways. We recommend that the signing for freeways adopt the SADC-RTSM code and standards.

6.8 Impact of Harmonization and Recommended Improvements

The adoption of the proposed sign shape and colour code will affect partner states to different extents due to the slight differences in the current practice. This is reflected in the Table 16 which presents the status of the partner states or actions they need to take upon the approval of our recommendations. Since only few signs will need to be replaced in the regulatory and warning groups (no parking, no stopping) and change in colour code is only for the guidance and information signs which are generally rarely installed, it is recommended that the change be implemented gradually. However, implementation for speed limit signs on all roads should be done according to the road map below to allow early realization of the expected safety benefits.

Table 16: Impact of adopting the recommended signs colour code across partner states

Partner State	Regulatory signs: Control and Command groups	Regulatory signs: Other Groups	Warning signs	Guidance signs	Information signs
Burundi	Her proposal for a new highway code (dated January 2011) generally conforms to the recommendations of this report on signs coding. Immediate implementation is therefore in order.				
Kenya	Current code is the same as the recommended code	To adopt reservation signs group code.	Current code is the same as the recommended code	To adopt and implement the recommended colour code	To adopt and implement the recommended colour code
Rwanda	Rwanda is in the process of implementing the coding following her adopting the SATC – RTSM signing system. To complete implementation of the recommended system, monitoring and evaluation is necessary.				
Tanzania	Adopted the SATC-RTSM system and has published her version of simplified road signs manual. Signing on the trunk roads generally conforms to the recommended system. However, there are signs which do not conform to the recommended code on local roads. To complete implementation of the recommended system, monitoring and evaluation is necessary.				
Uganda	Current code is the same as the recommended code	To adopt reservation signs group code.	Current code is the same as the recommended code	To adopt and implement the recommended colour code	To adopt and implement the recommended colour code

The proposed change to all-white pavement markings may cost the partner states up to 828 million US Dollars if all paved roads are paved according to the recommended markings during the year 2014. The estimate is based on assumption that durable materials 5 to 10 mm thick and 100 mm wide lines shall be applied along the centreline and edge lines, remarking of all zebra crossings and lane delineation on all multilane roads at a rate of 4 to 6 US $ per linear metre. If the project is limited to replacing the yellow pavement marking lines only the estimated cost may be reduced to about US Dollars 340 million. However, it is to be noted that signing and marking of roads is generally poor across the partner states and every effort should be made to secure the necessary funds not only for the pavement markings but also for widespread application of guidance and information signs to cater for convenient movement of people. The distribution of the costs for the component of all-white pavement marking across the partner states is shown in Table 17.

Table 17: An indication of financial implications for adopting all-white pavement marking system immediately

Partner State	Paved Centerline km*	Total painting / repainting cost in US $	Replacing yellow lines only in US $
Burundi	1,500	29,250,000	NA**
Kenya	15,000	292,500,000	97,500,000
Rwanda	2,000	39,000,000	NA**
Tanzania	13,000	253,500,000	169,000,000
Uganda	11,000	214,500,000	71,500,000
Total	42,500	828,750,000	338,000,000
*projected road length for the year 2014. **Should consider complete total painting/repainting of their network using durable materials.			

Justification for the recommended change to all-white pavement markings

The decision to apply durable, retroreflective all-white pavement marking system may be supported by the following benefits:

i. Under similar circumstances white pavement marking has higher visibility (retroreflectivity) than yellow markings. This is a strong incentive for the all-white pavement marking system on the grounds of safety benefits.
ii. At an equal initial luminance value (say, 100 millicandelas per square metre per lux [mcd/m^2/lux]), white lines have service lives that are longer than yellow lines by 42 percent. As an example, white lines have service life of 36 months versus 24 months service life for yellow lines, under similar traffic volume, surface materials etc. This is a strong incentive for an all-white pavement marking on the basis of cost saving.
iii. Retroreflective pavement markings have been shown to reduce the number of traffic crashes occurring under darkness under normal weather (dry weather) by about eleven percent. Since fatal crashes are more likely to occur under darkness than injury or property damage crashes, the safety gains are considerable (a loss of at least 60 million US Dollars in crash costs are expected to be saved annually across the partner states due to the application of durable retroreflective all-white pavement marking system).
iv. Use of durable markings increases the service life of pavement markings and thus a lower annual maintenance budget shall be required for the pavement marking systems.

On the legal side, Rwanda and Tanzania are members of SADC and are therefore expected to follow the SADC-RTSM. However, the SADC-RTSM systems allows for national variants in signs and markings among member countries. Given the economic and safety advantages of an all-white pavement markings partner states are urged to adopt the system sooner rather than latter when the cost of change will be even higher.

6.9 Implementation Road Map

We recommend the following roadmap for the implementation of the harmonized and improvements in road signing and pavement marking system:

i. Partner states amend their regulations to conform with harmonization recommendations within one year,
ii. Each partner state to develop and publish its road traffic signs manual and to up-date it's Highway Code according to the harmonized road traffic signs, markings and signals within two years.
iii. Partner states implement all-white pavement marking system within two years. Funding for this activity should be coordinated through the EAC for effective, coordinated implementation. Implementation over a longer period may be considered on the grounds of the significant financial implication. In this case all-white pavement markings should be applied on all new projects and during scheduled repainting on existing roadways. However, this option will delay the achievement of the safety benefits.
iv. The harmonized signs and installation speed limit signs at the beginning of each road and at intervals of ten to fifteen kilometres along all paved roads within five years, and
v. Each Partner state to ensure that training of drivers is done using the approved road signs schedule and an effective awareness campaign to inform road users regarding the changes is effectively implemented immediately after the amendment of the regulations.

The five year period for installation of retro-reflective signs and speed limit signs is adopted as part of the implementation of the UN Decade of Action for Road Safety (addressing the safer roads component) adapted in the framework for the development of the East African Road Safety Master Plan presented in Annex D. Partner states should be allowed to maintain in their regulations road signs peculiar to their own country's usage and most appropriate language provided that such signs shall not constitute violation of the harmonized schedules. Areas recognized and preserved as historical/heritage sites e.g. Mji Mkongwe – Zanzibar should be exempt from all proposed changes.

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FINAL REPORT

ANNEX A.7

HARMONIZATION OF VEHICLE SAFETY AND FITNESS

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GLOSSARY AND ACRONYMS

BICO	Bureau for Industrial Cooperation
CVSA	Commercial Vehicle Safety Alliance
DVELA	Driver and Vehicle Examination and Licensing Agency
EAC	East African Community
EATTFP	East African Trade and Transport Facilitation Project
FHWA	Federal Highways Administration
HGV	Heavy Goods Vehicle
ISO	International Standards Organization
JEVIC	Japan Export Vehicle Inspection Centre
KEBS	Kenya Bureau of Standards
LPG	Liquefied Petroleum Gas National Car Test (NCT)
LTSA	Land Transport Safety Authority
MCSAP	Motor Carrier Safety Assistance Program
NHVAS	National Heavy Vehicle Accreditation Scheme
NHTSA	National Highway Transportation Safety Administration
PMVI	Periodic Motor Vehicle Inspection
PSV	Passenger Service Vehicle
RMVI	Random Motor vehicle Inspection
SADC	Southern African Development Cooperation
SATCC	Southern African Transport and Communications Commission
SUMATRA	Surface and Marine Transport Regulatory Authority
TBS	Tanzania Bureau of Standards
UNBS	Uganda National Bureau of Standards
VCA	Vehicle Certification Agency
VOSA	Vehicle and Operator Services Agency
WHO	World Health Organization
WOF	Warrant of Fitness

EXECUTIVE SUMMARY

In examining the causes of motor vehicle accidents, a number of experts and studies have identified and categorized vehicle accidents to be caused by human factors, roadway and environment factors, and vehicle factors. In general, human factors are considered to be the most prevalent factor contributing to accidents, followed by roadway environment and lastly vehicle factors.
On vehicle factors, two types of vehicle-related failures can contribute to traffic crashes; *equipment-related* failure and *maintenance-related* failure. Equipment-related failures include both original component (genuine) and aftermarket component (non-genuine) that functions improperly or not manufactured or installed to standard. Maintenance-related failures result from an improper maintenance which may impair the function of the vehicle. The purpose of conducting periodic- or regular vehicle inspection, therefore, is to reduce accidents caused by *maintenance-related failures* of vehicle components.

This chapter presents main findings and recommendations of the harmonisation of vehicle periodic inspections for fitness and safety in EAC states. The objectives of this chapter are to:

- review the vehicle inspection practices around the world
- review the proposed COMESA-EAC-SADC framework and standard on vehicle inspection,
- study the existing situation on use of vehicle, accident statistics, framework and standards for vehicle inspection in EAC countries, and
- recommend the framework and minimum standards for periodic motor vehicle inspection (PMVI) for harmonization in EAC countries.

In all the EAC countries vehicle safety standards are captured in respective Road Transport and Road Traffic Acts. The Acts specify vehicle safety standards requirements and inspections that the authorities have to observe during the initial registration, road licensing, and operators licensing. However, the degrees of implementation and enforcement differ from country to country. In general the challenges facing most of the EAC countries are:

- inadequate *infrastructure and facilities* to conduct technical inspections.
- lack of *equipment and qualified personnel* to conduct technical inspection.
- inadequate *enforcement* of mandatory technical inspections.
- rampant m*alpractices and corruption* which hamper the effectiveness of the efforts that the governments are trying to institute.

Recommended PMVI Framework and Regulations

The recommended framework for harmonisation in EAC states has been achieved through a review of good practices around the world in high, middle, and low income countries. Other analyses conducted to supplement the proposal include:

- trend on advanced vehicle safety systems
- the current situation in the EAC countries on vehicle inspection and in particular the problems on infrastructure (facilities and equipment), human capacity, legislation and enforcement, malpractices and corruption, and the vehicle fleet.
- The standards and inspection specifications proposed for the COMESA/EAC/SADC countries.

To reduce accidents caused by maintenance-related component and system failures it is recommended that all vehicles using public roads including government and military vehicles are obliged to undergo a mandatory PMVI. The vehicle categories for PMVI should be:

- private light vehicles (GVM < 3500 kg);

- light passenger service vehicles (PSV up to 8 passengers);
- passenger service vehicles (PSV> 8 passengers);
- heavy goods vehicles (HGV);
- trailers;
- private motorcycles and three-wheelers;
- taxi motorcycles and three-wheelers.

The recommended inspection frequency for the above categories should be as follows.

Vehicle category	Initial PMVI (since new)	Frequency (months)
Private light vehicles (GVM < 3500 kg)	2 years	12months
Passenger service vehicles (PSV < 8 people)	1 year	6 months
Passenger service vehicles (PSV > 8 people)	1 year	6 months
Commercial trucks (<3.5 tonne) and Heavy goods vehicles (>3.5 Tonnes)	1 year	6 months
Trailers	1 year	6 months
Motorcycles and three-wheelers (private)	2 years	12 months
Motorcycles and three-wheelers (taxi)	1year	6 months

The recommended fees (USD) for the above categories of vehicle are shown below.

Vehicle category	Inspection Fee (USD)
Private light vehicles (GVM < 3500 kg)	15
Passenger service vehicles (PSV < 8 people)	15
Passenger service vehicles (PSV > 8 people < 30 people)	20
Passenger service vehicles (PSV > 30 people)	30
Commercial vehicles (<3.5 tonne)	20
Heavy goods vehicles (>3.5 Tonnes, 2 axles)	30
Heavy goods vehicles (>3.5 Tonnes, 3 and more axles)	40
Trailer (1 axle)	10
Trailer (2 and more axles)	15
Motorcycles (private)	5
Motorcycles (taxi)	5

It is further recommended that vehicles which pass the PMVI should be given a certificate and a sticker. The certificate should be put in the vehicle with other vehicle documents for police to inspect if needed at any time. The sticker should be placed at a prominent place, and should show: the vehicle registration number; the date when the next inspection is due and a code identifying the centre which the inspection was conducted.

Vehicles which fail the PMVI should be allowed to rectify the problem and report back for re-inspection of the faulty item within two weeks. It is proposed that 20% of inspection fee should be charged for re-inspection. Full inspection should be conducted if the vehicle is not taken back for re-inspection within two weeks, in which case full inspection fee should be charged.

Minimum Standards for PMVI

Vehicle systems and items that increase the likelihood of accident when they fail and those which increase the severity of casualties after the accident are the target for the recommended PMVI scheme. These systems/items are categorized as follows.

	System/Item that increase chances of accident and injuries when they fail	**Obvious system/item**
1	Systems/item that *impair the driver's control of direction and/or speed* of the vehicle	Steering, tyres, brakes, insecure driver's seat.
2	Systems/items that *impair the driver's view* of the road	Windscreen, wipers and wiper washer, side and rear view mirrors, headlights
3	Systems that *impair the visibility* of the vehicle from other road users	Lights, horn.
4	Components/items that intrudes into other users' road space or undue danger or nuisance	Oil leaks, sharp projections, excessive smoke, excessive noise, body modification
5	Components/items that *impair the built-in occupant protection*	missing or broken seat belts, insecure seats, weakened body structure
6	Components/items that *increase the risk of further injury* to the occupants and other road users after an accident	insecure fuel tank, inoperative emergency exits, battery security

It is further recommended that systems/items that cannot be visually inspected to assess their condition be tested by either special equipment which have been developed to internationally recognized standards specifically for that task, or by test driving the vehicle. Systems/items should not be dismantled for the purpose of technical inspection. The full recommended groups of items for safety for the PMVI are:

- **Group I**: Vehicle documents and identification
- **Group II**: Items which affect control of vehicle (brakes, wheels and tyres, steering system, suspension system, driver seat)
- **Group III**: Items which impair driver view of the road: (windscreen, wipers and wiper washer, side and rear view mirrors, headlights and spotlights alignment)
- **Group IV**: Items which reduce visibility of the vehicle: (lights, reflectors, horn)
- **Group V:** Items which impairs built-in occupant protection: (seats and seat belts, body structure, doors, floor and steps)
- **Group VI:** Items which intrudes other road users safety: (oil leaks, noise, excessive smoke, sharp protrusions, mud guards)
- **Group VII:** Items which increase risk of further injuries after accident: (emergency exit, fuel system, battery security)
- **Group VIII:** Extra safety items for HGV and Trailers: (rear under run protection, cab mounting, trailer couplings, turntable and king pin, trailer parking brake, trailer lights and reflectors, twist-locks for securing containers)
- **Group IX**: Uncertified modifications: Any modification which falls into any of the I-VIII groups above such as:

The minimum environment standards at the time of PMVI are recommended to be as follows:

- Spark Ignition Engines (Petrol, LPG, and CNG Engines):
  - Carbon monoxide (CO)
  - Hydrocarbons (HC)
  - On board diagnostic-II (OBDII) condition
  - Fuel tank cap
  - Noise

    - A/C refrigerant

- Compression Ignition Engines (Diesel Engines):
    - Particulate Matter (PM)
    - Fuel tank cap
    - Noise
    - A/C refrigerant

Supervision, Organisation and Ownership of PMVI

The operation of inspection facilities should be under private company and closely monitored by Government (ministry or department responsible for road transport). The vehicle inspection quality and performance also need to be continuously monitored and the degree to which performance targets are met be assessed. Inspection frequency, fleet size and length of inspection should determine the number of testing facilities and minimum staffing requirements.

Inspection manuals showing all the PMVI procedures and the pass/fail criteria for each item be developed before the exercise of inspection is established. The manuals should be accessible to the public to increase public confidence and transparency on the inspection scheme, as well as inform drivers and fleet operators on standards they are supposed to meet. Likewise, the inspection centers should be regulated and randomly audited in terms of inspection procedures and accuracy of equipment. The scheme should be designed in such a way that it would reduce opportunities for corrupt practices among vehicle inspectors.

We further recommend that inspection staff attend a specialized (tailored) course on vehicle technical inspection, and should be qualified to a specified standard of technical ability necessary in vehicle inspection. As such there is a need to develop a training curriculum on vehicle inspection covering technical inspection procedures, standards and specifications for vehicle safety items and systems, testing equipment, administration, and quality control. The pre-requisite for a vehicle inspector candidate should be a technician level in auto-mechanics or degree level in mechanical engineering.

Facilities and Equipment for PMVI

The premises for the inspection and certification of vehicles must have inspection area which:

- enables a safe and thorough inspection
- has sufficient lighting to enable good visibility of the vehicle being inspected and the equipment used in the inspection process.
- Complies with Occupational Safety and Health requirements
- roofed and with paved floor

It is further recommended that the centre should have offices for administrative works and document storage.

7 HARMONIZATION OF VEHICLE SAFETY AND FITNESS

7.1 Introduction

Vehicle safety is an essential issue that needs attention of most countries. This is because road transport is essential element in social development. While road transport is important, road accidents have claimed the lives of many people. The WHO (World Health Organization) estimated that in the year 2002, 1.18 million people died worldwide due to motor vehicle accidents. Besides the vehicle's occupants, this number also includes pedestrians and cyclists involved in these accidents. Another 50 million people were injured in road traffic related accidents during the same year.

In examining the causes of motor vehicle accidents, a number of experts and studies have identified three groups of factors that contribute to crashes;

- **Human factors** involve the actions taken by or the condition of the driver of the vehicle, including speeding and other traffic violations, as well as the effects of alcohol or drugs, inattention, decision errors, and age.
- **Roadway environment factors** that contribute to or are associated with accidents include the design of the roadway (for example, medians, lane width, shoulders, curves, access points, or intersections); roadside hazards (for example, poles, trees, or embankments adjacent to the road); and the roadway conditions (for example, rain, ice, snow, or fog).
- **Vehicle factors** include vehicle-related failures and vehicle design issues that contribute to accident.

In general, human factors are considered to be the most prevalent factor contributing to accidents, followed by roadway environment and vehicle factors. However, motor vehicle accidents are complex events that rarely have a single cause. Imagine how challenging it would be to point out the cause of an accident of overloaded bus that occurred on a narrow and wet road, driven by inexperienced driver, who had been drinking all night and was talking on a cell phone during the accident. It would likely be the combined effect of a number of these factors that contributed to the accident.

Two types of vehicle-related failures can contribute to traffic crashes: Equipment/component - related failure and maintenance-related failure. Equipment-related failures include both original component (genuine) and aftermarket component (non-genuine), that functions improperly or not manufactured, or installed to standard. Depending on the function of the component in the vehicle, failure of the component might lead to the loss of a vehicle's handling capabilities, resulting in an accident.

Maintenance-related failures result from an improper maintenance of vehicle components, which may impair the function of the vehicle. Examples of maintenance-related failures include worn tire or brakes. Analysis conducted by National Highway Transportation Safety Administration (NHTSA) of USA on accident records found that from 1997 through 2001, there were about 778,000 accidents in which police identified that a specific vehicle-related failure might have contributed to the crash. Where these failures were identified, brake systems and tires were identified at 29% and 27%, respectively.

One vehicle factor that NHTSA believes may have contributed in many accidents is underinflated tires. In 2001, NHTSA conducted a study that found 27% of passenger cars and 33% of light trucks were being driven with one or more underinflated tires. In 2005 the US government passed an Act which will require motor vehicles to be equipped with a tire-pressure monitoring system (TPMS) to warn the driver if a tire is significantly underinflated.

Research on traffic accidents in the USA and United Kingdom has shown that one of the significant factors causing road crashes was vehicle defects; this factor contributes 12% and 8% respectively to motor accidents in the two countries[1]. Other recent studies[2] in UK indicate that 25% of goods vehicles and 11% of large passenger carrying vehicles involved in accidents have contributory defects on vehicle.

The prevalence of defects in the vehicle fleet has been found to be lower in jurisdictions with regular inspections by up to 16%[3]. Comparisons of inspected cars and non-inspected cars in the same jurisdictions suggest a drop in crash rates for the former. Studies that have compared accident rates before and after the introduction of regular inspections have generally shown decreases in injury accident rates. However, other studies have found no effect of PMVI in accident rates, which suggest that inspection programs may influence and reduce accidents by increasing drivers' understanding of the need for regular maintenance, of safety issues and of the condition of their own car.

Studies on influence of vehicle defects on accident show big variation on results because of the method used in data collection. Accident reports from police who attend the accident are quick to blame the driver, but what causes the driver to lose control could be something else. Most traffic police are not qualified to identify vehicle defects after an accident has occurred. Despite these variations of opinions, a concluding statement can be made. To what extent could regular inspections help to reduce road traffic accidents? This depends on the quality of the inspection scheme and factors like the local economy, cultural backgrounds and political conditions.

In developing countries, it is likely that vehicle defects are more often a factor in accidents as vehicle fleet condition is generally much worse. The fleet is usually older with many vehicles imported second-hand from other countries, and there may be difficulties and cheating involved in obtaining genuine spare parts. In addition, to a shortage of genuine parts, maintenance skills are often scarce and knowledge of modern repair techniques is generally poor. Lack of a formal apprenticeship system or of technical training colleges can result in poor maintenance as there is little incentive to develop trained technical repair staff.

Roadside surveys conducted in 1996 in Kathmandu, Nepal, found only 40 percent of trucks and buses inspected had front lights and a nighttime survey of long distance buses found two thirds with one or no rear lights[4]. Vehicle lighting is even more important in developing countries where road lighting, and road marking and signs are inadequate and driving conditions are poor. The statistics for Thailand from the same study showed that vehicle defects accounted for some 27.54% of road accidents.

Unlawful modification of trucks by welding reinforcements or extensions to the chassis or the addition of an extra axle can result in a vehicle with a seriously compromised safety performance. This practice is common in many countries that have ineffective regulation of construction standards. Trucks may use a combination of chassis strengthening and an extra axle to double their

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[1] Ogden, K.W. (1996) Safer Roads: A Guide to Road Safety Engineering. Avebury Ashgate Publishing Limited, London
[2] ADB. Road Safety Guidelines for Asia and Pacific Countries
[3] George R, Narelle H, Naomi K. (2000). The effect of vehicle roadworthiness on crash incidence and severity
[4] Taneerananon P, et al. (2005) Journal of the Eastern Asia Society for Transportation Studies, Vol. 6, pp. 3482 - 3496, 2005

load carrying capacity. The brake characteristics of the vehicle change with added carrying capacity of the vehicle. Even if the brake system is connected to the added axle the response of the brakes will be compromised because the effectiveness of brake master cylinder will be reduced by added wheel brake cylinders to the added axle.

Buses are often constructed on truck chassis using materials and designs that offer little or no protection to passengers in the event of an accident. Leg room between seats is often exceptionally poor in order to squeeze the maximum number of seats in the vehicle, and the seat frames themselves are usually formed from angle steel, frequently causing amputation of limbs in an accident. Emergency exits, when provided, are often blocked by a row of seats, making their access difficult after an accident.

Vehicle gaseous, particulate and noise emissions also have major effects on the environment. When at high concentrations, and in certain atmospheric conditions, these can present a health hazard. This is especially important in cities within the tropics where large numbers of people live and work on the streets, or where the topography does not permit easy dispersal of such gases.

7.2 Objectives of the Assignment

The overall objective of the assignment is to propose a Periodic Motor Vehicle Inspection (PMVI) framework and standards to be used in EAC countries. The specific objectives are:

- To review the vehicle inspection practices around the world
- To review the proposed COMESA-EAC-SADC framework and standard on vehicle inspection
- To study the existing situation on use of vehicle, accident statistics, framework and standards for vehicle inspection in EAC countries
- To recommend the framework and minimum standards for periodic motor vehicle inspection (PMVI) for harmonization in EAC countries.

7.3 Approach and Methodology

The methodology adopted for the assignment entailed the following activities:

- Visit EAC Partner States for the purpose of collecting documents from each partner state related to regulation or standard for periodic vehicle inspection, and to make initial contacts with the responsible officials.
- Detailed documents review and situational analysis and preparation of draft working papers.
- Review of good practices around the world
- Collection of stakeholders and experts' views on the proposed framework and standards for PMVI through workshops, meetings, and interviews in each EAC Partner State.

7.4 Overview of Vehicle Inspection

7.4.1 Types of Vehicle Inspection

There are two categories of motor vehicle inspection that are used to check their compliance for safety and environment. These are:

- *Type approval inspection* for new and used imported vehicles and
- *Technical (roadworthy) inspection* for in-service vehicles.

Figure 1 highlights the frame of regulation used by most countries during the life of the vehicle and positions for type approval and technical inspections.

Figure 1: The chain of links of regulations in the life of the vehicle (Source: The vehicle Chain in Europe. A Survey of Vehicles and Driving Licence Procedures. 7th Edition.2011).

Type Approval Inspection

Type Approval standards (or homologation) is not a term confined to a particular industry. Type Approval is granted to a product that meets a minimum set of regulatory, technical and safety requirements. Generally, type approval is required before a product is allowed to be sold in a particular country. The requirements for a given product may vary around the world depending on the standards used in each particular country. Hence, used product imported from another country also needs to undergo type approval inspection to check if the product also complies with the local type approval standards.

Vehicle manufacturers (through dealers or government authority) present a certificate showing that the vehicles they intend to introduce in the market meet requirements of an approval scheme in the country they intend to sale the vehicle. In most developed countries such as EAC countries, the National Bureau of Standard (a government institution) have developed safety specifications or adopted the standards from developed countries and International Standards Organization (ISO) for the type of vehicles which meet standards and have been approved to be used in public roads.

In most developed countries type approval is undertaken by a special government authority or a company which is supervised by the government to conduct type approval for vehicles only. This is because of a huge volume of vehicle sales and cross-border transportation. In the UK Vehicle Certification Agency (VCA) is an executive agency of the Department for Transport, and is the UKs Type Approval authority, and a leading certification body. VCA have been supporting the automotive industry since the early 1970s, with offices in the UK, North America, Japan (Asia Pacific), Korea, China, Italy, Malaysia, India and Australia. The services that VCA provide include Type Approval testing and certification for all road-going vehicles, including cars, trucks, motorcycles, agricultural vehicles, buses and coaches, ambulances, fire engines and motor caravans, and replacement part systems and components. Figure 2 shows a common framework for quality control and in-service road worthiness which is used by most countries around the world.

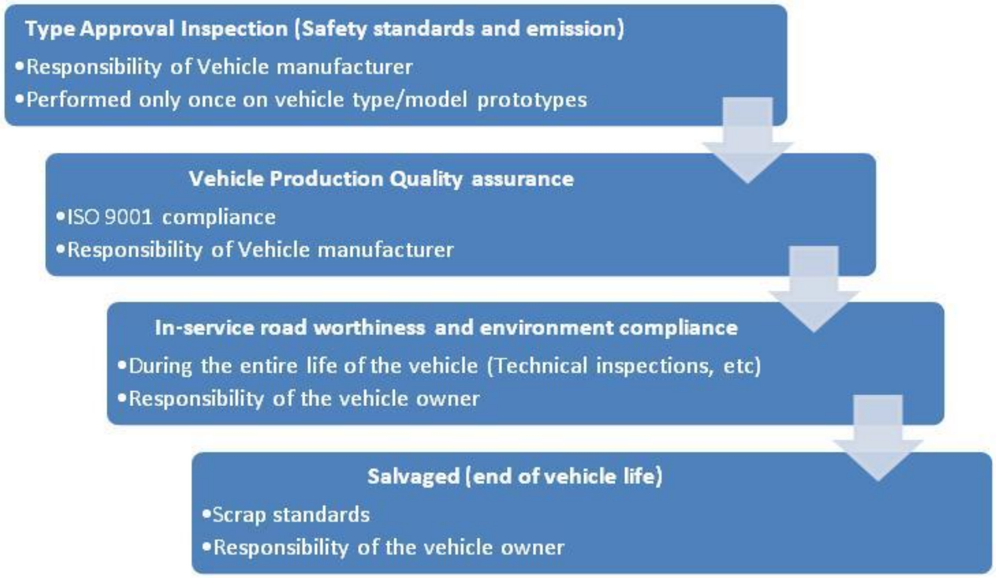

Figure 2: A common framework for quality assurance and in-service road worthiness of vehicle used by most countries around the world.

Technical Inspection

A technical motor vehicle inspection aims at checking the roadworthiness of the vehicle during its life. Technical inspections can be categorized as Regular or Periodic Inspection (PMVI) and Irregular or Random Motor vehicle Inspection (RMVI). Many countries have further regulations for a requirement to conduct technical inspection during changing of ownership, after a vehicle has been repaired from an accident and in some countries after modification on a vehicle.

The PMVI is probably the most used and most popular one in developed countries and other countries where PMVI exists. Regular can either mean an inspection with a certain frequency regarding time or regarding mileage. The former is mostly used through government regulations and it is easier to enforce than the later. In PMVI, every vehicle usually gets inspected after a certain amount of time and by this, a certain roadworthiness standard is maintained. Figure 3 illustrates a model of the levels of roadworthiness of an aging vehicle that might be achieved by PMVI for vehicles that are not fully maintained by their owners. PMVI ensures vehicles are restored to the required standards.

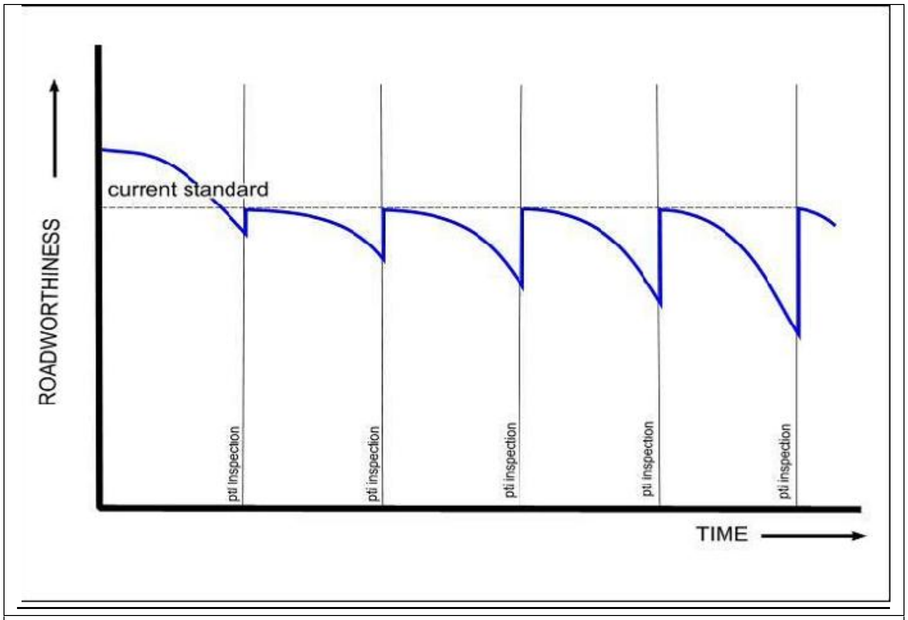

Figure 3: A model of vehicle safety deterioration (source: AUTOFORE. (2007). Study on the Future Options for Roadworthiness Enforcement in the European Union. CITA.)

The random vehicle inspection (RMVI) is the second most used official inspection and is more common in developed countries and countries which do not have PMVI schemes. Random means, that no one can predict when, and to what extent a vehicle will be inspected. Vehicles are picked out randomly or on suspicion for possible offences. In some countries with electronic databases, in random inspection the owner can be informed by the authority (which picks vehicles at random from a central database) to take the vehicle for technical inspection. Alternatively, the vehicle can be stopped at any time for roadside inspection; the latter is common for passenger service vehicles and trucks. The roadside inspections have to be performed by the police or some special governmental organizations which has the legal authority to stop the vehicle on the road.

The inspection at Change of Ownership is performed every time a vehicle is sold. Normally, it is a complete vehicle inspection comparable to a periodic motor vehicle inspection but concentrates more on checking the vehicle identity to ensure that it matches the vehicle documents. Inspection after accidents is performed in some countries, and the inspection often depends on the accident's severity. An inspection is conducted to check the quality of the repairs and if the vehicle is roadworthy again.

A vehicle is a complex machine comprised of many systems which depend on one another and a modification on one system can easily interfere with the functioning of another system or some aspect of dynamic performance of the vehicle. Hence, technical inspection after modification is mandatory in many countries. This is important measure for roadworthiness since vehicles get modified with garages and workshops that have no proper knowledge of vehicles and materials technology.

A good example here is the addition of an axle for trucks to increase carrying capacity without exceeding axle load limits at overload control points. In this example, the vehicle braking characteristics and turning ability are interfered with and can make the vehicle dynamic performance limited and sometimes unpredictable. Hence, experts should check if the vehicle systems are functioning properly and the overall performance of the vehicle has not been compromised by the modification.

7.4.2 Periodic Motor Vehicle Inspection (PMVI)

Worn, failed or incorrectly adjusted components can cause or contribute to accidents. Preventive maintenance and periodic inspection procedures help to prevent failures from occurring while the vehicle is being operated. Defects are often not attended to by the vehicle owner/driver sometimes because of limited skills and lack of technical knowledge and interest. It is easier and cheaper for some drivers to ignore warning lights and other symptoms with the hope that the problem will go away, hence, significant numbers of defective vehicles can be on the road.

The aim of any PMVI imposed by the government is to eliminate defects from the vehicle fleet by inspecting all vehicles on a regular basis and ensuring that any detected defects are repaired before allowing the car to drive on public roads. PMVI may have other effects on increasing road safety, such as promoting the number of newer vehicles on the road which may improve the safety of the vehicle fleet in a particular country, and also increase driver's awareness and culture on safety.

Some vehicle defects clearly contribute to the occurrence of accidents. However, challenges exist in identifying systems that can reduce the occurrence of such defects. Even when PMVI exist, a significant percentage of vehicles still have defects, rendering the vehicles to be not roadworthy. Yet only some of these defects appear to contribute to crashes. This would suggest that only in certain circumstances are vehicle defects contributing factors to crashes. This conclusion is not at all surprising as crashes can result from a large number of factors and a chain of events, with vehicle defects being just one of these factors.

As most vehicle owners or users do maintain their vehicles, the key question is what additional measures are required to minimize those safety related defects that have been found to be contributing factors in crashes. It would appear that such measures must recognize that defects arise due to various factors including: lack of awareness by the owner, normal deterioration with use, poor service or repairs, negligence, economic conditions, road condition in which the vehicle is operated, etc,.

In most countries where PMVI have been in existence for many years, the inspection on the safety aspects of the vehicle covers the following areas:

(i) *Systems that impair the driver's view of the road*, such as scored windscreen, inoperative wipers and wiper washer, missing mirrors and inoperative headlights.

(ii) *Systems that impair the visibility of the vehicle from other road users* or prevent the driver from indicating his or her intentions such as inoperative lights and inoperative horn.

(iii) *Systems that impair the driver's control of direction and/or speed of the vehicle* such as steering, tyres, brakes, insecure driver's seat.

(iv) *Components that intrudes* into other users' road space or undue danger or nuisance to others such as oil leaks, sharp projections, excessive smoke, excessive noise.

(v) *Components that impair the built-in occupant protection* in the event of a crash such as missing or broken seat belts, insecure seats, weakened body structure.

(vi) *Components that increase the risk of further injury after a crash has occurred* such as insecure fuel tank, inoperative emergency exits.

There are many vehicle components in the systems listed above, and the fact that periodic inspection require all registered vehicles to be inspected at least every year or two, clearly shows that thousands if not hundreds of vehicles need to be inspected everyday to cope with the demand in a particular country. Some systems require only visual and audible inspections or shaking to identify if there are defects or if it has worn beyond the minimum requirements. Other systems, however, require dismantling to check worn out parts or by using testing equipment which have been designed and developed specifically for the task.

In reality it is not practical to open the systems and components for inspection in PMVI (such as measuring thickness of brake lining, etc.) This will not only take a lot of time to inspect one vehicle, but also parts such as seals, lock nuts, and retainer clips and pins which are recommended by manufacturers to be replaced after each dismantling, will have to be replaced.

7.4.3 PMVI Checklist

The most common safety items which are in the PMVI list in most countries include:

- Brake system
- Wheels and tyres
- Steering system
- Headlights
- Windscreen
- Horn and lighting system
- Suspension system
- Towing attachments(trucks)
- Body and structure
- Mirrors
- Seats and seatbelts
- Engine compartment
- Driveline
- Fuel system
- Exhaust system
- Vehicle Identification

The main safety items in any vehicle are brakes, tyres, steering, trailer couplings, headlight and lighting. Other countries have established a list of unlawful modifications which are also included in the checklist. Furthermore, most countries have extended items to be checked on school buses, PSV, and HGV.

The common systems checked in PMVI for environment compliance are:

- Carbon monoxide (CO), hydrocarbons (HC), Nitrogen oxides (NO_x), and particulates in the engine exhaust emission
- Noise

Brake system performance

The braking system is one of several key safety-related items. Catastrophic brake failure, such as sudden air loss in air-brake system for heavy vehicles, may lead to loss of control and the driver's inability to recover. Progressive brake deterioration, such as brake shoe wear without corresponding adjustment, can be even more dangerous because it may appear manageable during normal driving, but may increase the effective reaction time during emergency braking which can precipitate to an accident.

Tyres Condition

The tyres are also key safety-related items. A tyre that is worn out or damaged may fail as a blowout and result in loss of control of the vehicle. The principal indicators of deterioration are tread wear, tread and sidewall damage, and air leakage. When the tread on a tyre is worn down there is not enough groove depth left to handle the water during rain conditions. The water forms a wedge under the tread and lifts the tyre off the pavement (hydroplaning), and it is a potentially dangerous situation because the tyre is no longer in contact with the road and is skimming over the water with no directional stability or traction which can cause to lose control of the vehicle during turning, accelerating or braking. Modern vehicles are fitted with Anti lock brake system (ABS) and electronic stability control (ESC) which may help the driver to recover.

Steering System and wheel alignment

The steering system can fail catastrophically or deteriorate progressively and can increase steering wheel play which will make it harder for the driver to steer. Excessive play is a principal indicator of deteriorating steering system components which may eventually lead to a catastrophic failure. Wheel alignment affects the tyre wear and defect formation, as well as vehicle’s handling performance. Apart from road safety, other aspects as sustainability and environment also benefit from a proper wheel alignment, by reducing the rolling resistance, and therefore fuel consumption.

Trailer Coupling and Fifth Wheel Hitches

The objective of checking trailer coupling and fifth wheel hitch is to prevent accidents due to trailer separation due to improper hitching, or inadequate or damaged equipment. Pintle hooks and ball hitches can uncouple if improperly latched. Hitch mounts could separate due to damage or lack of maintenance. Other components to be checked include the landing gear, the air lines and electric lines which are hooked to the trailer.

Lights and Reflectors

The objective of checking the lights is to reduce the number of accidents due to other drivers’ inability to see (your) vehicle. Lighting system and reflectors enable the other driver to see the vehicle and its movement in time.

Headlights

It is importance that headlights be aimed properly in order for it to perform its best. Lights that are aimed incorrectly will not only perform poorly but may also offend oncoming traffic. Slight variances in filament position can translate to large variances in beam pattern. Headlights and spotlights can become misaligned in many ways. One of the most common is during replacing of the headlights after bodywork repair.

Vehicle Exhaust Emission

Both gasoline and diesel fuel contain mixtures of hydrocarbons, which are compounds that contain both hydrogen and carbon atoms. In a “perfect” engine, oxygen from the air would convert all of the fuel’s hydrogen to water, and carbon to carbon dioxide. In reality the combustion reaction process does not go to completion due to the high speed of the engines, and hence some reactions are not completed before the exhaust process starts and emissions such as carbon monoxide (CO) and hydrocarbons (HC) are emitted to the atmosphere. Furthermore, the combustion reaction produce high temperature and makes nitrogen to react with oxygen to form nitrogen oxides (NOx) which are also release to the atmosphere. In addition, significant amount of HC can be released if the fuel tank cap does not seal the tank properly. Leaking gas caps are a major source of evaporative HC emissions.

Hence both petrol and diesel engine vehicles are equipped with emission control systems that reduce (but do not eliminate) pollutants that can be harmful to the environment or public health. Nitrogen Oxides (NOx) and Hydrocarbons (HC) react in the presence of sunlight to form ground-level ozone or "smog", which can irritate eyes, damage lungs and aggravate respiratory problems. Symptoms include coughing, shortness of breath and difficulty breathing. Hydrocarbons also contribute to global climate change and many are considered hazardous air pollutants. Carbon Monoxide (CO) reduces the flow of oxygen in the bloodstream and is of particular concern to people with heart disease.

In addition to these emissions (which are common to both types of internal combustion engines), diesel vehicles also produce significant amounts of particulate matter, or soot. Significant percentages of the particulates found in diesel exhaust are fine particles, measuring 2.5 microns or less. These solid particles and liquid droplets are so small that they can be inhaled deep into the lungs and cause health problems. Large concentrations of fine particles can be seen as haze, dust, and/or soot, reducing outdoor visibility.

7.4.4 Unlawful Modifications on Vehicles

In some countries modifications can cause a vehicle to fail technical inspection if the modification was not certified by an appointed authority to ensure that it does not affected any safety-related or performance standard of the vehicle. Modifications on engine (normally changing to another type) on their own, are not a subject to a safety related standard. However, if a re-power exceeds certain criteria, it can affect other related safety standards such as brakes. In most cases of changing the engine to a different type involves adjusting the engine mounting brackets and welding to the chassis which will affect the structural requirement of the chassis material. Similarly, fitting a bull bar or any equipment such as a winch might violet the external projections provisions and welding or drilling to the chassis.

In New Zealand, the following modifications require certification from Land Transport Safety Authority (LTSA)[5]:

- An engine that is not of the same type and specification as fitted when the vehicle was manufactured
- An engine modification that significantly increases the engine's horsepower (i.e., in excess of 20%)
- A road wheel that:
    - cannot be identified as being made by a recognised manufacturer, or
    - is not suitable for the vehicle, or
    - is not fitted in accordance with the manufacturer's instructions, or
    - has been modified by heating, drilling, grinding, welding, etc
- A suspension or steering system:
    - with a component that has been bent, cut, plated or heated, or
    - that has a modification kit installed (anti-roll bars, after market springs, lowering kits, etc.), or
    - with a repositioned steering column
- A brake system that has been modified in any way that is not approved by the vehicle manufacturer.
- A body or chassis modification that may affect the strength or rigidity of the vehicle's load bearing structure, e.g.
    - saloon car to convertible or utility, or
    - change in chassis length, or

[5] LTSA New Zealand. (2004). Vehicle Inspection Requirement Manual. Heavy Vehicles Specialist Certification.

  - × addition or removal of cross members, or
  - × alteration of load bearing panels, mountings or brackets especially close to seat belt anchorages

On the other hand, some modifications have been considered to not adversely affect the safety performance of a vehicle, provided they meet the safety standard criteria:

- ✓ Branded wheels and tyres of a different size, provided:
  - ✓ they do not project beyond the line of the original mudguard, and
  - ✓ they lie within 25 mm of the manufacturer's track width, and
  - ✓ the tyre/wheel rolling radius is unchanged
- ✓ Changed road springs and shock absorbers that are direct substitutes for the originals, being mounted in the same way without lowering or raising the vehicle
- ✓ Suspension bushes that are direct substitutes for the originals
- ✓ Any modifications to the original engine model and type for the purpose of increasing its power by no more than 20%
- ✓ After-market steering wheels (not where airbags were originally fitted) providing they do not increase the risk of injury to the vehicle occupants
- ✓ Additional or changed instruments and accessories that are either directly ahead of the steering wheel or do not intrude into an area where they could add or cause injury to a vehicle occupant in a collision
- ✓ Cosmetic changes to the bodywork that are secure and do not reduce driver field of view or increase the risk of injury to pedestrians
- ✓ Glazing or aftermarket window tinting provided they have Visible Light Transmittance (VLT) of more than 35% at the front windows.

Bull bars are widely used throughout the world and have been accepted as an accessory for most types of vehicles, including passenger cars, four-wheel drives, forward control vehicles, etc. However, the continued development of vehicle technology and ongoing improvements in vehicle safety systems have led to a situation whereby a bull bar may interfere with the sophisticated safety systems designed into modern vehicles.

In the guideline for bull bar design published by the Department of Transport in Australia, the main causes of concern are outlined as follows[6]:

- × *Incompatibility with vehicle air bags*: A badly designed bull bar may interfere with the operation of an air bag, making it activate at the wrong time. In the worst case, this can cause significant additional injury to the occupant.
- × *Nullification of crumple zones*: Modern vehicles have crumple zones that protect the occupants by cushioning the impact of a front-end collision as shown in Figure 4. A badly designed bull bar may make the front end of the vehicle significantly more rigid, thereby nullifying the effect of the crumple zones thus causing a more severe impact for the occupants of all vehicles involved.
- × *Incompatibility with other vehicles*: Significant research is being undertaken by vehicle manufacturers towards improving the "crash compatibility" between vehicles involved in vehicle-to-vehicle collisions. A badly designed bull bar can negate these design features, thus increasing the risk of more significant injuries to the occupants of both vehicles.
- × *Danger to pedestrians:* Scientific studies have shown that a badly designed bull bar may greatly increase the risk of injury to a pedestrian hit by a car, even at relatively low speeds.

[6] Vehicle Safety and Standard Branch. Bull bar construction guideline. Circular to Industry. 2010.

Figure 5 shows pedestrian collision scenarios with a good and bad design of a bull bar. In a good design the pedestrian rolls onto the bonnet of the car and the risk of injury is minimised. Whereas, in the bad design the contact point is higher and the pedestrian is pushed forward and bent around the bull bar which has a much higher risk of spinal, pelvic and head injuries.

Figure 4: Crumple zone absorb collision energy during accident to reduce occupants deceleration force (Source: www.volkswagen.co.uk)

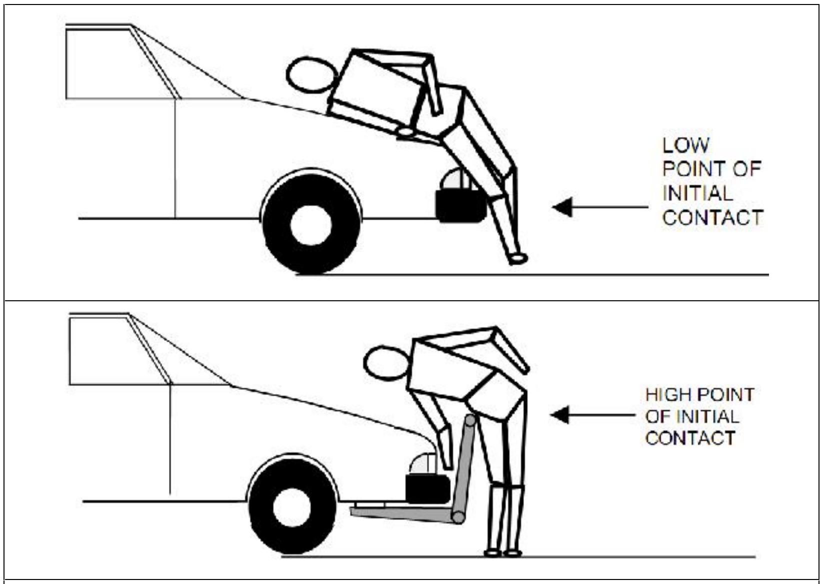

Figure 5: Effect of contact point to pedestrian injury risk for good (above) and bad (below) design of bull bar (Source: Vehicle Safety and Standard Branch. Bull bar construction guideline. Circular to Industry. 2010)

Further regulations relevant to bull bar design prohibit dangerous projections, sharp corners and obstructions to lighting and field of view. Any bull bar, together with any attachments, must not reduce a driver's ability to safely drive the vehicle to which it is attached. When sitting in the driver's seat with the seat located at its rearmost position, it shall be possible to see either the surface of the road, 11 metres in front of the driver's eye or the front edge of the original body when looking across the top of the bull bar as shown in Figure.6. Figures 7 and 8 show good and bad design of bull bars.

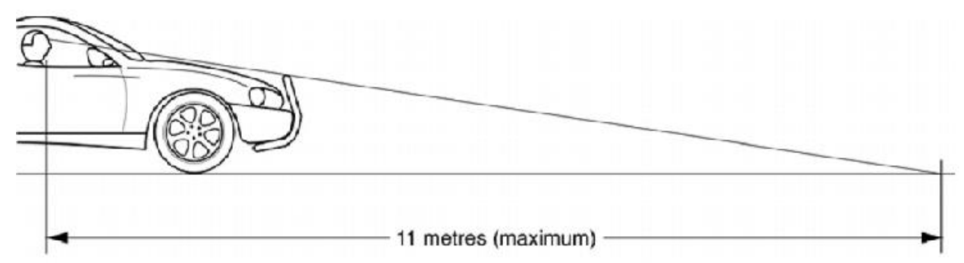

Figure 6: Field of view for a good design of bull bar (Source: Vehicle Safety and Standard Branch. Bull bar construction guideline. Circular to Industry. 2010)

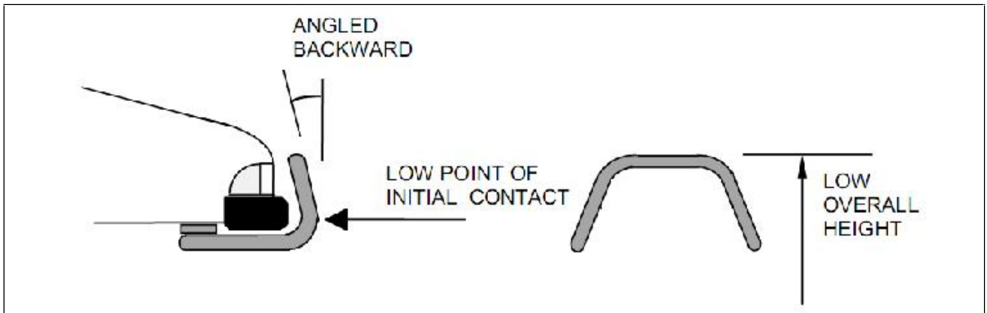

Figure 7: Good design of bull bar (Source: Vehicle Safety and Standard Branch. Bull bar construction guideline. Circular to Industry. 2010)

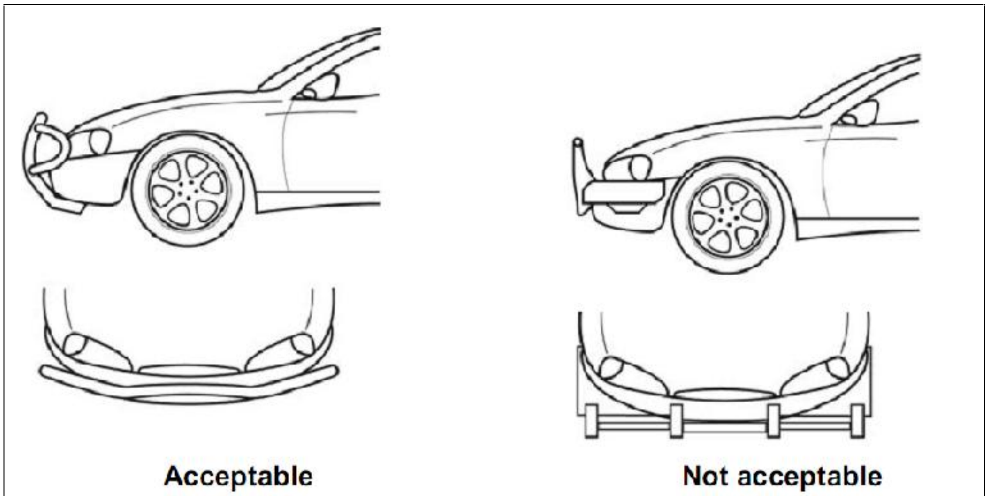

Figure 8: Good and bad design of bull bars (Source: Vehicle Safety and Standard Branch. Bull bar construction guideline. Circular to Industry. 2010)

7.5 Current Situation around the World

The World Forum for Harmonization of Vehicle Regulations (WP.29)[7] offers a unique framework for globally harmonized regulations on vehicles. The benefits of such harmonized regulations are tangible for road safety, environmental protection and trade. WP.29 is a permanent working party created more than 50 years ago in the institutional framework of the United Nations with a specific mandate and rules of procedures. It works as a global forum allowing open discussions on motor

[7] World Forum for Harmonization of Vehicle Regulations (WP.29)

vehicle regulations. Any country member of the United Nations, and any regional economic integration organization set up by country members of the United Nations, may participate fully in the activities of the World Forum and may become a contracting party to the Agreements administered by WP.29.

The World Forum for Harmonization of Vehicle Regulations (WP.29) administers three international agreements on motor vehicles: the 1958 Agreement and the 1998 Agreement on the construction of vehicles, and the 1997 Agreement on periodical technical inspections. WP.29 ensures the consistency between the three types of regulations or rules produced by the three agreements.

The 1997 Agreement on periodical technical inspection of vehicles provides that Contracting Parties applying one Rule annexed to the agreement shall reciprocally recognize the international technical inspection certificate issued by the inspection authority of another Contracting Party. The Agreement has currently one Rule on pollutant emissions and 11 Contracting parties. A draft Rule on vehicle safety is under consideration by WP.29.

Most countries in the World have their own rules and processes of ensuring the roadworthiness of vehicles that operate within their countries. This type of regulation serves different purposes in different countries, but mostly as a safeguard against accidents from reduced performance or failure of a system or component, and environmental pollution caused by vehicles exhausts emissions and noise.

Hence, most countries have two types of periodic inspections; the first inspection is a vehicle safety test and is designed to check if the vehicle systems and components which are directly related to the safety of driver, passengers, and other road users are in good condition and safe mechanically for the vehicle to be on the road. The second inspection, emission test, checks the level of certain pollutants which are emitted with the exhaust gases. The two inspections are normally conducted together and can have the same frequency of inspection.

7.5.1 European Union (EU)

The EU roadworthiness testing policy was framed over twenty-five years ago (framework Directive 77/143/EEC) and originally only included the necessary testing activities for buses, taxis and ambulances. Different modifications expanded the Directive's scope to include the inspection of cars and light vans and also detailed technical provisions were adopted, in particular concerning the testing of vehicle brakes and exhaust emissions. The framework Directive and all its amendments were consolidated within Directive 96/96/EC, which has been adapted several times (Directives 1999/52/EC, 2001/9/EC, 2001/11/EC, regulation (EC) No. 1882/2003 & directive 2003/27/EC). The last adaptation was Directive 2003/27/EU to require more stringent emission testing for 'Euro3' petrol and 'Euro 4' diesel driven vehicles.

Directive 2000/30/EC also stipulates that heavy commercial vehicles shall be subject to 'targeted' roadside inspections. This has been adapted by Directive 2003/26/EU to bring its technical inspection standard in line with the subsequent amendments of Directive 96/96/EC. Nevertheless much of the aspects related to the real application of the roadworthiness enforcement methods are not specifically mentioned in the directive or are described only as minimum requirements which each country may increase.

European Directives are the minimum required levels for all the European countries. Two main roadworthiness enforcement methods are clearly described, PMVI (referred in Europe as periodic technical inspection, PTI) for most of the vehicle categories reinforced with roadside inspection for some specific ones. This means that in all European countries there is a consolidated vehicle inspection centre network able, if needed, to extend the roadworthiness tests to other vehicle categories.

Although all Member States must comply with the periodic testing requirements of Directive 96/96/EC, arrangements in each country vary significantly. On one extreme, vehicles in Austria, Holland, Great Britain (for cars and light goods vehicles) and Ireland (for heavy goods vehicles) are inspected in private sector testing organizations working in competition (decentralised testing) and these organizations are permitted to repair and trade in the vehicles they inspect. At the other extreme, countries such as Luxembourg and Sweden have single independent testing organizations. Vehicles are inspected in a smaller number of high volume facilities (centralized testing) which are not permitted to repair or trade in vehicles. Practices for roadside inspection of heavy vehicles also vary significantly. Some countries use the Police, who are sometimes specialists and some not, while others have special non-police agencies that use specially trained inspectors.

In 2007, an international consortium led by Motor Vehicle Inspection Committee (CITA) commissioned a study[8] (AUTOFORE) to EU. The purpose of the AUTOFORE project is to recommend future improvements in roadworthiness enforcement in the European Union. The following are highlights related to PMVI from the study:

On Organization

- Around 80% of the institutions conducting PMVI in EU countries are private organizations controlled by the governmental body.
- 65% of the EU countries which use private companies do not allow them to do any other vehicle-related business such as garage, or selling spare parts
- In few countries (Latvia, Hungary and Greece) the company conducting inspection is paid by the government for the job, while in the rest of the EU countries all inspection costs are covered by the vehicle owner, and in some countries the inspecting company pays the government a fee. In Finland, Spain, and Denmark the fee is proportional to the number of vehicles inspected.
- In few countries (Latvia, Hungary and Greece) the inspection facilities are owned by the government and are rented to a private company conducting PMVI.
- In most EU countries the inspection fee is fixed by the government

On Legislation

- In most countries it is mandatory to repair the fault if the vehicle failed the PMVI test in any category. In general it is forbidden for the car to circulate without the valid inspection certificate.
- About 52% of the countries are considering changes in the roadworthiness enforcement methods.
- Only in 20% of the countries the inspection centres can conduct PMVI on a vehicle from another EU country through bilateral agreements.
- 88% of countries which do not conduct PMVI after a vehicle has been modified have shown interest to include that provision.
- 80% of countries which not conduct PMVI after a vehicle has involved in an accident have shown interest to include that provision.

[8] AUTOFORE. (2007). Study on the Future Options for Roadworthiness Enforcement in the European Union. CITA.

- 20% of countries which do not conduct PMVI during change of ownership have shown interest to include that provision.
- About 80% of the countries consider that its first inspection is done after the correct number of years and 73% consider the inspection frequency as correct.
- All EU countries show clear acceptance and recognition to a final reciprocal recognition of PMVI conducted in other EU countries.
- Almost all countries recognize that Directive 96/96/EC is not a correct frame in order to arrive to this object.

On Mandatory Vehicles for PMVI

- In all EU countries it is mandatory to conduct PMVI for private cars, passenger car (<8 people), passenger car (>8 people), goods vehicles (<3500 kg), goods vehicles (>3500 kg), trailer (>3500 kg)
- In 80% of the EU countries it is mandatory to conduct PMVI for trailers (<3500 kg)
- In 48% of the UE countries it is mandatory to conduct PMVI for all agricultural tractors
- In 61% of the UE countries it is mandatory to conduct PMVI for motorcycles
- In 68% of the UE countries it is mandatory to conduct PMVI for all Caravans

On Compliance and Enforcement Means

- 55% of the EU countries use stickers/decals on the windscreen or at the number plate.
- Some countries do not use stickers/decals (Finland, Denmark, Netherlands and Sweden), instead the vehicles which pass PMVI are entered in a digital database linked to vehicle registration database which are easily accessed by law enforcement officers.
- In some EU countries (32%) the certificate is required for any traffic legal/official matters

On Failing PMVI

- In 50% of the countries failures of PMVI are fined.
- In the rest of countries (except Slovenia and Netherlands) the vehicle loses its right to be driven on public roads. This prohibition is applied in different ways, in Denmark the number plate and in Italy the registration document is withdrawn.

7.5.2 Europe

Before January 2008, the Netherlands used a stricter schedule of inspecting vehicles 3 years after first registration, and must be inspected annually thereafter (3-1-1-1). Since 2008 the standard schedule is 4-2-2-1-1. Vehicles more than 8 years old must therefore still be inspected annually like in the old system. However, vehicles running on LPG or diesel continued with the old system (3-1-1-1). The reason for the different inspection schedule is the fact that these vehicles tend to have a higher annual mileage than vehicles with a petrol engine. Furthermore, trucks and trailers (>3500 kg), and buses are required to be inspected annually since first registration (1-1-1-1).

The Dutch test checks various points of safety and reliability of the vehicle, in accordance with the European Directive 96/96/EC. Certification is digital and there are no visible stickers or decals on the vehicle. The inspections are conducted by around 9,000 private garages accredited by the government authority which also monitors the activities of these garages through random checks.

In **Austria:**

- Light private vehicles must be inspected after 3 years, then after a further two years and every year after that (the 3-2-1-1 scheme).
- PSV, HGV and motorcycle must undergo inspection every year (1-1-1-1 scheme).

- The charge for a periodical inspection is determined by market forces, and lies between 25 and 35 euro.
- The accreditation and supervision of inspection garages is performed by the Government agency.
- The vehicle inspection covers both aspects of vehicle safety and emissions standards.

Finland has a long tradition of vehicle inspection, dating back to year 1917. Vehicle inspection was initially carried by cities and provincial inspectors but starting from year 1968 the inspection moved to national governing body of road vehicle administration. In addition:

- Vehicle inspection is conducted by certified private garages where the government keeps track of inspection quality.
- PMVI is required for all passenger cars, vans, trucks, and for trailers with maximum structural weight of more than 750 kilograms.
- Inspection interval depends on vehicle class and usage.
- For privately-used passenger car the scheme is 3-2-1-1
- For commercial vehicles the scheme is 3-1-1-1.
- For trailers (>3500 kg) the scheme is 2-2-2-2
- Motorcycles, mopeds, tractors and snowmobiles do not have annual inspections

In **Bulgaria**:

- The PMVI scheme is 3-2-2-2 for light private vehicles
- Commercial vehicle scheme is 1-1-1-1
- The administrative charge for the periodical inspection is 15 Euro.
- Garages and inspectors must take an admission examination before they can carry out periodical inspections.
- Garages are subject to regular checks on their inspection activity by the government.
- The inspection of commercial vehicles is carried out by private garages admitted and monitored by the government authority
- Motorcycles are not inspected periodically.
- All inspections are carried out in accordance with Directive 96/96/EC and the latest amendments.

In **Estonia**:

- PMVI is conducted by private stations and the government acts at supervisory role.
- 8.43% of the vehicles did not pass the PTI at the first attempt.
- In 2008 the average age of vehicles at PMVI PTI was 13 years.
- The PMVI scheme is 3-2-2-2.
- PSV of more than 10 years old the scheme is 1-1-1-1
- Vehicles conducted PMVI from other EC countries are accepted without inspection.

The **Germany** PMVI framework is as follows:

- Private passenger car scheme is 3-2-2-2
- However, heavy goods vehicles need to be inspected every year irrespective of their age. (1-1-1-1)
- The inspections are conducted at inspection centres operated by private companies which are supervised by the government agency.
- Mobile inspection centres are common.
- A sticker is placed on the rear number plate showing the date the certificate expires.

Vehicle testing has been mandatory in **France** since 1992.

- The first inspection is carried out after four years then subsequently after every two years, (4-2-2-2)
- A blue and white decal is affixed inside the windscreen to indicate when the next PMVI is due.

In **Gibraltar** the government department of transport is responsible for carrying out roadworthiness testing. The framework for PMVI is as follows:

- Private motor vehicles: 4-2-2-2
- HGV, PSV and taxis: 1-1-1-1
- All testing facilities are owned by the government
- The fees for a roadworthiness test ranges between Pound sterling 15 – 45 depending on the class of vehicle

In **Hungary**:

- PMVI and other vehicle inspections are conducted by both the government authority, and private certified garages.
- The scheme for light private vehicles is 4-2-2-2
- Commercial vehicles scheme is 1-1-1-1
- Agricultural vehicles the scheme is 5-5-5-5

In **Iceland:**

- Technical inspections, both type approvals and regular inspections, are carried out by private companies.
- The companies are supervised by the government department
- Light private vehicle scheme for vehicles constructed before 2009 is 3-1-1-1
- For vehicles constructed after 2009 is 4-2-2-1
- Antic vehicles and trailers scheme is 2-2-2-2
- HGV and PSV are inspected annually, (1-1-1-1)
- Stickers applied to the number plate are used for enforcement.

In **Norway:**

- PMVI are carried out by private centres under government supervision
- The centres are not allowed to carry out repair works
- PMVI scheme for private light vehicles is 4-2-2-2
- Taxis, ambulances, HGV and PSV: 1-1-1-1
- PMVI test results are rated depending on how dangerous they are, with the grades of 1, 2 and 3.
- If a vehicle has errors marked with 1, or no errors at all, the car will pass immediately.
- Errors marked with 2 will require a re-inspection, but the owner can use the vehicle until the inspection deadline.
- Errors marked with 3 are serious errors, and prohibits the user from using the car until those errors are repaired.
- The PMVI cost is usually between 400-700 Norwegian Kroner.

In **Slovenia:**

- The schedule for PMVI for light private cars is 4-2-2-1-1.
- Inspections are conducted by private companies but supervised by a government authority
- The fee for the periodical technical inspection for passenger cars is Euro 30.

In **Spain**:

- Vehicle inspections are conducted by private companies, but are supervised by the Ministry of Industry.
- The schedule for PMVI for private light vehicles is 4-2-2-2-1-1
- For motorcycles the schedule is 5-2-2-2-
- HGV <10 years old 1-1-1-1, and for trucks older than 10 years it is every 6 months.
- The enforcement is by means of a sticker on the windscreen.

In **Sweden:**

- The PMVI schedule for light private vehicles is 3-2-1-1
- Vehicle inspections are conducted by private companies, but are supervised by the government department.

In **Poland:**

- Light private vehicles and motorcycles inspection scheme is 3-2-1-1
- Scheme for PSV and HGV > 3500 kg is 1-1-1-1
- For agricultural tractors and trailers, and mopeds 3-2-2-2
- Inspection conducted by authorized private companies and supervised by the government
- An additional, separate inspection is required for cars that are running on Liquefied Petroleum Gas (LPG).
- Pass of inspection is confirmed in registration certificate. When the vehicle does not pass the inspection, the owner is requested to rectify and submit the vehicle for re-inspection.

In **Ireland** periodic roadworthiness tests referred to as the National Car Test (NCT) were introduced in 2000. The framework is as follows:

- Private vehicles scheme is 4-2-2-2
- A private company - National Car testing Service Ltd (NCTS) conducts inspections under the supervision of the Road Safety Authority. There are 43 testing centres across the country.
- A sticker which must be displayed on the windscreen is issued to cars that pass the test.
- An average of 685,000 vehicles is tested each year with approximately 95% of vehicles passing the test.

In **Italy**:

- Light private vehicles must be inspected 4 years after the date of the first registration, and subsequently inspected every 2 years, (4-2-2-2).
- Commercial vehicles schedule is 1-1-1-1.
- Vehicles that are intended for use in city centres must be subjected to an annual emissions test.
- The vehicles are inspected in private garages monitored by the government authority.
- These garages are also allowed to carry out repairs.
- The administrative charges for the periodical inspection amount to 30 Euro.
- Garages and inspectors must take an admission examination before they can carry out periodical inspections.
- Garages are subject to regular checks by the government authority on their inspection activity.
- Since 2001, motorcycles and mopeds are also inspected periodically.

In **Latvia** all vehicles, regardless of age and class, are inspected annually (1-1-1-1) by private garages supervised by government authority.

In **Lithuania**:

- PMVI are conducted by private companies and supervised by State Road Transport Inspectorate (SRTI) under the Ministry of Transport and Communications.
- Stickers on number plates are used for enforcement
- Light private vehicles scheme is 3-2-2-2
- Other class of vehicle have a different tighter schedule of inspection.

In **Luxembourg**:

- The scheme for private light vehicles is 3.5-1-1-1
- Buses and lorries are inspected every 6 months (0.5-0.5-0.5-0.5)
- Agricultural tractors 2-2-2-2.
- The inspections are performed by private company supervised by government authority
- The kilometre readings are also recorded during the tests.
- Vehicles which pass the test are recorded in National database and no stickers or other exterior indications are applied to the vehicles to show they have been inspected.
- There is however an inspection certificate delivered, which is part of the mandatory documents to be in the vehicle

In **Switzerland**:

- Technical inspections for motorcycles is also compulsory, and the schedule of inspection is 4-3-2-2
- The scheme for GHV, trailers (>3500 kg) and PSV is 1-1-1-1
- Private light vehicles schedule is same as for motorcycles (4-3-2-2)
- The charge for the technical inspection varies between CHF 35 and CHF 60.

In **United Kingdom**:

- Vehicle and Operator Services Agency (VOSA), is responsible for type approval inspections prior to registration, and periodic testing.
- VOSA supervise garages which are approved to conduct inspections
- Light private vehicles schedule of inspection is 3-1-1-1
- Heavy Goods Vehicles, trailers, , coaches and buses schedule is 1-1-1-1
- Agricultural tractors are exempted from PMVI
- VOSA has own centre and conducts special inspections and tests of commercial vehicles, Lorries and buses.

7.5.3 America

The United States and Canada PMVI system are somewhat similar where States or Provinces are free to decide PMVI as well as its specifications. The states can decide to conduct emission tests periodically in accordance to the respective Clean Air Acts, while safety inspections are done annually or bi-annually depending on the state. These inspections are done by the Department of Motor Vehicles (DMV) or private testing centres duly approved by the state's Department of Transportation.

Canada has integrated carrier responsibility, risk management, periodic inspection and roadside enforcement through their nationwide National Safety Code (NSC). The NSC consists of 16 safety standards. The ones relevant to vehicle roadworthiness inspection are:

- *Preventive Maintenance:* The objective of this standard is to ensure all commercial vehicles are subject to a systematic regular preventive maintenance program by the fleet operator.
- *Periodic Motor Vehicle Inspection (PMVI) requirements*: Annual or semi-annual inspections are undertaken by government approved inspectors.

- *On-road Inspections:* This standard is linked to the North America-wide roadside inspection system that is administered by the Commercial Vehicle Safety Alliance (CVSA). Approximately 3million inspections are undertaken each year throughout North America using the CVSA standard inspection program.
- *Trip inspections:* This Standard specifies the minimum requirements for the daily inspection undertaken by the driver or the fleet operator.
- *Safety Rating:* This standard recognizes that safety resides, first and foremost, with the management of the fleet operator. It specifies the *safety rating scheme* that each jurisdiction must adopt as a means of rating the safety performance of each of the fleet operators in that jurisdiction.
- *Facility Audits:* Facility audits are undertaken at the fleet operator premises and are carried out by Government inspectors or by third party auditors.

In implementing the National Safety Code, Ontario has taken a four-pronged approach involving: Safety Standards; Detection; Deterrents; Incentives. The programs that have been implemented are aimed at *identifying both* safe *and unsafe operators* so enforcement resources can be used effectively. Strong deterrents have been introduced as a means of encouraging compliance, and meaningful incentives have been introduced as a means of making investment in safety pay. This includes: pre-clearance at borders for safe operators; fee exemptions; 'excellent' and 'unsatisfactory' ratings for carriers published on the internet (www.carriersafetyrating.com); and reduced insurance premiums. Ontario now has the lowest number of fatalities per 10,000 drivers in North America. Truck related fatalities are historically at their lowest level from 300 in 1996 to 99 in 2004

The approach taken in the **United States** is very similar to that used in Canada. Both countries participate in the Commercial Vehicle Safety Alliance (CVSA) as a means of ensuring uniform inspection and enforcement procedures, uniform data systems and reciprocity of enforcement between the various state, province and territory jurisdictions. Implementation in the US was made possible by linking the CVSA requirements and federal regulations to federal funding to the states through the Motor Carrier Safety Assistance Program (MCSAP). Before MCSAP the Federal Highways Administration (FHWA) conducted a limited number of roadside driver and vehicle inspections but the States have now assumed that role.

In PMVI, the pass-fail criteria in the US are widely available to ensure transparency. Vehicles that fail an inspection are placed out of service until suitable repairs have been undertaken. An Inspection Selection System (ISS) that assists officers with the selection of vehicles for roadside inspection has now been introduced throughout the US fleet operators (rather than individual vehicles) that are now targeted through ISS are the ones with a poor safety record having an unsatisfactory safety rating or a higher than average out-of-service rate from previous inspections or very few or no roadside inspections in the previous 2 years.

In **Brazil**:

- Vehicles undergo safety inspection as well as emission levels and noise levels.
- The government accredits private companies to conduct these tests.
- Vehicles which pass inspection are issued with an electronic stamp associated with the vehicle's license plate, which is used for law enforcement officers to identify vehicles that are due for annual inspection.

7.5.4 Asia and Oceania

In **Japan:**

- The PMVI schedule for light private vehicles is 3-2-2-2
- Commercial vehicles schedule is 1-1-1-1
- Vehicles with engines less than 250 cc are exempted from PMVI.
- In Japan inspectors also check to determine if a vehicle has been illegally modified and vehicles that have been modified are subject to a separate inspection.
- Private garages conducts PMVI supervised by a government authority

In **Singapore:**

- Motorcycles and mopeds are also required to take a safety and emissions inspection. The inspection schedule is 3-2-2-2-1-1
- Buses, taxicabs, commercial vehicles, and trailers schedule is 1-1-1-1 and after 10 years every six months.

The government of **Thailand** has established regulations for vehicle inspection requiring all motorized vehicles to submit to tests as follows:

- The inspection schedule for private passenger cars, pickups and vans is 7-1-1-1
- Motorcycle mandatory inspection schedule is 5-1-1-1
- The government has authorized the task of inspection to private inspection centers.
- The inspection fees are; motorcycle (60 baht = 1.5 US$); light vehicle under 1600 kg (150 baht = 3.75 US$); and light vehicle over 1,600 kg (250 baht = 6.25 US$)

Each state or territory in **Australia** has the authority to set its own regulation pertaining PMVI. Vehicles in the following categories must be inspected by an authorised vehicle examiner:

- When registering a brand new vehicle for the first time (usually the dealer does this prior to delivery)
- Upon transfer of ownership if the vehicle is over six years old
- Before transferring a registration from interstate regardless of the vehicle's age
- Before re-registering a vehicle if the previous registration was cancelled or expired more than 12 months ago
- If a defect notice is issued

Once a vehicle is registered, it generally will not need to be inspected again if it remains with the same owner. However, the Australian Capital Territory has a random inspection scheme, vehicles can be spot checked whether occupied or not. If faults are discovered, a defect notice can be issued in which case the owner will be required to have repairs made, and obtain a full inspection from a licensed examiner with 14 days to clear the notice. If egregious safety violations are found, the vehicle's registration can be suspended on the spot and the operator will need to have the vehicle towed.

In South Australia, The majority of privately owned vehicles do not need to be inspected to be legally driven or have their interstate registration transferred to South Australia. Vehicles with certain types of modifications, specifically engine, chassis, wheel-base, seating capacity, brakes, steering, or suspension modifications require a certificate of roadworthiness prior to registration (or as a condition of continued registration). Additionally, vehicles cited for safety faults by the police, vehicles that were previously written off and/or salvaged., rebuilt, self-constructed or homemade vehicles (such as classic car restorations or kit cars), vehicles transferred from interstate that are over seven years old and weigh more than 4.5 tonnes (10,000 lbs), vehicles that were transferred from left-hand to right-hand drive, buses with a seating capacity of 13 persons or more are all required certificate of road worthiness before re-registration.

There are a number of accreditation schemes in Australia that are aimed at encouraging heavy vehicle transport operators to adopt good safety management practices. One of the scheme is Trucksafe, which was set up in the early 1990's and is owned by the Australian Trucking Association, In the mid 1990's the Australian authorities recognized the potential of voluntary accreditation as an alternative to conventional methods of enforcement and initiated the development of the National Heavy Vehicle Accreditation Scheme (NHVAS). The aim of the NHVAS is to increase transport efficiency through reducing the cost of compliance.

With the maintenance management module, operators who are accredited are not required to undertake mandatory PMVI and are subjected to less on-road enforcement. The maintenance management module includes a *Vehicle* Roadworthiness *Guideline* that gives practical information about wear, damage or change to the more important systems in a vehicle. The Australian Chain of Responsibility principle recognizes that there is a wide ranging chain of responsible persons involved in the transport network. It targets not only drivers and operators of heavy vehicles, but also "off road" parties who through either their actions or inactions have influenced the vehicle and/driver's ability to comply with road transport laws.

The **New Zealand** Transport Agency requires most vehicles to maintain a Warrant of Fitness (WOF) through PMVI from licensed inspectors. The framework for PMVI is as follows:

- Cars and light vehicles less than six years must be inspected every year (1-1-1-1-1-1). Whereas older than 6 years require inspection after every 6 months.
- Heavy vehicles and passenger service vehicles (PSV) requires a six-monthly inspection regardless of the age of the vehicle.
- Each vehicle using public roads must display a WOF sticker in the top right corner of its windscreen (as viewed from inside the vehicle).
- The sticker indicates when the vehicle passed its last WOF inspection and shows when the next inspection is due.

7.5.5 Summary of World Review

The review of PMVI practices around the world reveals that there is an extensive variety in the frequency of PMVI. According to EU directive 77/143 EEC the maximum PMVI frequency recommended for EU countries is 4-2-2-2. However, some countries specify stricter schedules, which is OK. Latvia has the most severe frequency with an inspection every year, only New Zealand has a similar strictness from the other side of the world. In all countries reviewed PSV, HGV, and taxis have stricter PMVI schedule than private light vehicles and in some countries the frequency changes after a vehicle has reached a certain age after first registration.

With regard to the organisations by which the inspections are conducted, it appears that many countries use a model in which the *inspections are conducted by private inspection centres that are supervised by a governmental authority*. In many countries including Austria, Italy, the Netherlands and the United Kingdom, commercial garages perform the technical inspection for certain classes of vehicles, but are closely monitored by the government. Only in Gibraltar and Latvia does the licensing authority itself conduct the periodic inspections.

In many countries around the world, a sticker must be attached to the vehicle in order to show that the vehicle has passed the periodic technical inspection. These stickers are placed either on the windscreen or on the number plate, according to the rules in the specific country.

In most countries new vehicles are exempted from PMVI until the vehicle has reached a certain age. In many countries this regulation does not apply to commercial vehicles (PSV, taxis, and HGV). In other countries vehicle inspection is mandatory when transferring the vehicle title to a new owner. Most countries also give exemption to vehicles belonging to government and government institutions military, and fire brigade trucks.

In most countries the owner is given time to repair the vehicles which fail the inspection and come back for re-inspection after a specified time. In most countries there is a fee for re-inspection. The most common safety systems and components which are in the PMVI list in most countries include:

- Brake system
- Wheels and tyres
- Steering system
- Headlights
- Windscreen
- Horn and lighting system
- Suspension system
- Towing attachments(trucks)
- Body and structure
- Mirrors
- Seats and seatbelts
- Fuel system
- Exhaust system
- Vehicle Identification

7.6 Review of Proposed SADC-COMESA-EAC Standard

In 1996, SADC member states signed a protocol[9] to harmonize and enhance the overall quality of road traffic in the region with the emphasis on promoting acceptable levels of safety, security, order, discipline and mobility on the roads and protecting the environment and road infrastructure. The following are included in the protocol with respect to vehicle safety standards and roadworthiness:

- Member states agreed to ensure acceptable harmonised levels of vehicle fitness in the Region. Member States shall develop and implement a harmonised vehicle fitness system which shall entail harmonised standards in respect of:
  - ✓ -vehicle testing, including vehicle categories to be tested, the frequency of testing and testing procedures;
  - ✓ -testing stations and testing equipment, including certification, inspection and calibration;
  - ✓ -vehicle examiners, including training and certification; and
  - ✓ -vehicle fitness standards.
- Member States shall require roadworthy certification and/or vehicle fitness certification in respect of all vehicles in their territories as proof that vehicles have undergone the required tests.
- Member States shall develop a common format for roadworthy certification and vehicle fitness certification.
- A Member State shall recognise the roadworthy certification and/or vehicle fitness certification issued in another Member State in respect of a vehicle registered in such State

[9] SADC (1996) SADC Protocol on transport, Communications and Meteorology, SADC Secretariat, Gaborone.

for the purposes of the free movement of such vehicle within its territory: Provided that a Member State shall retain the right of scrutiny to prevent the use of falsified documentation.

- Member States agree that, with the exception of the Republic of Angola, they shall phase out the registration of left-hand vehicles; provided that left-hand vehicles registered prior to the termination of registration of such vehicles shall be permitted to continue operating thereafter.
- Member States shall adopt and implement harmonised regulations on vehicle safety and equipment on or in respect of vehicles, as well as markings on vehicles, and for this purpose, Member States who have not yet acceded to the UN Agreement Concerning the Adoption of Uniform Conditions of Approval and Reciprocal Recognition of Approval for Motor Vehicle Equipment and Parts, 1958, shall consider acceding thereto.
- Member States shall develop maximum or minimum standards, as the case may be, in respect of equipment required on or in respect of vehicles.

The drive towards harmonization of motor vehicle overload control in the EAC, COMESA and SADC has been underscored in several regional agreements developed by the same regional communities. Among many programmes for tripartite harmonization are vehicle safety standards and environment compliance. The standard and specifications for vehicle roadworthiness for the SADC region has been under development for the past few years. Initially the South African standard for testing vehicles was used as a basis for developing the SADC standard. The draft standard for SADC countries which was prepared by the SADC Panel of Expert on Road Transport was upgraded as a tripartite draft standard in 2009[10]. In the draft the recommended standard lists of items to be inspected is shown in Table 1.

Table 1: Items to be checked for all vehicles

Items to be Checked for All Vehicles	
1. Information and registration details	32. Hand levers controlling mechanical braking systems (inspection in vehicle)
2. Smoke emission	33. Service brake pedal (inspection in vehicle)
3. Road wheels and hubs	34 Service brake operation (inspection in vehicle)
4. Size and type of tyres	35 Hand-operated air brake valves (inspection in vehicle)
5. Condition of tyres	36 Chassis or frame
6. Bumper bars, protective devices (bull bars), bonnets, roof carriers and similar fittings	37 Electrical wiring and equipment
7. Wheel flaps	38 Engine
8. Trailer coupling on drawing vehicle	39 Engine and transmission mountings
9. Coupling on trailer	40 Drive train (motor cycles/tricycles/quadrucycles only)
10. Trailer landing legs	41 Oil leaks
11. Rear under run protection devices	42 Fuel system and fuel tank
12. Mudguards	43 Suspension units
13. Cab mounting	44 Shock absorbers
14. Load body or side car	45 Stub axles, wheel bearings, control arms and kingpins
15. Motor cycle/tricycle/quadrucycle fittings	46 Stabilizers and anti-roll bars
16. Safety belts (restraints)	47 Steering mechanism

[10] COMESA/EAC/SADC (2010). The Testing of Vehicles for Roadworthiness.

17. Doors	48 Power steering
18. Floor and steps	49 Drive train (all vehicles)
19. Seats	50 Wheel alignment
20. Mirrors	51 Braking system
21. View to front and sides	52 Trailer parking brake
22. Windows and windscreen	53 Brakes and braking performance (motor cycles/tricycles/quadrucycles only)
23. Windscreen wipers (if a windscreen is fitted)	54 Braking performance (light and heavy vehicles)
24. Speedometer	55 Dimensions
25. Hooter or audible warning device	56 Lamps and lighting
26. Driving controls	57 Retro-reflectors
27. Steering wheel	58 Use of retro-reflective material
28. Steering column	59 Flasher-type direction indicators
29. Handlebars and steering (motor cycles/tricycles/quadrucycles only)	60 Twist-locks for securing containers
30. Air or vacuum warning device of braking systems (where applicable)	61 Load containment
31. Braking systems — Build-up of air pressure or vacuum (where applicable)	

Additions to be Checked for Buses	Additions to be Checked for Mini-Buses
1. Hand levers controlling mechanical braking systems (inspection in vehicle)	1 Sides and roof
2. Service brake pedal (inspection in vehicle)	2 Entrances and exits
3. Service brake operation (inspection in vehicle)	3 Seats
4. Hand-operated air brake valves (inspection in vehicle)	4 Windows and windscreen
5. Chassis or frame	5 Number of persons that may be carried
6. Electrical wiring and equipment	6 Inspection record for minibuses
7. Engine	
8. Engine and transmission mountings	
9. Drive train	
10. Oil leaks	
11. Fuel system and fuel tank	

7.7 Current Situation in EAC Countries

The common causes of traffic accidents in EAC countries are[11]:

- Human error which accounts for about 80% of the road traffic crashes.
- Defective vehicle condition which accounts for about 10%
- Road condition which also accounts for about 5%
- Environment factors which account for about 5%

[11] *WHO Global Status Report on Road Safety, 2009*

Table 2: Number of Vehicles and Traffics accident deaths recorded in East African Countries (*Source: WHO Global Status Report on Road Safety, 2009*)

Country	Population (2007)	No of Registered Vehicles	Reported No of Traffic Deaths	Estimated Traffic Deaths/100,000 Population
Burundi	8,508,232	59,486	63	23.4
Kenya	37,537,716	1,004,243	3,760	34.4
Rwanda	9,724,577	61,000	308	31.6
Uganda	30,883,805	363,658	2,838	24.7
Tanzania	40,453,513	577,949	2,595	34.3

In all EAC countries vehicle safety standards are captured in respective Road Transport and Road Traffic Acts. Through the Acts regulations and provisions were derived on vehicle safety standard requirements and inspections that the authorities have to observe during the initial registration, road licensing, and operators licensing. The mandatory vehicle inspection exists for most of the EAC partner states. However, the degree of implementation and enforcement differ from country to country. On the other hand, corruption and other malpractices have been a common phenomenon. In general the challenges facing most EAC countries are:

- Inadequate *infrastructure and facilities* to conduct technical inspections. Only Rwanda has one modern facility which started operation in 2008 (Figure 9). The vehicle testing centre at Ramera can conduct an average of 200 PMVI per day. The facility has 3 inspection lines (two for trucks and busses and one for private cars)
- Lack of *equipment and qualified manpower* to conduct technical inspection. Only the vehicle testing center at Ramer (Rwanda) has modern Germany technology equipment for vehicle safety and environment compliance inspection purposes.
- Inadequate *enforcement* of mandatory technical inspections. In most EAC countries there are regulations for mandatory PMVI, after the vehicle is involved in road accident, and when the vehicle has been modified. However, due to scattering of road safety and road licensing, and operator licensing activities in various government departments and authorities it becomes difficult to have effective enforcement of the regulations. For example in Tanzania, the police conducts technical inspection, SUMATRA (Surface and Marine Transport Regulatory Authority) deals with license for operators, and Tanzania Revenue Authority issue road license to operators and private cars. The procedure is not computerized and there is no easy checking on operator compliance with other authorities.
- *Malpractices and corruption* are rampant and hamper the effectiveness of the little efforts that the governments are trying to institute

Figure 9: Vehicle inspection centre at Kigali Rwanda (L), and roller brake tester display at the same centre.

Table 3 shows the status of PMVI frameworks in EAC countries. Table 4 shows the items for inspection in the regulations.

Table 3: Current status of vehicle inspection framework in EAC countries

	Framework Components	**Tanzania and Zanzibar**	**Kenya**	**Uganda**	**Rwanda**	**Burundi**
1	Regulations on vehicle safety	Yes	Yes	Yes	Yes	Yes
2	Legal instrument for PMVI (minimum roadworthy standards for vehicle components)	Yes	Yes		Yes	
3	Government authority overseeing PMVI	Police	Police	Police	Police	
4	Number of vehicle inspection centres	0		0	1	
5	Average capacity of inspection in each centre				200 – 250 vehicles/day	
6	1st PMVI after registration (months)					
	PSV		12		18	
	HGV		12		18	
	Private		12		18	
	Motorcycles and 3-wheelers		12			
7	Frequency of PMVI (months)					
	PSV	6	12		6	
	HGV	6	12		6	
	Private	12	12		12	
	Motorcycles and 3-wheelers	-	12		-	
8	Extra items for checking PSV	Yes	Yes		Yes	
9	Extra items for checking HGV	Yes	Yes		Yes	
10	Enforcement means	Sticker at windscreen			Sticker at number plate	
11	Inspection of motorcycles	No	No		No	
12	PMVI fees					
	PSV				RF15,000	
	HGV				RF25,000	
	Private				RF10,000	
	Motorcycles and 3-wheelers					
13	Fee for PMVI re-inspection					
	PSV				20% of	
	HGV				inspection	
	Private				fee	
	Motorcycles and 3-wheelers					
14	Availability of modern testing equipment	No	No	No	Yes	
15	Inspection after modification on vehicle	Yes	Yes		Yes	
16	Inspection after road accident	Yes	Yes		Yes	
17	Inspection at transfer of ownership		Yes			
18	Random (roadside) inspections	Yes	Yes		Yes	

Table 4: PMVI items in the current regulations in EAC countries

	PMVI Items	Tanzania and Zanzibar	Kenya	Uganda	Rwanda	Burundi
	Brake system	✓	✓		✓	
	Wheels and Tyres	✓	✓		✓	
	Steering system	✓	✓		✓	
	Headlights	✓	✓		✓	
	Windscreen	✓	✓		✓	
	Horn and lighting system	✓	✓		✓	
	Suspension system	✓	✓		✓	
	Towing attachments(trucks)	✓	✓		✓	
	Body and structure	✓	✓		✓	
	Mirrors	✓	✓		✓	
	Seats and seatbelts	✓	✓		✓	
	Engine compartment	✓	✓		✓	
	Driveline	✓	✓		✓	
	Fuel system	✓	✓		✓	
	Exhaust system	✓	✓		✓	
	Exhaust emissions	✓	✓		✓	
	Vehicle Identification	✓	✓		✓	

In all EAC countries vehicle inspection and certification is performed by the police departments. This framework is not practical because the same police department also enforce the road traffic and safety laws by impounding vehicles that are not roadworthy. However, in most EAC countries plans are underway to change the institution arrangement for vehicle inspection to be under government department or authority in the respective ministries dealing with Transport.

In Tanzania, a Draft Bill to establish Driver and Vehicle Examination and Licensing Agency (DVELA) which will be a corporate body under the Ministry of Transport has been proposed. Among the objectives of DVELA is to promote to ensure the use of road worthy vehicles on the roads and in other public places. To achieve its objectives DVELA shall have the following functions in relation to improving the quality of registered vehicle:

- inspect, test and register motor vehicles;
- issue vehicle registration certificates;
- issue vehicle examination certificates;
- license and regulate private garages to undertake vehicle testing;
- advise the Minister on policy formulation and development strategy for the achievement of the object of the DVELA;
- ensure strict compliance with this Act and regulations made under it regarding vehicle examination and licensing;
- carry out such other functions as are incidental to the attainment of the object of the DVELA.

To date, DVELA has not started to perform its duties and at present, Police function as vehicle inspectors as well as law enforcers.

With exception of government vehicles and few big companies and individuals, the majority of vehicles imported in EAC countries are used vehicles mainly from Japan, Korea, and Europe. Some EAC countries have appointed foreign companies in these countries to conduct pre-export verification of conformity and roadworthiness inspection as a requirement for vehicle to comply with local used vehicles safety standards. The Tanzania Bureau of Standards (TBS), Uganda National Bureau of Standards (UNBS), and Kenya Bureau of Standards (KEBS) have appointed some foreign companies including Japan Export Vehicle Inspection Center (JEVIC) to conduct pre-export Roadworthiness Inspection of used motor vehicles from Japan, United Kingdom, Singapore and South Africa. Vehicles imported to Uganda from the above countries will be subjected to a penalty upon arrival if not inspected from the country of origin.

The requirements for inspection were initiated by the Bureau of Standards to minimize the risk of substandard vehicles entering local markets. All used vehicles must meet the local safety standard requirements through pre-export inspections which for the case of vehicles imported from Japan; it is a requirement that the vehicle passes the Japanese PMVI (Shaken). Uganda and Kenya have additional requirements. In Uganda the vehicle Air Condition (A/C) system should not use R12 gas. Whereas in Kenya the additional requirements are:

- The vehicle must be less than 8 years of age
- The vehicle must be right hand drive

7.7.1 Motorcycle Taxis

Increase of urbanization in towns and cities in EAC countries has caused an increase of urban peripheries with lack of planned infrastructure such as roads and public transportation. The road networks in populated urban peripheries are generally underdeveloped and in poor condition, and most of the cities in EAC countries are without state-owned public transport companies. This situation has caused an alarming increase of transport demand in the towns and their peripheries and consequently the establishments of informal public transport system.

Informal transport supply involves the operation of vehicles of various sizes, from small cars to cargo vehicles that have been unlawfully modified to carry passengers and of recent motorcycle taxi, which from the point of view of unplanned urbanization, poor road conditions, and increasing traffic jams has find a big share of market in public transportation.

Literature review on the subject reveals that *motorcycle taxis* (see Figure 10) are common in most developing countries in African and Asian with a consequent on increasing injuries cause by motorcycle taxi accidents. At Bugando Hospital in Mwanza (Tanzania), motorcycle accidents accounted for 37.5% of all road traffic injuries in 2010, with riders contributing 55.2%, passengers 33.9%, and pedestrians 10.9%[12].

Factors contributing to motorcycle accidents include recklessness, indiscipline, lack of respect to other road users, and not using helmet for the rider and passengers. The extent of law enforcement of traffic laws to motorcycle taxis in EAC countries such as the use of helmets, reflective clothes, overloading, etc., differ from country to country and between towns and cities.

[12] Tanzania Journal of Health Research , Vol. 12 (4). October 2010.

Figure 10: Motorcycle taxi in EAC cities, towns, and villages. (Source: Newsletter of the National Institute of Medical Research, Tanzania. Vol. 4-1, 2011).

7.7.2 Unlawful Modifications

Unlawful or uncertified modifications are common in EAC countries. The objectives of these aftermarket modifications vary from increasing payload, protecting body structure, to mere looks and personalization of vehicle. The most common modifications are:

- Chassis modification (Extending, shortening, and strengthening using different materials)
- Removal or addition of axle
- Conversion of cargo truck to tipper or tanker and vice versa
- Replacing engine with another of different model or type
- Bull bars
- Roof racks
- Towing connections
- Mounting of equipment (winch, crane) to the chassis
- Extra seat rows in PSV
- Extra fuel tank (trucks)
- Suspension lift
- Tinted overlays to windows
- Aggressive horns, and sirens imitating ambulance or police vehicles

7.8 Proposed Harmonized PMVI Framework and Standard for EAC Countries

The proposed PMVI framework has been designed to ensure as far as possible that all vehicles are properly maintained and are periodically inspected to make sure that they comply with certain important minimum requirements of the regulations. The proposed framework has been achieved through a review of good practices around the world in high, middle, and low income countries. Other analyses conducted to supplement the proposal include:

- Trend on advanced vehicle safety systems
- The current situation in EAC countries on vehicle inspections and in particular the problems on infrastructure (facilities and equipment), human capacity, legislation and enforcement, malpractices and corruption, and the vehicle fleet.

- The standard and inspection specifications proposed for COMESA/EAC/SADC countries

In reality PMVI should not involve dismantling any equipment or item of the vehicle for inspection. Hence, in order to check the condition of some systems which are not possible to inspect visually it is important to use appropriate testing equipments which have been specifically developed for the purpose. These systems include brakes, steering system, suspension, headlights, and emissions.

The time required for an individual test will depend on the number of items required for testing, testing equipment and the complexity of the inspection itself. The average time for an inspection should be set as the result of initial trials and will be different for different categories of vehicles. It is important the vehicle inspectors are not overloaded and also the vehicle owners/drivers should not wait for unacceptably long time.

7.8.1 Legal Frame

Mandatory Vehicles for PMVI

We **recommended** that:

- Vehicles which should be mandatory for PMVI to be all vehicles using public roads including government and military vehicles.
- The categories for compulsory PMVI should be:
  - ✓ Private light vehicles (GVM < 3500 kg)
  - ✓ Light passenger service vehicles (PSV< 8 people)
  - ✓ Passenger service vehicles (PSV> 8 people)
  - ✓ Heavy goods vehicles (HGV > 3.5 Tonnes)
  - ✓ Trailers
  - ✓ Private motorcycles and three-wheelers
  - ✓ Taxi motorcycles and three-wheelers

Initial Inspection, Frequency, and Fees

In a scenario where all resources are available for conducting PMVI schemes, the decision on the frequency of inspections should depend on the average age of the fleet and the categories in the fleet. Categories of vehicles which cover more mileage and those which transport many passengers should be inspected more frequently. From statistics it is obvious that commercial cargo trucks and passenger service vehicles (PSV) are given high priority on the frequency of inspection. In most countries around the world PSV and HGV are also inspected more frequent than private vehicles. Other factors causing trucks and PSV to be inspected more frequent include:

- The competitive nature in transportation sector may prompt operator to use the vehicle which is due for maintenance
- Heavy trucks need to be inspected more frequent than private vehicles because of the damage they normally cause to the road infrastructure and to other vehicles when involved in an accident.

Of recent there has been an increase of motorcycle taxis and the consequent accidents in most towns and cities in EAC countries. The main cause of most motorcycle taxi accidents includes; the rider inexperience, recklessness, and overloading. However, the poor conditions of the motorcycle also contribute to the accidents.

Our **recommended** inspection schedule for the proposed categories of vehicles is as shown in Table 5.

Table 5: Recommended frequency of PMVI

Vehicle category	Initial PMVI (since new)	Frequency (months)
Private light vehicles (GVM < 3500 kg)	2 years	12months
Passenger service vehicles (PSV < 8 people)	1 year	6 months
Passenger service vehicles (PSV > 8 people)	1 year	6 months
Commercial trucks (<3.5 tonne) and Heavy goods vehicles (>3.5 Tonnes)	1 year	6 months
Trailers	1 year	6 months
Motorcycles (private)	2 years	12 months
Motorcycles (taxi)	1year	6 months
Ambulance and other emergency vehicles	1 year	6 months

The fees schedule for different categories of vehicles is based on fees schedule in the region (Kenya, Rwanda, Botswana). The recommended fees in USD are shown in Table 6.

Table 6: Recommended fees for PMVI

Vehicle category	Inspection Fee (USD)
Private light vehicles (GVM < 3500 kg)	15
Passenger service vehicles (PSV < 8 people)	15
Passenger service vehicles (PSV > 8 people < 30 people)	20
Passenger service vehicles (PSV > 30 people)	30
Commercial vehicles (<3.5 tonne)	20
Heavy goods vehicles (>3.5 Tonnes, 2 axles)	30
Heavy goods vehicles (>3.5 Tonnes, 3 and more axles)	40
Trailer (1 axle)	10
Trailer (2 and more axles)	15
Motorcycles (private)	5
Motorcycles (taxi)	5

Enforcement

Vehicles which pass the PMVI should be given a certificate and a sticker.

We **recommended** that:

- The certificate should be put in the vehicle with other vehicle documents for police to inspect if needed at any time.
- The sticker should be placed at the bottom right side of the windscreen. The sticker should show:
  ✓The vehicle registration number
  ✓The date when the next inspection is due.
  ✓A code of the centre which the inspection was conducted

Vehicles which fail PMVI should be allowed to rectify the problem and report back for re-inspection of the faulty item within two weeks.

We recommended that:

- 20% of inspection fee should be charged for re-inspection.
- Full inspection should be conducted if the vehicle is not taken back for re-inspection within two weeks, in which case full inspection fee should be charged.

7.8.2 Recommended Minimum Standards for Technical Inspection in PMVI

Apart from the driver factor, the extent to which a certain item is more likely to cause an accident than others when it fails depends on a number of factors at the time of an accident, including:

- Speed of the vehicle
- Weight of the vehicle
- Type of vehicle
- Road condition
- Environment conditions (rain, day/night, and other obstructions including traffic)

Vehicle systems and items that increase the likelihood of accident when they fail and those which increase the severity of casualties after the accident are the target for the recommended PMVI scheme. These are categorized as shown in the Table 7.

Table 7: Categories of systems that are target to PMVI

	System/Item that increase chances of accident and injuries when they fail	**Obvious system/item**
1	Systems/item that *impair the driver's control of direction and/or speed* of the vehicle	Steering, tyres, brakes, insecure driver's seat.
2	Systems/items that *impair the driver's view* of the road	windscreen, wipers and wiper washer, side and rear view mirrors, headlights
3	Systems that *impair the visibility* of the vehicle from other road users	Lights, horn.
4	Components/items that intrudes into other users' road space or undue danger or nuisance	Oil leaks, sharp projections, excessive smoke, excessive noise, body modification
5	Components/items that *impair the built-in occupant protection*	missing or broken seat belts, insecure seats, weakened body structure
6	Components/items that *increase the risk of further injury* to the occupants and other road users after an accident	insecure fuel tank, inoperative emergency exits, battery security

The obvious systems/items in the above categories are listed in the table but there are many more components involved and it should be the task of the inspector to identify them. The fact that periodic inspection require all registered vehicles to be inspected at least once in every few years, clearly shows that thousands if not hundreds of vehicles need to be inspected everyday for every mandatory vehicles to be inspected. Some systems require only visual and audible inspection to identify if they are defected or missing.

Other systems/items are impossible to judge on their condition through visual inspection and they require either to be dismantled for inspection or to be tested by equipment which have been designed and developed to internationally recognized standards specifically for the task. **We recommended the latter** for harmonized PMVI scheme to be introduced in EAC countries. This is because it eliminates human error and bias and improves inspection quality and consistency which are important in the harmonized inspection scheme.

Furthermore, it is *not practical to open the systems and components for inspection in PMVI*. This will not only take a lot of time to inspect one vehicle, but also parts such as seals, lock nuts, and retainer clips and pins which are recommended by manufacturers to be replaced after each opening, have to be replaced.

We **recommended** that:

- Systems/items that cannot be visually inspected to assess their condition be tested by either equipment which have developed to internationally recognized standards specifically for that task, or by test driving the vehicle.
- Systems/items should not be dismantled for the purpose of technical inspection.

Recommended Safety Items for Inspection

The minimum recommended list of system/items for PMVI inspection presented below has been achieved by thorough consideration of the existing situation in EAC countries and the fact that most EAC countries do not have PMVI schemes and for those who have started the schemes is still in the infant stages. The review of good practices around the world has been comprehensively done and the proposed COMESA-EAC-SADC standard for testing of vehicles has been studied in the view of future harmonization.

Consideration has been given to the fact that the average age of the vehicle fleet in EAC region is higher than most of the countries which have been reviewed. Furthermore unlawful modifications are common practice to increase vehicle payload or other reasons depending on the type of modification. Most drivers and vehicle owners have no knowledge on the effects of modifications on road safety and they are not aware on the existence of standard specifications to be observed when conducting modifications. Other reasons contributing to unlawful modifications include lack of clear regulations, inadequate law enforcement, and lack of certified garages and workshops for carrying vehicle modification which meet international standards.

We **recommended** that:

- A list of common vehicle *modifications* and their respective approved designs be made publicly available to vehicle owners and garages/workshops.
- Inspection of unlawful modification be included in PMVI checklist.
- Vehicles which are found to have uncertified modifications should be failed.
- The PMVI centre should not be responsible to assess vehicle modifications. That task should be left to "Bureau of Standards" or other similar authority for certification of modifications

Prior to the technical inspection, the vehicle number plate has to be checked to see if it is visible and properly secured. The vehicle identification marks have to be checked to make sure that they match the vehicle registration card as follows:

- Make
- Model
- Model number
- Body type
- Colour
- Class of vehicle
- Year of manufacture
- Vin/Chassis number

- Engine number
- Fuel used
- Number of axles
- seating capacity
- Tare weight
- Gross weight

We **recommend** the following items for technical inspection for PMVI are categorized into nine groups as follows:

- ✓ *Group I: Vehicle documents and Identification*
- ✓ *Group II: Items which affect control of vehicle* (Brakes, wheels and tyres, steering system, suspension system, driver seat)
- ✓ *Group III: Items which impair driver view of the road*: (Windscreen, wipers and wiper washer, side and rear view mirrors, headlights and spotlights alignment)
- ✓ *Group IV: Items which reduce visibility of the vehicle: (*Lights, reflectors, horn)
- ✓ *Group V: Items which impairs built-in occupant protection*: (Seats and seat belts, body structure, doors, floor and steps)
- ✓ *Group VI: Items which intrudes other road users safety:* (Oil leaks, noise, excessive smoke, sharp protrusions, mud guards)
- ✓ *Group VII: Items which increase risk of further injuries after accident:* (Emergency exit, fuel system, battery security)
- ✓ *Group VIII: Extra safety items for HGV and Trailers:* (Rear under run protection, cab mounting, trailer couplings, turntable and king pin, trailer parking brake, trailer lights and reflectors, twist-locks for securing containers)
- ✓ *Group IX: Uncertified modifications:* Any modification which falls into any of the I-VIII groups above such as:
  - Chassis (strength, dimensions, mounting of equipments)
  - Axles (addition, removal)
  - Body (change of body for different purposes, e.g. from cargo body to fuel tanker)
  - External attachments (bull bars, roof racks, etc)
  - Windows and windscreen visibility (tinted glass and excessive stickers)
  - Noise (aggressive horns, imitations of ambulance or police sirens)

We **recommend** the following items for inspection for each proposed category of vehicles are as show in Table 8.

Table 8: The **recommended** minimum items for each category of vehicle during PMVI

Group	Systems/Items for Inspection	All vehicles	Light vehicles	PSV >8 people	HGV >3500 kg	Motorcycles
I	Identification marks	✓				
II	Brakes	✓				
	Wheels and tyres	✓				
	Steering system	✓				
	Suspension	✓				
III	Windscreen	✓				
	Wipers and wiper washer	✓				
	Side and rear view mirrors	✓				
	Headlights and spotlights alignment	✓				
IV	Lights	✓				
	Reflectors	✓				
	Horn	✓				
V	Seat belts		✓	✓	✓	
	Seat	✓				
	Body structure	✓				
	Doors	✓				
	Entrance /exit			✓	✓	
	Floor and steps			✓	✓	
VI	Oil leaks	✓				
	Noise	✓				
	Excessive smoke	✓				
	Sharp protrusions	✓				
	Mud guards			✓	✓	✓
VII	Emergency exit			✓		
	Fuel system	✓				
VIII	Rear under run protection,				✓	
	Cab mounting				✓	
	Trailer couplings				✓	
	Turntable and king pin				✓	
	Trailer parking brake				✓	
	Trailer lights and reflectors				✓	
	Twist-locks for securing containers				✓	
IX	Uncertified modifications*	✓				

*Chassis strength, dimensions, mounting of equipments; Axles addition/removal; Change of body type; External attachments e.g., bull bars, roof racks, etc; Windows and windscreen visibility, e.g., tinted glass and excessive stickers; Noise, e.g., aggressive horns, imitations of ambulance or police sirens.

Recommended Minimum Environment Standards for Inspection

We **recommend** the following minimum environment standards at the time of PMVI are as shown below:

- Spark Ignition Engines (Petrol, LPG, and CNG Engines):
    - Carbon monoxide (CO)
    - Hydrocarbons (HC)
    - On board diagnostic-II (OBDII) condition
    - Fuel tank cap
    - Noise
    - A/C refrigerant
- Compression Ignition Engines (Diesel Engines):
    - Particulate Matter (PM)
    - Fuel tank cap
    - Noise
    - A/C refrigerant

7.8.3 Supervision and Organization of PMVI

Supervision

Police should not be directly involved in vehicle inspection supervision because there may be a conflict of interest between the police department which inspects the vehicles and the police officers who stops the vehicle for random inspection or law enforcement. Furthermore, the law enforcement department is not technically oriented in human capacity to follow up or cope with the rapid advancement of vehicle management and safety systems. The police have an important part to play in the enforcement of road safety regulations and compliance on inspections through checking of stickers and random roadside visual and audible inspections.

The operation of inspection facilities should be under private company and closely monitored by **Government Ministry or Department responsible for road transport**. The vehicle inspection quality and performance need to be continuously monitored and the degree to which performance targets are met assessed. Inspection frequency, fleet size and length of inspection determine the number of testing facilities and minimum staffing requirements.

Recommendations:

- ✓ The operation of inspection facilities should be under private company and closely monitored by **Government Ministry or Department responsible for road transport**
- ✓ The PMVI scheme need to be continuously monitored and the degree to which performance targets are met assessed.
- ✓ Inspection frequency, fleet size and length of inspection determine the number of testing facilities and minimum staffing requirements

Quality Management

Manuals and documentation are necessary to ensure uniformity on items to be inspected, procedures for inspection for different categories of vehicles, the pass/fail criteria for each item checked, certification and enforcement. The manuals should also be accessible to the public to increase the public confidence and transparency on the inspection scheme and to inform the drivers and fleet operators the standards they are supposed to meet. The regular audit of testing facilities, updating on procedures, and random sampling of recently tested vehicles for re-checking are necessary to ensure that the scheme maintains the standard for inspections and the scheme can be seen to be open and honest. The scheme should be designed in such a way as to reduce opportunities for corrupt practices by drivers to the vehicle inspectors.

The testing facilities must operate in accordance with a formalized quality system. All procedures related to the quality system should be documented as to cover all the provisions of the inspection standard and any such further procedures deemed necessary for the assurance of good housekeeping. The preparation, approval and implementation of any changes, amendments or modifications to any work instruction or test method shall be clearly defined and any amendments to existing documents shall be documented. The test facility shall establish and maintain a system to control all documentation that relates to the provisions of this standard.

Recommendations:

- ✓ Inspection manuals showing all the PMVI procedures and the pass/fail criteria for each item should be developed before the exercise of inspection is established
- ✓ The inspection manuals should be accessible to the public to increase public confidence and transparency on the inspection scheme and to inform the drivers and fleet operators the standards they are supposed to meet
- ✓ The inspection centers should be regulated and randomly audited in terms of inspection procedures and accuracy of equipment.
- ✓ The PMVI scheme should be designed in such a way as to reduce opportunities for corrupt practices to vehicle inspectors.

Vehicle Inspectors

Inspection staffs that are qualified to a set standard of technical ability are necessary in the vehicle inspection scheme in order to make judgmental decisions on pass/fail criteria for items which are visually inspected. Training of inspection staff in methods, standards, administration, and control of the systems and equipment is essential before the exercise of inspection begins.

The pre-requisite for a vehicle inspector candidate must be a technician level in auto mechanics or degree level in mechanical engineering. A training course on vehicle technical inspection should be designed for different classes of vehicles and the curriculum for the course to be common throughout the EAC region.

Recommendations:

- ✓ Inspection staffs should attend a specialized (tailored) course on vehicle technical inspection and should be qualified to a set standard of technical ability necessary in vehicle inspection
- ✓ Training curriculum on vehicle inspection should be developed and should cover technical inspection procedures, standards and specifications for vehicle safety items and systems, testing equipment, administration, and quality control.
- ✓ The pre-requisite for a vehicle inspector candidate should be a technician level in auto mechanics or degree level in mechanical engineering.

7.8.4 Facilities and Equipment

We **recommended** that the premises for the inspection and certification of vehicles must have inspection area which:

- enables a safe and thorough inspection
- has sufficient lighting to enable good visibility of the vehicle being inspected and the equipment used in the inspection process.
- Complies with Occupational Safety and Health requirements
- roofed
- paved floor

It is further recommended that the centre should have offices for administrative works and document storage.

We recommend the following equipment as shown in Table 9 to be used for inspection of vehicles for roadworthiness and environment compliance:

Table 9: Recommended equipment for PMVI

Recommended Equipment		**Purpose**
1	Optical headlight meter	Measures the alignment of the light beams for pitch and direction
2	Roller brake testing machine	Measures the brake force on each axle separately and brake imbalance between wheels of the same axle and between axles.
3	Steering system checking machine (shakers)	Checks wear on wheel bearings, steering linkages and suspension
4	Suspension tester	Measure the suspension imbalance of wheels on a same axle
5	Side slip testing machine	Wheel alignment
6	Black smoke (opaque) meter	Measures the amount of particulates in diesel engines
7	Emission meter	Measures the levels of carbon monoxide (CO) and hydrocarbons in petrol engines
Other Recommended Equipment and Items		
8	Garage pit	For inspection underneath (heavy and light vehicles)
9	Hydraulic lift	Light vehicles only
10	Crow bars	For checking play in steering linkages
11	Inspection mirrors	For checking inaccessible locations
12	Torch	For checking play underneath the vehicle
13	Tyre tread depth gauge	To measure the depth of tyre tread

7.8.5 Management and Ownership of Inspection Facilities

There are three options for the ownership of inspection facilities: privately operated; publicly operated; and a combination of the government owning facilities and equipment and private company conducting inspections. The advantages and disadvantages of these alternative arrangements are outlined below:

Private Ownership

- ✓ Reduced capital cost to the government
- ✓ Close supervision of testing standards, facilities, and training of testing station staff.

Government Ownership

- ✓ Increase capital cost
- ✓ High possibilities of corruption and malpractices

Government-Private Ownership

- ✓ Reduce capital cost to both
- ✓ Easier to monitor and supervise by the government authority

It is **recommended** that the ownership of the facilities and operation of inspections be under private companies.

Recommendations:

- ✓ We **recommend** that the ownership of the PMVI facilities and operation of inspections be under private companies.

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FINAL REPORT

ANNEX A.8

HARMONIZATION OF DRIVERS TRAINING AND TESTING

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GLOSSARY AND ACRONYMS

ADTSEA	American Driver and Traffic Safety Education Association
BICO	Bureau for Industrial Cooperation
CBTA	Competency-Based Driver Training and Assessment
CIEFA	*Commission Internationale des Examens de Conduite Automobile*
CPC	Drivers Certificate of Professional Competence
DSA	Driving Standards Agency
DVA	Driver and Vehicle Agency
DVELA	Driver and Vehicle Examination and Licensing Agency
EAC	East African Community
ECA	Economic Commission for Africa
EU	European Union
FESARTA	Federation of East and Southern Africa Transporters Association
GVM	Gross Vehicle Mass
HGV	Heavy Goods Vehicle
ITC	Industrial Training Centre
NHTSA	National Highway Transportation Safety Administration
NIT	National Institute of Transport
PDL	Pre-driver Licensing
PSV	Passenger Service Vehicle
RTSA	Road Transport and Safety Agency
SADC	Southern Africa Development Community
SPC	Safe Performance Curriculum
SSATP	Sub Saharan Africa Transport Policy Program
TIN	Tax Identification Number
TRA	Tanzania Revenue Authority
TRRL	Transport and Road Research Laboratory
USA	United States of America
VETA	Vocational Education and Training Authority
VORT	Vehicle on Road Test
WHO	World Health Organization

EXECUTIVE SUMMARY

The five EAC partner states, Kenya, Uganda, Tanzania, Rwanda and Burundi had agreed to standardize road transport laws and regulations in the EAC in order to improve the efficiency of domestic, transit and cross-border road traffic. This report is on the current situation of driver training and testing in EAC countries, SADC and other countries in Africa and around the world. It is part of the effort for the standardization of transportation facilities undertaken by EAC. It addresses two ToRs of the assignment, namely (1) develop common training manuals for driver training for different classes of vehicles (2) To develop common methods and procedure for testing drivers of different categories.

The study reviewed the situation in developed and developing countries. It was noted in the review that the traditional methods for training drivers had been fairly uniform throughout the world in terms of areas of coverage during training, but with notable variations in training durations, theory tests, and in practical test duration and procedures.

Data on the increasing trend in road accidents have alarmed many countries around the world. Evaluation studies on the effectiveness of the traditional methods of training and testing drivers, and the subsequent methods which replaced traditional methods have both been evaluated in many countries. Among them are the USA, some European and Asian countries, and in South Africa. Among the shortfalls in the training system which have been identified includes:

- Short periods on driver training courses and most of the course duration is spent teaching basic vehicle handling skills;
- Training mostly geared towards passing the driving test, rather than teaching the student to become safe and competent drivers –this is for countries where driver training establishments are privately owned;
- Defensive driving training and hazard perception skills, as an accident prevention strategy is not taught adequately;
- Most driving courses train basic vehicle handling skills and traffic codes and do not train new drivers on environmental factors that affect driving, complex perceptual skills to avoid risky driving situations, driver impairments, how to handle emergency situations, personal readiness and self awareness;
- In most driver training courses training instructors do not ensure that learners not only learn the rules, but also understand the reasons behind the rules;
- Good quality training of instructors is not emphasized;
- Quality control of instructors and driving schools is not adequate.

In recent years some developed countries have incorporated computer technology in training and testing of drivers which has reduced the training costs involved and improved the quality and consistency of theoretical testing of candidates.

The approaches to the proposed framework for driver training and structure for testing drivers have been developed based on good practices, studies and approaches which have been taken by various researchers, government and non-government institutions and organizations around the world. Among them include:

- Driver and Vehicle Licensing Agency (DVLA) – United Kingdom
- K53 Licensing System – South Africa
- Transaid: An independent United Kingdom NGO
- BICO –University of Dar es Salaam, Tanzania

This report proposes the driver training syllabus and testing structures in EAC Countries for riders of motorcycles (capacity < 125 cc), and drivers for light vehicles (GVM < 3500 kg), Heavy Goods Vehicle (GVM > 3500 kg), and Passenger Service Vehicles (PSV).

8 HARMONIZATION OF DRIVERS TRAINING AND TESTING

8.1 Introduction

8.1.1 Background

Road deaths continue to be a huge killer across Africa, often pushing families further into poverty and creating a massive burden on an under resourced health service. Through investment in driver training, drivers can become safer and more risk aware, ultimately reducing the number of fatalities and transportation

According to the World Health Organization (WHO)[1], over 1.2 million people die each year on the world's roads, and between 20 and 50 million suffer non-fatal injuries. In most regions of the world this trend is still increasing. It is reported that over 90% of the world's fatalities on the roads occur in low-income and middle-income countries, which have only 48% of the world's registered vehicles. Low-income countries have higher road traffic fatality rates (21.5 per 100 000 population) than high-income countries (10.3 per 100 000).

Formal driver education is an essential part in the process of teaching people how to drive. Driving schools are crucial as they are the educational institutions tasked with providing the relevant education that learners need in order to become competent drivers. Because this education is vital in driver socialisation, it has to be structured and carried out in a way that ensures maximum affectivity. This education is essential in ensuring that learners acquire all the relevant driving skills

The need for an effective driver testing and training system in developing countries is important for the following reasons:

- Most developing countries have the mixture of motorized and non-motorized modes with an inadequate road network and hierarchy, and poor traffic control methods which contribute to difficult driving environment;
- The rapid increase in motor vehicles with drivers which are not properly trained will result in a proportionate increase in accidents.

The key components to ensure that drivers in the roads are competent to handle their vehicles and the environment surrounding them are appropriate training and testing. However, it has been observed that, the driver training courses are taught over short periods of time and most of the course is spent teaching theory and basic vehicle handling skills. This leaves minimal time to try and teach safe driving skills, and safety messages can be overwhelmed by attitudes, motivations, peer influences, and other lifestyle factors that shape driving styles. Additionally, the audience for driver education may be relatively unmotivated regarding safety, the primary motivation being to learn enough to get a driver's license[2].

The standards required to obtain a driving licence still vary between the EAC countries. This compromises safety because drivers from one country may not have been trained to the same standard as those from other countries. This might pose a greater risk to themselves and other road users. In addition, uniform training and testing of drivers is among the objectives of the EAC is to facilitate the free movement in road transportation between member states. In order to meet this objective safely, there should be harmonisation of syllabus for driver training and testing of drivers in

[1] Global Status Report on Road Safety.2009. World Health Organization (WHO)

[2] Allan F. Williams, David F. Preusser and Katherine A. Ledingham. (2009). Feasibility Study on Evaluating Driver Training.

order to harmonize road user behaviour. Other elements which affects road user behaviour and which have to be harmonized include:

- age requirements;
- driver license renewal periods and renewal requirements for elderly drivers
- traffic regulations, signals, signs, and codes; and
- penalties and sanctions for traffic offences.

However, these are dealt with in other thematic areas.

8.1.2 Objectives

The objectives in this task are:

- To develop common training manuals for driver training
- To develop common methods and procedure for testing drivers of different categories.

The main task includes:

- Reviewing training methods and syllabus and driver testing and assessment procedures from EAC member states, SADC countries, and other countries in Africa and around the world.
- Proposing training methods and syllabus for driver training in EAC countries
- Proposing driver testing and assessment procedure for driver candidates in EAC member states.

8.1.3 Methodology

The main activity in this task is through desk study. Other tasks include: interviews and discussions to different experts and stakeholders in the region, and assessing the information gathered to assist in developing driver training guidance and testing procedure in EAC countries.

8.2 Overview of Driver Training and Testing

Experience of most countries with growing numbers of vehicles and drivers is that a program of structured training, followed by a valid and reliable test of competence, can contribute towards a reduction in road accidents. Most vehicle accidents result from a lack of planning, anticipation, concentration, or control by those involved. New drivers require appropriate instructions and have to be guided through situations where a lack of experience can make them vulnerable and have serious consequences.

In most countries around the world and especially developed countries, driver training is seen as a necessary requirement to get a driving license. The normal approach is to follow a syllabus that covers sufficient elements to enable the student driver to pass a driving test which in most countries is conducted by a government institution which then issues a driving license. Ideally, the syllabus and the training should aim to prepare learner drivers for all potential hazards and situations and not just those tested by the examiner at the time of the test, but in reality, it cannot.

Driving is a skill that takes years to master properly. Learning should not stop when the candidate passes the driving test. Safe driving is as much about attitude as about ability to control the car. Driver training should not only introduce this message but should reinforce it throughout training. During training the learning driver should also acquire the knowledge of the Highway Code and the motoring laws, and a thorough understanding of the responsibilities of a driver.

Generally, the syllabus for driver training should cover the following main topics:

- legal requirements;
- car controls, equipment, and components;
- road user behaviour;
- vehicle characteristics;
- road and weather conditions;
- traffic signs, rules, and regulations;
- car control and road procedure; and
- additional knowledge about driving-related situations.

Defensive Driver Training, either as part of the initial training or as an advanced driver course, has been found to be effective, especially in reducing accidents. Once the initial training syllabus has been determined, it may be useful to consider defensive driver training as an accident prevention strategy.

A study in Asia suggested to consider training and testing drivers of non-motorized vehicles in developing countries, even though it is likely to be impractical, but it may alleviate accidents since they mix with motorized vehicles in the same roads and their drivers/handlers follow traffic codes even though they were not formally trained and they do not possess any class of driving license like motorized vehicles drivers[3].

The driving test is the main indicator to judge if the trainee has understood the lessons given in a particular training syllabus. In most countries driver training establishments are privately owned and training is mostly geared towards passing the driving test rather than teaching the student to become a safe and competent driver. In most countries drivers test comprises of the theory and practical components.

8.2.1 Theory Test

A theory examination is aimed to test the trainee knowledge of the Highway Code (traffic signs, traffic regulations), as well as basic knowledge of vehicle performance and simple safety checks before using a vehicle. Learner driver knowledge may be tested by oral or written examination. Oral test is appropriate where the candidate's literacy rate is low. In this case, the oral examination may be carried out by the driving examiner either immediately before or after the practical driving test and it should follow a set pattern. Literacy is not necessarily a requirement for a good driver, although ability to read and understand road signage is essential. Hence, written test or oral tests should be available in all the main languages in use within the country or the use of translators should be permitted

In some developed countries the theory test is computer-base where the test is normally taken before the practical driving test and passing the theory test is a prerequisite for the practical test. Written test has an advantage over oral test that it is less labour intensive and ensures consistency and less human bias and error. The number of test questions and pass requirements vary between countries. A study conducted for Asian-Pacific countries proposed that test covering sufficient topics will require about 50 questions and last about 45 minutes and theory test should include at least the following topics[4]:

- traffic regulations;
- vehicle handling;

[3] Road safety guidelines for the Asian and Pacific region. Asian Development Bank. 1998.
[4] Road safety guidelines for the Asian and Pacific region. Asian Development Bank. 1998.

- vehicle manoeuvring procedures;
- hazard perception; and
- effects of weather and road conditions on driving.

8.2.2 Practical Test

Few countries conduct the whole practical test on public roads. In this case they use off-road facilities to examine the technical control of vehicles; i.e., emergency stop, hill start, manoeuvres, and parallel parking. Normally the off-road test is carried out first and this is followed by the on-road test, which should be conducted in light traffic on normal roads. All test routes should ideally be on-road, although it is acceptable to have a combination of off and on-road routes. They should be as uniform as possible and include a common range of typical road and traffic conditions.

The test route should be chosen to test trainee hazard perception skills without exceeding their capabilities. Pedestrian crossings and junctions and roundabouts, should be included to provide opportunity for “give-way” sensitiveness and road-sharing behaviour of the candidate. The on-road test should include (but not be limited to) the following evaluations to know if the candidate has understood the lessons given during training:

- take proper precautions before starting the engine;
- make proper use of controls;
- position the vehicle correctly on the road and make normal stops safely;
- drive at a speed appropriate for the conditions;
- make effective use of rear-view mirrors;
- give all necessary signals;
- show alertness and anticipation of the actions of other road users;
- overtake, meet, and cross the path of other vehicles safely;
- act properly at road junctions;
- take appropriate action at pedestrian crossings; and
- take prompt and appropriate action on all traffic signs, road markings, traffic lights, signals by traffic controllers and other road users.

The training and testing of drivers of heavy trucks and passenger buses must include extra skills, hence more time for practical test. It is recommended that the time taken to examine a candidate for a heavy vehicle should be about twice that for a driving test for a light vehicle. The potential for damage in accidents by heavy vehicles is extremely high because of the high momentum they carry, hence drivers for HGV must be trained to a much higher standards that those for light vehicles. Standardized assessment forms are necessary to ensure consistency and to facilitate monitoring and to provide feedback. Errors should be pre-coded and thus standardized.

It is also recognized that successful candidates will experience many different conditions that cannot be examined in the tests. These include night driving and driving in rain, mist, sand, mud, ice, or snow, and on high speed highways or gravel roads. In spite of these deficiencies, it is not feasible to extend tests to any other conditions than those experienced on the day of the test. There is, therefore, an opportunity for special driving schools to offer training courses in such conditions.

8.2.3 Technological Developments in Drivers Training and Testing

Recent technological advances in driver training and testing are:

- Driving simulator
- Computer controlled theory test

Driving simulators are being increasingly used for training drivers all over the world. Studies have shown that driving simulators are proven to be practical and effective educational tools to impart safe driving training techniques. Other uses of simulators in driver-related training and research include:

- Training in critical driving conditions
- Training for impaired users
- Analysis of the driver behaviours
- Analysis of driver responses
- Analysis of the user performances
- Evaluating user performances in different conditions (handling of controls, etc)

In a simulator different types of vehicles can be configured for use as light vehicle, heavy vehicle, etc. Special utility vehicles can also be simulated such as, dump trucks and other construction vehicles, and buses. An added advantage of driving simulators is training large number of drivers at the same time thus saving time and reducing costs. Driving simulators are used at research facilities for many purposes. Some vehicle manufacturers operate driving simulators.

In addition, driving simulators allow researchers to study driver behaviour under conditions in which it would be illegal and/or unethical to place drivers. For instance, studies of driver distraction would be dangerous and unethical (because of the inability to obtain informed consent from other drivers) to do on the road. With the increasing use of various in-vehicle information systems and interruptive items such as satellite navigation systems, cell phones, DVD players and e-mail systems, simulators are playing an important role in assessing the safety and utility of such devices.

In some countries touch screen computer system is currently used to take their theory test. The touch screen system is easy to use and the candidate selects the correct answers by simply touching a computer screen. Only one question is displayed on the screen at any time. In most systems the candidate will be able to move backwards as well as forwards through the test questions and he/she will also be able to change any of the answers he/she may feel need to be changed.

The advantages of computer touch screen theory test are reduced testing costs and elimination of human influence on marking the test. The computerized system also identifies the candidate using the finger print meaning that no one else can sit the theory test for the candidate.

8.3 Review of Current Situation in the World

8.3.1 Harmonization of Drivers Training and Testing in European Union (EU)

The standards required to obtain a driving licence vary in Europe and still vary between the EU Member States. Since the 2nd Driving Licence Directive adopted in 1991, there has been mutual recognition of driving licences throughout the European Union. The 3rd Driving Licence Directive adopted in 2006, imposed further harmonisation of the driving test within the European Union. In 2009, findings from the study were presented by the unit responsible for Energy and Transport in the EU with the objective to provide a framework to define guidelines and recommendations for efficient driver training and traffic safety education in the European Union[5].

The EU Driver Training Directive stipulates that all professional drivers, whether newly qualified or existing, of vehicles requiring driving licences in all categories of groups C and D (vans and trucks

[5] Driver training and Traffic safety education. A Consultation Paper. *Presented by the unit responsible for road safety policy of the Directorate-General for Energy and Transport.2009*

above 3.5 tonnes GVM, buses, coaches and minibuses) will be required by the legislation to hold not only a driving licence but also a Drivers Certificate of Professional Competence (CPC), which is renewable every five years. Driver CPC is a professional qualification to ensure professional drivers are familiar with best practices and the legislative requirements that affect them. Training is designed to confirm and expand on the existing knowledge and skills of each driver to ensure that they continue to be confident, safe and fuel-efficient drivers. This will also enable drivers to keep up to date with ever changing regulations and to benefit from state of the art training throughout their whole career[6].

In 2003 Commission Internationale des Examens de Conduite Automobile (CIEFA), which is an International Commission of Drivers Testing Authorities, conducted a workshop in Berlin on Harmonizing the Assessment Criteria of Driving Test in European Countries[7]. CIECA's Berlin workshop identified two competing tradition philosophies when it comes to assessing candidates during the practical driving test. The first, traditional, approach is a mistakes-based system, such as in the UK, where candidates start with full marks and are marked down according to the mistakes they make. The other tradition approach is a competence-based system, such as in Sweden, which focuses on strengths and a global appreciation of the candidates' driving ability, rather than his faults.

The tradition competence-based systems also take into account driving faults or errors, but not to the same level of detail as in tradition mistakes-based systems. Countries with competence-based systems are reluctant to base the test result for the candidate on fault categories for specific driving errors for reasons such as:

- every driving situation is different and pre-prescribed fault categories are insensitive to this;
- fault categories encourage examiners to look for faults rather than to look for competencies, and
- test candidates are encouraged – during training and during the test – to avoid making mistakes, rather than to develop or demonstrate their general safe driving competencies.

What the *competence-based system* looks from the practical test is the ability of the candidate to drive independently and in a way that is:

- Safe
- Cooperative
- Facilitates the traffic flow
- Considers health, environment and other people's needs and
- Conforms to existing laws and regulations

To fulfil the above competencies, the candidate must demonstrate a sufficient level in the following skills:

- preparation for driving
- vehicle manoeuvring
- observation and attentiveness
- communication with other road users

[6] Driver training and Traffic safety education. A Consultation Paper. *Presented by the unit responsible for road safety policy of the Directorate-General for Energy and Transport.2009*

[7] Harmonisation of the Assessment of Driving Test Candidates. CIECA Road Safety Charter working group. European Road Safety Charter. April 2006.

- positioning
- speed adaptation and adapting to traffic

In contrast, the mistakes-based system looks for the faults the candidate makes in the same areas the competence-based system look for abilities. The examiner of the mistake-based system must feel safe throughout the test for the candidate to pass. In certain circumstances, a single error will automatically lead to failure if the error is categorized as “dangerous error”. A dangerous error is one that immediately endangers the safety of the test vehicle, its passengers or other road users. It is the responsibility of the examiner to assess the gravity of the fault, or lack of specific driving skills, depending on the immediate circumstances during the test, according to country practice regarding the assessment of the driving test.

A standardized Euro-test based on the best practices of all member countries is planned to be implemented throughout the European Union. This planning is at an advanced stage. It is one of the conditions of entry for any aspiring member country that their driver testing will follow similar patterns.

8.3.2 Europe

In Europe there are many different types of licensing systems. They can be divided into three main categories[8].

- The traditional model
- pre-test learning period models
- post-test practical experience models

The traditional models offer the simplest approach to driver training; they include the test-only model, or driving school + test. The pre-test learning period models put more emphasis on building up experience before the test; they include the accompanied driving model and the graduated driver licensing model. The post-test practical experience models place emphasis on developing good habits and building up experience after the test.

United Kingdom (UK)

In UK, a written theory test replaced the previous informal question-and-answer session held at the end of the driving test in 1996. This is based on a multiple choice question paper and has to be passed before the candidate can sit the practical driving test. All of these systems are administered by the Driving Standards Agency (DSA), a division of the Department of Transport formed in 1990. DSA has its own central training establishment where initial and refresher courses are given to all examiners. Promotion courses and pilot exercises for possible improvements to test practices are a regular feature.

The British driving test is a test which all British learner drivers must pass to obtain a full driving licence. Different tests are available for users of different vehicles, from car drivers, to motorcyclists and HGV drivers. The test is separated into three distinct parts:

- a multiple-choice theory test;
- a hazard perception test; and
- the practical test.

It is necessary to pass all three parts before a full driving licence is granted. The multiple-choice test is performed on a touch screen computer system. For candidates of light vehicle licenses, the test

[8] Harmonisation of the Assessment of Driving Test Candidates. CIECA Road Safety Charter working group. European Road Safety Charter. April 2006.

has 50 multiple choice questions and the candidate must answer at least 43 of them correctly to pass. The test lasts for 57 minutes although candidates with certain special needs can apply for more time. For HGV and PSV driver candidates, the tests have 100 questions, with a pass mark of 85 and lasts for 1 hour 55 minutes.

In the hazard perception test examinees watch fourteen one-minute clips (nineteen clips for lorry and bus candidates) filmed from the perspective of a car driver and the applicant have to indicate, by clicking a mouse button or touching the screen, when they observe a developing hazard. The sooner a candidate reacts to a developing hazard, the more points are scored, from five down to one, with no score if the candidate reacts too late. The pass mark is different for different types of licenses applied by the candidate.

It is necessary to pass both components of the theory test before proceeding to the practical test. The practical test is taken on the road, with a professionally trained DSA examiner directing the candidate around a pre-determined route. The examiner marks the candidate for driving faults, serious faults, and dangerous faults. A candidate will fail the test if he or she accumulates any serious or dangerous faults, or more than fifteen driving faults. If a candidate accumulates several driving faults in the same category, the examiner may consider the fault habitual and mark a serious fault in that category. The test usually lasts 38 to 40 minutes for light vehicles driver candidates, or approximately 70 minutes when the candidate is taking an extended test after having had their licence revoked. The test lasts for about 90 minutes for HGV and PSV driver candidates.

Before getting to the car, the examiner will ask the candidate to read a car number plate at a distance of approximately 20 metres. If the candidate fails to read the first number plate correctly, then the examiner asks the candidate to read a second number plate. If the candidate cannot correctly read the second number plate, then the examiner must use a tape measure to measure the correct distance between the candidate and a third number plate. If the candidate cannot read the third number plate, then the candidate is deemed to have failed and the test will not continue. If the candidate needs glasses to read the number plate then these must be the ones worn whilst completing the rest of the test.

Before the candidate is taken out onto the road, the examiner asks two questions about car maintenance and safety. A failure to answer one or both of these questions correctly would result in a driving fault being marked against the candidate. The questions that may be asked are changed from time to time.

The controlled stop, more commonly referred to as the "emergency stop", is an exercise which determines the ability of the candidate to stop the vehicle promptly yet under control during an emergency. The simulation is performed by the examiner raising his or her hand and saying, "STOP!"

During the test, the examiner will also ask the candidate to carry out one manoeuvre from the following list:

- Turn in the road (3 point turn)
- Reverse around a corner
- Reverse park into a space either parallel, oblique, or right-angle in a marked bay in an off-road car park

Manoeuvres are selected at random by the examiner depending on the route chosen and conditions on route.

Generally, the candidate must demonstrate an ability to drive in various road and traffic conditions and react appropriately in actual risk situations. The conditions typically encountered on test include driving in urban areas as well as higher speed limit roads where possible; this includes dual carriageways but not motorways as motorways in Britain can only be used by full licence holders. The object of the test is to ensure that the candidate is well grounded in the basic principles of safe driving, and is sufficiently practised in them to be able to show, at the time of the test, that they are a competent and considerate driver and are not a source of danger to themselves or to other road users.

Many driving instructors offer tuition in automatic cars as well as manual cars, or may specialise in automatic driving lessons. If a learner passes the driving test in an automatic, then the full licence granted to them will entitle them to drive only automatic transmission cars or any other type of car which has only two pedals and no manually operated clutch. The full automatic licence acts as a provisional licence for manual gearbox cars.

Denmark

In Denmark, a new system of mandatory driver education was implemented in 1986. The new system emphasized:

- defensive driving;
- basic skill training on a closed course;
- progressively more difficult training on the road and at night;
- classroom instruction in conjunction with practical training, and a written test.

An evaluation study was conducted in 2009 to evaluate the effect of mandatory training to new drivers. The annual number of crashes was examined for a total of 11 years before and after the program. Results indicated that for the same age group, the drivers affected by the mandatory training experienced a 35% decrease, compared with a 17% decrease among the comparison group. Thus the new program appeared to have a positive effect[9].

Sweden

Sweden has improved the traditional drivers training by incorporate elements that help drivers gain insight into risk factors and their own skill limitations. During training drivers are exposed to various driving situations at an off-road centre, up to 20 exercises such as assessing stopping distances and estimating safe driving distances, and undergo a process of self-diagnosis facilitated by group discussions.

Finland

A new program used in Finland includes a compulsory course for drivers with 6 to 24 months of driving experience, with the focus on self-awareness and little emphasis on skill or vehicle control. The program involves a mixture of on-road and off-road driving, one-to-one feedback and group sessions, and facilitated discussions on risk identification and management.

8.3.3 Asian and Pacific Countries

Australia

Two schemes are used to obtain driving licence in South Australia: The Vehicle on Road Test (VORT) and Competency-Based Driver Training and Assessment (CBTA) schemes. CBTA involves multiple training sessions spent by the learner driver with an Accredited Driving Instructor with

[9] Allan F. Williams, David F. Preusser and Katherine A. Ledingham. (2009). Feasibility Study on Evaluating Driver Training

assessment proceeding on a cumulative basis. When all of the nominated tasks have been completed successfully, the driver is issued with a Certificate of Competency which enables him or her to obtain a Provisional Licence from a Transport South Australia Customer Service Centre. The VORT is similar to the traditional approach to obtaining a Provisional Licence. It comprises a single practical test of the same skills tested for in the CBTA course. It is conducted mainly by authorised instructors over a standard test route.

An evaluation study to compare the effectiveness of the VORT and CBTA schemes on road safety was conducted by Road Accident Research Unit of University of Adelaide to determine whether there is a meaningful difference between them in terms of the rate at which recently licensed drivers are involved in crashes and moving traffic offences. The general conclusion was that CBTA appeared to have little effect on the rate of crash involvement in the first 12 months of driving on a Provisional Licence, compared with drivers who took the VORT, but it did appear to reduce the rate of traffic offences[10].

A program offered by the Royal Automobile Club of Victoria, Australia[11], (Parents Plus), combines parents and professional instructors in the management of pre-license driving, starting with an initial driving lesson where a parent is invited to join the learner and instructor. The intention is to encourage parents to ask questions and request advice on how to manage supervised driving instruction, motivate the provision of 120 hours of supervised driving, and introduce parents to support and guidance materials on providing supervised on-road experience.

China

Each province in China has designated driving schools and these normally have large off-road areas with networks of roads, intersections, parking practice areas, and ramps for hill starting. They also own fleets of vehicles that are used for driver instruction and trainees must pass a series of modules as part of their training. The test itself is carried out by traffic police examiners who visit the driving schools by arrangement to conduct the test. Some of the test is conducted on the off-road facility and if the driver performs satisfactorily, the remainder of the test is undertaken by driving 15-20 minutes on the public roads in normal traffic[12].

Fiji

In Fiji, the driving test is conducted largely on the normal public road network and only parking and a few other basic manoeuvres are tested off-road. A recent funded road safety project implemented improvements such as the introduction of a new theory test that could be conducted orally and had a preset pass mark, and standardized test routes. Training was also given to all driving examiners, and monitoring system has been improved and checks have been imposed on driving examiners. Efforts have also been made to strengthen driving schools in Fiji by developing a code of practice and standardized training curriculum. A defensive driving course was also developed and many driving instructors were trained as defensive driving instructors.

Kazakstan

Driver training in Kazakstan is undertaken by a mixture of organizations with the best training schools being well-equipped with classrooms containing displays of traffic signs, and technical

[10] Kloeden CN and McLean AJ (2000). Evaluation of Competency-Based Driver Training and Assessment In South Australia. A Report for Transport SA. Road Accident Research Unit. The University of Adelaide

[11] Allan F. Williams, David F. Preusser and Katherine A. Ledingham. (2009). Feasibility Study on Evaluating Driver Training.

[12] Road safety guidelines for the Asian and Pacific region. Asian development Bank. 1998.

displays of vehicles, components, and systems. Students take about three months to complete the training, which includes 190 hours of theoretical training plus eight hours on a driving simulator and 22 hours driving a vehicle. However, in the test only 10 theory questions are asked and a short practical test, lasting about 15 minutes, is given.

Singapore

In Singapore, special large off-road areas are available containing a network of roads with various types of junctions, ramps, and parking practice areas for learner drivers to practice on. During testing of drivers part of the driving test is carried out off-road at these locations and part of the test is conducted on the road network in normal traffic.

8.3.4 United States of America (USA)

The standard driver education in the USA has long been based on the 30 classroom hours +6 hours behind the wheel formula for learner drivers[13]. The 30+6 formula fit the high school curriculum format, and for many years, driver education was primarily taken in high schools, or in commercial courses, which in most cases mirrored the high school format. Traditional methods of education – classroom instruction and on-road training – continue to be used widely. However, commercial programs are more varied, and developments in computer technology have led to changes in the way driver education is delivered, featuring simulator technology and computer-assisted learning, often involving interactive programs.

The American Driver and Traffic Safety Education Association (ADTSEA) program expands the standard 30+6 formula to 45 hours of classroom and 8 hours of driving instruction. The ADTSEA program was introduced in the States that have graduated licensing systems. These systems may provide some motivation to apply safe driving practices, since many of them have contingent advancement provisions in which progress toward graduation is halted or reversed if traffic violations or accident occur. The ADTSEA program has a parent component encouraging parent involvement in the licensing process in providing supervised driving practice and enforcing graduated licensing rules.

From 1970's many studies have been conducted in USA to evaluate drivers training programs, and to develop curriculum and teaching methods which will produce safer drivers. The National Highway Transportation Safety Administration (NHTSA) was urged to develop a driver education program and evaluate it in a way that would be definitive. This long-term research and development program took place in the late 1970s and early 1980s. It involved, among other exercises, a thorough analysis of the driving task. The tasks rated as having high or moderate importance were used as the basis for a new driver education program called the Safe Performance Curriculum (SPC).

The SPC was the most advanced driver education program ever developed in the USA, and involved over 70 hours of instruction allocated between classroom, simulator, closed-course behind-the wheel training, and on-road training, including some at night. The total hours and behind-the-wheel experience far exceeded the typical driver education course. These training included:

- basic vehicle control skills;
- environmental factors that affect driving;
- complex perceptual skills to avoid risky driving situations;
- driver impairments;
- emergency situations, and

[13] Allan F. Williams, David F. Preusser and Katherine A. Ledingham. (2009). Feasibility Study on Evaluating Driver Training.

- personal readiness.

A second driver education program, the Pre-driver Licensing curriculum (PDL), was also developed and evaluated. PDL provided minimal training in skills required to pass a license test, including about 20 hours of classroom, driving range, and simulation instruction. PDL students received only one hour of on-road instruction. The total hours of instruction and on-road experience were thus substantially less than in the standard high school driver education course (30+6). Special teachers were hired specifically for the driver education courses. They were highly trained, motivated, and closely supervised. They taught both the SPC and the PDL courses to minimize instructor influence on the evaluation outcome. The two courses were taught at four training centres, designed and built specifically for the project.

8.3.5 Harmonization of Drivers Training and Testing in SADC Region

In 1996, SADC member states signed a protocol[14] to harmonize and enhance the overall quality of road traffic in the region with the emphasis on promoting acceptable levels of safety, security, order, discipline and mobility on the roads and protecting the environment and road infrastructure. With regards to training and testing of drivers member states agreed to develop and implement harmonized standards to the training and testing for learner's and driving licences and professional driving permits. Other items that were agreed to be developed and be harmonized include:

- a curricula for training drivers;
- examination and testing procedures;
- inspection and certification of drivers testing facilities;
- training and certification of examiners; and
- training and certification of driving instructors.

In the protocol it stipulates further that each SADC Member State shall issue, upon successful completion of the relevant test in respect of a certain licence code, a driving licence for the categories to be agreed upon. Other agreements include adoption of harmonized format for driving licences and recognition of driving licences issued according to the agreed SADC codes and format by other Member States.

Furthermore, a licence issued in one Member State shall be valid for the driving of a vehicle in respect of which it has been issued in any other Member State. In addition, the provision stated that Member States shall harmonize learner's licence testing and codes and professional driving permit with a view to encouraging the mutual recognition of each other's learner's licences.

Currently, Transaid is working with the Federation of East and Southern Africa Road Transport Associations (FESARTA) and SADC to establish harmonisation of driver training standards across the region[15].

8.3.6 Africa

A study conducted in selected countries in Africa revealed that driver error is the leading cause of accidents[16]. It is also known that in developing countries, a large proportion of drivers learn to drive

[14] SADC (1996) SADC Protocol on transport, Communications and Meteorology, SADC Secretariat, Gaborone.

[15] Transaid. (2010). Raising Professional Driver Training Standards; Transaid's Support to the Industrial Training Centre (ITC)

[16] Downing, AJ. (1989). Driver training in Africa: 2nd African Road Safety Congress. Addis Ababa 1989

in apprenticeship with other drivers. As a result, a significant number of drivers on the roads have not attended the formal system of training through driving schools.

The problems of poor knowledge and driver behaviour in developing countries have been described to likely be due to some extent, on inadequacies in driver training and testing[17]. Professional driving instruction tends to be limited because:

- driving instructors are not properly tested or monitored;
- there is no driving or instruction manuals; driving test standards and requirements are inadequate.

Research by Downing (1989) noted that, the methods and procedures of driver training and testing in Africa as a whole have often not kept pace with improvements in developed countries. Many countries do not test or monitor their driving instructors nor is there a manual for drivers or instructors. There is then considerable scope for raising driving standards in Africa by improving driver training. However the improvements to be considered must be appropriate for the driver's abilities and conditions prevailing in Africa.

The United Nations Economic Commission for Africa (ECA) developed a guide for heavy goods vehicle (HGV) drivers with the assistance of the Transport and Road Research Laboratory (TRRL), UK. The objective of the guide is to provide and explain a standard of driving and related matters which will lead to:

- Improving driving standards
- Improving instruction standards
- Increasing harmonisation of driving and instruction standards throughout Africa.
- Fewer road accidents.

The guide was made into modules for different groups rather than a single comprehensive manual. The reasons for this were as follows:

- Most African countries already had comprehensive highway codes.
- Although these codes were aimed at all road users, in practice they were usually only read by learner drivers applying for a licence. Therefore the benefits for other groups were likely to be minimal.
- Individual guides prepared and targeted at specific groups were likely to be more effective in raising standards of knowledge and behaviour than a multipurpose code.

Having opted for a modular approach, the Unit agreed with the ECA that for the first phase of the African Highway Code work it would develop a guide for heavy vehicle (HGV) drivers. This target group was chosen for the following reasons:

- It was a clearly identifiable group of drivers who were relatively easy to communicate with through fleet operators and their associations; and
- It was an important group economically and HGV accounted for a reasonable share of the traffic in Africa.
- A large number of HGV drivers operated across national borders and therefore an international driving guide was more appropriate to this group.

The guide was aimed for both learner and qualified drivers who are literate and it is being produced in English and French.

[17] C J Baguley and G D Jacobs. Traffic Safety Issues for the Next Millennium. Transport Research Laboratory.

South Africa

South Africa has one of the highest road accident fatalities in the world. Over the 12-month period from 1 January 2009 to 31 December 2009 the number of fatal crashes reported was 10,857 with human factor contributing 82.85%, vehicle factor 9.13%, and road and environment 8.02% to fatal crashes during 2009[18].

Despite the high rate of accident, South Africa has better institution arrangements and higher training and testing standards compared to most African countries. These include:

- Driver testing and licensing are unified across the country , well managed , and centrally controlled;
- Driver testing and licensing starting from learner to driver are managed by one institution;
- Computerised systems are used in testing ,licensing and record keeping;

Furthermore, proposed SADC driving licences codes are used. These are:

- A and A1(≤ 125 cc)-motor cycles
- B –Light Motor Vehicles (GVM ≤ 3500 kg) with a Trailer ≤ 750kg
- EB –Light Motor Vehicles (GVM≤ 3500 kg) with a Caravan
- C1 -Heavy Motor Vehicles (GVM≤ 15000kg)
- EC1 – Heavy Motor Vehicles (GVM≤15000kg) with a Trailer
- C – Heavy Motor Vehicles (GVM >16000 kg)
- EC –Heavy Motor Vehicles with trailer (GVM>16000 kg)

It is well known that the majority of deaths on South African roads are caused by human error. The Driver Training Section of Department of Transportation in South Africa has drawn up the standards for driver training and the testing of applicants for their drivers' licences (K53). It has also trained instructors in the various traffic training colleges to train their own examiners according to the prescribed standards. The Driver Training Section is also responsible for the evaluation and grading of all Licence Testing Centres in conjunction with the provincial inspectorates. Both facilities and examiners are evaluated to ensure that the appropriate standards are adhered to.

The first investigation into corruption and malpractice in the issuing of learners' and drivers' licences was carried out by the South African Police Service in KwaZulu-Natal, who are looking at allegations of serious irregularity regarding 35 driving schools, 5 examiners and 3 000 licences over a nine month period. It is alleged that licences were being sold for up to R1 400 each.

In South Africa, learner's licences are issued by the licensing authority to customers who pass the learners test. The testing is based on a syllabus followed by all learning to drive. The country has started using computerised touch screen testing where human influence on marking is eliminated. The computerized system also identifies the candidate using the finger print meaning that no one else can do the theory test for the candidate. The information issued to learner driving licence is electronically captured on the national data base.

Only candidates who have passed the learner's test and are in possession of learner driver license are allowed to do the Driver Licence testing. Driver testing is conducted by qualified examiners following the set examination procedures. With the exception of Motor Cycle, where testing is done only on the yard, tests for motor vehicles are also carried out in designated public roads. Yard testing is done on the uninterrupted environment where the standard markings and obstacles are set for conducting the yard test. It consists of intersections, stopping, overtaking and lane Change.

[18] www.arrivealive.co.za

Pre-trip inspection is conducted prior to the yard test. Testing is terminated at any stage where a candidate fails a critical testing. Road testing can proceed only where a candidate has passed all yard testing.

K53 Licensing System was a project reference number initially given in the 1980's, to the development of a standard means of measuring driver proficiency which was based on the UK system. K53 comprised of three volumes, one for light motor vehicles (Vol.1), one for heavy vehicles (Vol. 2) and one for motor cycles (Vol. 3). The system for testing heavy vehicle drivers was officially implemented on 1 January 1992. A fourth volume was added to provide for the testing of light motor vehicle combinations Vol. 4).

Another important addition was the introduction of a minimum vehicle length of 3 metres for a light motor vehicle test (category B) and 6 metres for a heavy vehicle test (categories C1, C). The level of proficiency required to pass the heavy vehicle yard test was significantly increased by reducing the number of allowable penalty points from 50 down to 20. Both practical tests are guided by detailed prescribed Modules which are contained in K53 volumes. For example, there are 53 Modules for heavy vehicles testing as listed below[19:]

Module1.	Pre-trip inspection - interior
Module 2	Pre-trip inspection - exterior
Module 3	Starting procedure - manual transmission
Module 4	Starting procedure - automatic transmission
Module 5	Mirrors - use of
Module 6	Signalling
Module 7	Signalling - hand signals (turning left)
Module 8	Signalling - hand signals (turning right)
Module 9	Signalling - hand signals (stop or sudden reduction of speed)
Module 1	Signalling - horn
Module 11	Clutch - use of
Module 12	Moving off - manual transmission
Module 13	Moving off - automatic transmission
Module 14	Steering
Module 15	Left turn
Module 16	Reverse in straight line
Module 17	Alley docking - to the right
Module 18	Incline start - manual transmission
Module 19	Incline start - automatic transmission
Module 20	Speed control
Module 21	Gear changing - up (manual transmission)
Module 22	Gear changing - down (manual transmission)
Module 23	Gear changing - up (automatic transmission) (manual selection)
Module 24	Gear changing - down (automatic transmission) (manual selection)
Module 25	Following other vehicles
Module 26	Lane changing
Module 27	Stopping - in traffic (manual transmission)
Module 28	Stopping - in traffic (automatic transmission)
Module 29	Stopping - for parking (manual transmission)
Module 30	Stopping - for parking (automatic transmission)

[19] www.foresightpublications.co.za

Module 31	Traffic control signals
Module 32	Intersections - turning left
Module 33	Intersections - turning right
Module 34	Intersections - proceeding straight
Module 35	Intersections - stop signs
Module 36	Intersections - yield signs
Module 37	Intersections - uncontrolled
Module 38	Intersections - traffic lights (flashing red)
Module 39	Intersections - traffic lights (steady red)
Module 40	Intersections - traffic lights (green)
Module 41	Intersections - traffic lights (flashing amber)
Module 42	Intersections - traffic lights (steady amber)
Module 43	Intersections - roundabout
Module 44	Block pedestrian crossing -uncontrolled
Module 45	Level crossing - guarded
Module 46	Level crossing - unguarded
Module 47	Overtaking - to the left of a hazard
Module 48	Overtaking - to the right of a hazard
Module 49	Being overtaken - on the left hand side
Module 50	Being overtaken - on the right-hand side
Module 51	Freeways - entering
Module 52	Freeways - exiting
Module 53	Freeways - passing off and on ramps

An evaluation study conducted at the University of KwaZulu (Natal) observed areas of flaws in the K53 licensing system[20]. It reports that the K53 licensing system has loopholes that undermine it and therefore undermine the whole process of driver socialisation. It further pointed out that the problems lie with the ways in which the modules are taught and tested on as well as the way in which the system is structured which negatively impact on driver socialisation and driver behaviour thereafter. Some of the shortcomings reported in the study include the following:

- there does not seem to be a specific procedure in place to regulate driving schools, for example there are many driving schools in the Durban area that are not registered and are therefore illegal, but they continue to operate;
- It is up to each individual driving school to determine the minimum driving lessons that learners can take before the test;
- It is up to the inspector concerned on how one is tested and the manner in which the road test is conducted is dependent on the inspector;
- There seems to be no uniformity in the testing process and no third party quality control;
- There is also too much emphasis on sequences. Although one may see its importance, there does not seem to be a need why it has to be carried out in a particular way. If there is a need then it is not explained. This brings us to the lack of explanations of rules and regulations.
- At the moment the emphasis seems to be on memorising the rules and reproducing them at the test. There is no process in place that ensures that learners not only learn the rules, but also understand the reasons behind the rules.

[20] Thokozani Nkomonde: "Passing the Test:" A Critical Evaluation of Formal Driver Education In South Africa. University of KwaZulu Natal

Another concern which was observed in the study is that, if one is in possession of a driving license for heavy vehicles, then they may also legally drive a light motor vehicle. This does not take into consideration that there are certain skills that one has to learn to drive a light motor vehicle that are specific to that class of vehicles. Motorcycle drivers may also drive legally without driving licenses without supervision because of a loophole in the system.

Among others, the study recommended revision of K53 licensing system to adopt the Graduated Driver Licensing, in which learner and new drivers have restricted and temporary licences for a period of at least a year into the driving career. During this period they are not allowed to drive in certain conditions unless under supervision with a person with a license of the same class or above.

Zambia

Transaid is an independent UK non-government development organisation working in the transport sector of several African countries. In August 2008, Transaid partnered with the Industrial Training Centre (ITC) in Zambia. The ITC is a government training institution mandated to provide automotive and electrical engineering, driver training and other vocational programmes. Transaid partnered with the ITC to build their capacity and improve HGV driver training so that the trainers were able to train to internationally recognised standards. Prior to this intervention there was no formal way for a driver to access quality HGV driver training in Zambia. The project was set up with support from a UK consortium of Transaid member companies from the transport and logistics industry who provided funding, resources and expertise[21].

In the first two years the project worked to develop and coordinate inputs and appropriate shared best practice. This included work on vehicle and classroom based skills, teaching techniques, increasing driver training expertise and institutional capability. Six trucks, two trailers, a bus and a forklift truck were donated to the project. The project also focused on training of trainers to enable skills to remain in country ensuring project sustainability.

Six driver trainers have been trained to deliver HGV driver training to an internationally recognised standard, including one senior driver trainer. By 2010, over 1000 participants were trained at the ITC, including those taking refresher courses. The training has been delivered to public organisations, government departments and local transport operators. In addition to the training of drivers, training of transport managers was also conducted.

An International professional driver training study tour was arranged in 2012 to take driver trainers from Zambia to see best practice transport operations in the UK. In addition, Transaid has facilitated exchanges between the ITC in Zambia and the National Institute of Transport (NIT) in Tanzania to share skills relating to driver training across these two key government institutions and to support with the harmonisation of standards across main trade routes. Table 1 shows categories of trainees and training conducted by ITC since the formal Transaid partnership and provision of training vehicles between January 2009 and March 2012.

[21] Transaid. (2010). Raising Professional Driver Training Standards; Transaid's Support to the Industrial Training Centre (ITC)

Table 1: Training achieved by Transaid in Zambia between January 2009 and March 2012

Class of training	Total number of drivers trained	Course duration
Truck/Bus	1,259	3 - 4 weeks
Refresher and & dangerous goods training	1,144	3 -4 days
Motorcycle	1,102	10 days
Driver assessment	1,239	1 day
Total number. Trained	4,744	

In addition to working with the ITC, Transaid has been working with the Zambian government Road Transport and Safety Agency (RTSA) to advise on policies relating to truck and bus driver training. This has included issues surrounding driver training and testing, working time directive, legislation and the potential introduction of breathalysers. The RTSA have ruled that anyone wanting to obtain a HGV driving license in Zambia must now provide evidence of having received training at the ITC, which is a positive step towards enforcing quality driver training standards.

8.3.7 Trend in Vehicle Technologies

Driving an automatic car is easier than driving a manual car because you don't have to learn how to change gears; the car does all the work for you. Vehicle transmissions are generally categorized into manual transmissions (MT) and automatic transmissions (AT). Other versions of AT also exists including continuously variable transmissions (CVT) which has an outstanding advantage in fuel economy. Recently, the DCT (Dual Clutch Transmission), which may be regarded as the third generation automatic transmission system following has been adopted by vehicle manufacturers.

The general public perception in many countries other than North America had been that the AT vehicles have poor fuel consumption, inferior acceleration, poor reliability, and higher price. Many drivers who have years of experience with MT vehicles had also claim to feel uncomfortable and less in control when driving AT vehicles. On the other hand the main reasons for people to opt for AT vehicles had been:

- Easy to drive
- Increased safety as driver concentrates on steering and braking only
- Protection of *power-train* system by abusive and careless drivers.
- Swift gear changing with no jerking during gear changing no loss of torque transmission during gear changing

Over the years there has been increasing technological improvements on the AT systems which have caused rapid penetration of AT vehicles in the markets around the world, even for bigger trucks, The improvements have been largely catalyzed by Government's legislations on environmental issues which spearheaded vehicle manufacturers to engage in extensive development of power-train management systems on vehicles which limits driver control and are easier to achieve with AT systems. The current trends common to these transmission types include use of low-viscosity transmission oil, torque converter lock, and increasing number of gears.

The modern AT vehicle has better fuel economy than MT vehicle and are more reliable. AT vehicles having up to 8 gears are now manufactured which has caused even heavy trucks with AT to be available in the market. Truck fleet operators have reported substantial drop in accident rates because drivers can concentrate on traffic situation and road conditions and upcoming hazards instead of when to change the gear and what gear to be in.

Fleet operators have reported that veteran drivers at first don't like AT, but soon change their minds when they see how much work they save, and some decided to delay retirement because they can easily cope with the physical demand of AT trucks. Urban congestion is another reason for increasing trend in AT vehicle in cities around the world as it does become tiring to drive a vehicle with a manual transmission in bumper-to-bumper traffic. Figures 1 shows the increasing trend and the predicted trend of trucks with AT in Europe from 2005-2015. Figure 2 shows the percentage of AT vehicles in the world.

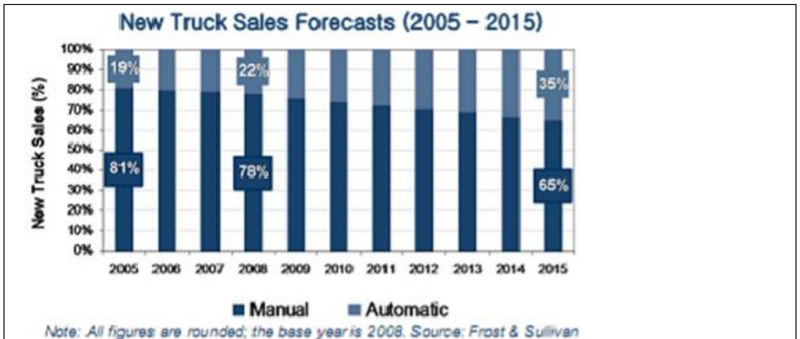

Figure 1: Trend in the population of AT trucks in Europe[22]

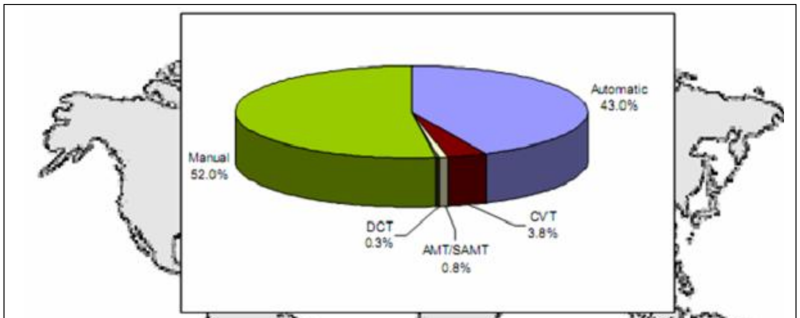

Figure 2: Trend in the population of AT vehicles in the World[22]

Other advancements in vehicle technology include Anti-lock Brake System (ABS), Electronic Brake Distribution (EBD), traction control system (TCS), electronic stability control (ESC), cruise control, hybrid and electric vehicles. Despite the fact that most of these advanced systems do not affect the drivability of the vehicle to meet normal traffic road rules but in emergency situation the driver have to be aware of the abnormal behaviour of the vehicle with such system. A good example is ABS, where in a vehicle without ABS the driver is trained to pump the brake in slippery road conditions, whereas, with ABS the driver is advised to maintain brake force steadily to allow ABS to work.

Despite the fact that vehicles equipped with AT are less complex to drive but they need different procedures and techniques during starting, driving in slow traffic, and in mountainous roads. In

[22] Frost and Sullivan. (2010). Global Commercial Vehicle Power-train Systems market

some countries around the world, drivers who have passed their driving test in a vehicle with an automatic transmission will not be licensed to drive a manual transmission vehicle. Conversely, a manual license will allow the driver to drive both manual and automatic vehicles.

Examples of driving license restrictions (Automatic Only) when the candidate pass driving test in AT vehicle are Croatia, Dominican Republic, Israel, United Kingdom, some states in Australia, France, Portugal, Latvia, Lebanon, Lithuania, Ireland, Belgium, Germany, Pakistan, the Netherlands, Sweden, Austria, Norway, Hungary, South Africa, Trinidad and Tobago, China, Hong Kong, Macau, Mauritius, South Korea, Romania, Singapore, Philippines, United Arab Emirates, India, Estonia, Finland, Switzerland, Slovenia, Republic of Ireland and New Zealand[23]

8.4 Review of Current Situation in EAC Countries

8.4.1 Institutional Arrangement

According to Tanzania Road Safety Policy[24] there have been a number of government-led initiatives geared towards improvement of road safety in the country. Notable initiatives include: Involvement of the National Institute of Transport (NIT) and the Vocational Education and Training Authority (VETA) in driver training in the early 1990s. In 1996 Tanzania passed legislation for compulsory driving training in certified training schools before being tested for licence[25]. Driver's licensing system in Tanzania has many shortfalls that are contributing to the increase of bad drivers on the roads. These include:

- Corruption in the process to get driving licence
- Poor quality of vehicles used in the driving schools
- Poor standard of training in certified schools
- Training is directed more at passing the test and getting the licence.

The National Road Safety Policy proposed the establishment of Driver and Vehicle Examination and Licensing Agency (DVELA). The objective of DVELA is to promote good driving standards in the country; and ensure the use of road worthy vehicles on the roads. To achieve its objectives DVELA shall have the following functions in relation to improving the quality of drivers on the roads:

- establish standards and methods for the training and testing of driving instructors and drivers of motor vehicles and riders of motor cycles;
- establish standards and methods for the training and testing of vehicle examiners;
- provide syllabi for driver training and the training of instructors;
- issue driving licences;
- register and license driving schools;
- license driving instructors;
- maintain registers containing particulars of licensed motor vehicles, driving instructors, driving schools and drivers of motor vehicles;

To date, DVELA has not started to perform its duties, and at present, Traffic Police and Vehicle Inspectors function as both driving examiners and vehicle inspectors. Driver licensing is carried out by the Tanzania Revenue Authority (TRA) and a traffic law enforcement institution. The two institutions are typically not situated at one place. Customers normally have to move documents

[23] www.wikipedia.org

[24] National Road Safety Policy. Ministry of Infrastructure Development. Dar es Salaam. 2009

[25] Cuthbert W. Chiduo and Philemon Minja. Road Safety in Tanzania: What Are the Problems?

from one place to another, with no document control by the overseeing authorities. The licensing done by the tax collection institution has basically no relationship with vehicle safety considerations. In addition, the mix of powers and functions contributes to an unstable situation whereby the same institution on one hand tests drivers before they are issued with driving licences and on the other hand monitors the violations made by the drivers.

Prior to issuing the full driving licence, the Traffic Department issues a certificate of competency to applicants who pass both oral and practical tests. Thereafter, the applicant pays a fee at TRA and obtains a licence which he/she has to return it to Traffic Department for endorsement and record keeping. It has been observed that the process has loopholes for corruption for applicants to get a driving license without being trained or even being tested. Furthermore, record keeping is poor and not reliable because[26]:

- The book where driving licences are recorded is not kept in the safe place.
- The recording is hand written and there is no backup should anything happen to the book.
- There was no dedicated person for capturing such crucial and delicate information, meaning no one to take full responsibility should something go wrong.

In 2010 Tanzania introduced the SADC driving license system which replaced the old system[27]. The objective of the new system is to monitor and control issuance of driving licenses with a view to address the problem of fake licenses which are the causative for many road accidents. Apart from creating electronic database to avoid double allocation of serial numbers and help monitor drivers, the smart card licence cannot be forged because it bears specific features including water marks and fingerprints. Other features are: the driver’s name, date of birth, licence number, date and place of issue, expiry date, Tax Identification Number (TIN), gender, address and signature.

8.4.2 Current Situation on Driver Training and Testing in EAC Countries

Driver training in Tanzania is provided by the driving instructors normally at driving schools both approved and accredited by the Traffic Department .Driving instructors are issued with formal permit allowing them to train drivers on and out of public roads. Driving schools are commonly privately owned and dedicated to driver training.

Some driver training schools are under institutions such as VETA. Most of driving schools do not have formal training grounds apart from public road. Few training schools have very basic training facilities; a good example is one of VETA in Dar es Salaam which has a training ground in the sub-urban area of Dar es Salaam. The training ground is gravel surfaced with basic facilities[28].

Before getting a full driving licence, the applicant is issued with a Learners License if the applicant displays a good knowledge of the rules of the road, road signs; and if the applicant is not suffering from any disease or any other disability which, in the opinion of the examiner, would render him incapable of driving and controlling a motor vehicle without endangering the public safety. The learner driver has to drive under supervision until one is issued with the driver’s license. The holder of a learner license is not allowed to drive on the public road a vehicle carrying passengers, for hire, or drive a vehicle without affixing the standardized plates or discs to the front and rear of the motor vehicle unless he has another valid driving license of other class.

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[26] Afrisa (2011). Standards for Driver Training and Licensing Centres. Project Reference No. 10037/2008/2009. Ministry of Works. Tanzania
[27] http://dailynews.co.tz
[28] Afrisa (2011). Standards for Driver Training and Licensing Centres. Project Reference No. 10037/2008/2009. Ministry of Works. Tanzania

The driving school is required to train a learner on traffic signs, and rules. And only after the learner has passed that training are they allowed to be trained on driving the vehicle .This process is entirely controlled by the driving schools with no policing and monitoring from the authorities. Currently there are no standardised testing facilities (Grounds) to be used for testing drivers in Tanzania. Traffic department make own arrangement for areas to be used for testing drivers, predominantly on public roads. Yard testing which is supposed to be done in un-interrupted condition is also not done in the standardised manner.

In 2002, the Bureau for Industrial Cooperation (BICO)[29] were assigned a project by the Ministry of Works to propose standards and procedures for driver training, testing, examination and licensing of light and heavy vehicles. The proposed driver training is based on the concept of defensive driving.

After carrying out training in driving, the holder of the learner driving license may apply on the prescribed form to an examiner for a certificate of competency to drive motor vehicles of any class. The examiner set a date for testing after receiving the prescribed fee, learner driving licenses, application form and other documents as prescribed. The examiner in the traffic authority in the District or Regional Headquarters shall process the applications for the driving test. The driving test covers[30];

- How the driver controls the vehicle, including smooth operation and correct use of the gears, clutch and brakes.
- How the driver comply with the rules of the road and road signs, signals and markings.
- How to use defensive driving systems including observation, signalling, road safety and coping with the road traffic problems.
- How to demonstrate courtesy with other road users.

The study by BICO proposed that the applicant should sit for basic written theory test of multiple choice format. The test should contain 45 questions. The test should be of 45 minutes. If the person cannot read the examiner can read, and then the examiner should read the questions and the applicant would select the answer. In this case the test should take an hour. The practical test includes:

- Pre-trip inspection of the vehicle;
- A yard test: reversing, parking, moving off on an inclined and;
- A practical driving test in traffic otherwise known as road test.

The pre-trip inspection and the yard test must be completed in 20 minutes if applicant passed pre-trip inspection, then the road test on public roads must be completed between 20 and 45 minutes. If the driver fails the yard test, the test should be stopped and the driver has to apply for the test on a later stage. In general, the practical test will be stopped immediately and the candidate deemed failure in the event of violation of traffic law; uncontrolled or dangerous action; and collision or mechanical failure. Specifically the candidate can be failed on any of the following:

- A violation of any traffic law, road sign, signal or road marking
- An uncontrolled, dangerous driving
- An avoidable collision
- Bumping obstacles during alley docking
- Mounting the kerb during parallel parking
- Mounting the kerb during turning in the road (three-point turn)

[29] BICO (2002). Standards: Driver Testing, Examination and Licensing. Ministry of Works.
[30] BICO (2002). Standards: Driver Testing, Examination and Licensing. Ministry of Works

- More than two attempts of parallel parking from each side
- More than two attempts of reverse parking (alley docking) from each side
- More than two attempts at an emergency stop
- Rolling back on an incline stop
- Rolling back when moving off; More than three movements during turning in the road (only one attempt allowed)
- Inability to perform a hand signal when instructed to do so

The candidate is also not allowed to do the practical test if using a vehicle that is not licensed and if it is not roadworthy.

For heavy vehicles there are additional manoeuvres which the candidate has to perform to pass the test. The test must be administered in an area closed to normal traffic. It should contain provisions for left turn, alley docking, and incline start. The road test shall be conducted on a pre-established route with the following features;

- A tarred multi-lane road containing at least two controlled intersections and clearly demarcated road marking.
- At least one intersection controlled by a four-way stop sign
- At least four intersections controlled by stop signs or robots.
- At least two intersections controlled by yield signs, one with the right of the way at one of the intersections and other without the right of the way.
- A quiet street where the emergency stops can be executed
- At least two intersections with two way traffic should be crossed with a right turn.

Transaid which has been working with the Zambia government through Industrial Training Centre (ITC) since 2008, teamed up with Tanzania's National Institute of Transport (NIT) in 2010 to build its capability to deliver quality Heavy Goods Vehicles (HGV) and Passenger Service Vehicles (PSV) driver training.

The NIT and Transaid (together with key stakeholders) are in the process of developing curriculum for PSV with the aim of increasing the levels of competency amongst PSV drivers in Tanzania. The PSV programme objectives are for the drivers to:

- Drive the vehicle safely and responsibly.
- Comply with the Road Traffic Act,
- Comply with all road signs, signals & markings,
- Ensure good Customer Care is delivered,
- Maintain transport documentation and record fuel consumption,
- Comply with vehicle axle load and dimensions regulations,
- Follow basic procedures for breakdowns and accidents,
- Carry out a Drivers Daily Walk-round Check,

For PSV drivers with licences a refresher course programme has been designed and is intended to take 2 weeks. It proposes that theory training should take 27 hours which should be conducted over 5 days of the first week. Practical training should allow a minimum of 1 hour driving per student followed by a driving test of not less than 20 minutes. For new PSV Drivers the time for practical training is increased as appropriate for each class of PSV licence from 4 passengers (i.e. Taxi) up to 31 passengers and over (i.e. full size passenger bus)

The theory test by NIT will have duration of 3 hours and will consist of 100 questions, of which:

- 30 questions are on Road Signs, Signals and Markings,
- 20 questions are on the Road Traffic Act,
- 20 questions are on Defensive Driving,
- 15 questions are on Basic Mechanical Principles,
- 15 questions are on the remaining theory modules.

The pass Criteria is 70% (minimum) required overall, and minimum of 70% required in the Road Signs, Signals and Markings section of the examination.

The final practical driving test will consist of a minimum of 20 minutes driving during which driving faults will be noted. These faults will fall into two categories:

- Minor Faults
- Serious Faults

The minimum pass Criteria is:

- No Serious Fault committed AND
- No more than 8 minor faults committed overall
- No more than 2 minor faults of the same type committed

The PSV programme by Transaid/NIT is divided into 14 modules of which Modules PSV T01 to PSV T10 are classroom based Theory (T) modules, and modules PSV P01 to PSV P04 are vehicle based Practical (P) modules. Students for each class of PSV licence (C, C1, C2 or C3) study the same curriculum; however, the practical modules will utilise a PSV of the appropriate class. 2 below shows the module topics for the Transaid/NIT programme.

Table 2: Module proposed in the Driver Training course for PSV drivers

MODULE	TOPIC
PSV T01	Philosophy of Driving
PSV T02	Road Signs, Signals & Markings
PSV T03	The Road Traffic Act
PSV T04	Defensive Driving
PSV T05	Basic Mechanical Principles
PSV T06	Fitness to Drive
PSV T07	Transport Documentation & Fuel Consumption
PSV T08	Vehicle Axle Load & Dimension Control
PSV T09	Managing Breakdowns & Accidents • Part A – Accidents & Breakdowns • Part B – First Aid
PSV T10	Customer Care
PSV P01	Driver's Daily Walk Round Check
PSV P02	Start, Move Off, Stop & Park the Vehicle Safely • Part A - Starting the engine • Part B - Moving off • Part C - Stopping and Parking
PSV P03	Drive the Vehicle Safely and Responsibly • Part A - Monitor and respond to information from instrumentation, driving aids and the environment • Part B - Operate the Major Controls effectively

PSV P04	Negotiate the Road correctly • Part A - Maintain an appropriate position on the road • Part B - Negotiate bends safely • Part C - Negotiate all types of junctions, including roundabouts, and all types of crossings • Part F - Cooperate with other road users • Part G - Identify and respond to hazards • Part H - Drive Defensively • Part I - Follow the principles of ecologically responsible driving (Eco-safe)

In 2010 The Ministry of Works commissioned Afrisa Consulting Ltd to propose technical requirements for a Driver Testing Center with yard test and other required facilities which will be constructed in several regions in the country. The concept of the design of the Center is based on simplicity, cost effectiveness, and easy to construct anywhere in Tanzania, allowing flexibility for modification to suit particular site-specific conditions. In addition, it is a requirement that the Centre should have separation of direct candidate contact with officials during the processing of license to reduce corruption and other unethical practices. In general, the main purpose of the centre is to improve and harmonize driver testing in the country. The facilities in the proposed Drivers Testing Centres include[31]:

- Offices
- Parking areas
- Canteen
- Yard test with roadways and markings for testing Light motor vehicles, heavy motor vehicles, and motor cycles.
- Vision testing facilities
- Electronic motor cycle testing apparatus which measures speed and allocate points
- Online computer system linked with the Head Office

Traffic legislations in Uganda, Kenya and Tanzania are fairly similar in the registration of motor vehicles, issuance of driving licenses, training of drivers, use of motor vehicles and loading restrictions, use of traffic signs and signals, requirements for insurance, and in the enforcement regimes.

The requirement for driver training prior to being issued a license is another common requirement. Tanzania, Kenya and Uganda also have similar conditions to be fulfilled in order to qualify for a public service vehicle (PSV), e.g., a minimum age requirement and a specified number of years' driving experience.

8.5 Lessons from the Review on Driver Training and Testing Programs

Data on the increasing trend in road accidents have alarmed many countries around the world. Evaluation studies on the effectiveness of the tradition methods of training and testing drivers, and the subsequent methods which replaced traditional methods have both been evaluated in some countries. Among them are the USA, some European and Asian countries, and in South Africa. Among the shortfalls in the training system which have been identified includes:

- Short periods on driver training courses

[31] Afrisa (2011). Standards for Driver Training and Licensing Centres. Project Reference No. 10037/2008/2009. Ministry of Works. Tanzania

- Much of the course duration is spent teaching basic vehicle handling skills;
- Primary motivation of many participants is to learn enough to get a driver's license, and participants are relatively unmotivated regarding safety;
- Training mostly geared towards passing the driving test, rather than teaching the student to become safe and competent drivers –this is for countries where driver training establishments are privately owned;
- Defensive driving training and hazard perception skills, as an accident prevention strategy is not taught adequately;
- Most driving courses train basic vehicle handling skills and traffic codes and do not train new drivers on environmental factors that affect driving, complex perceptual skills to avoid risky driving situations, driver impairments, how to handle emergency situations, personal readiness and self awareness;
- In most driver training courses training instructors do not ensure that learners not only learn the rules, but also understand the reasons behind the rules;
- Good quality training of instructors training is not emphasized;
- Quality control of instructors and driving schools is not adequate.

8.6 Proposal for Harmonization

From the point of view of harmonization of training and testing of drivers in EAC countries, the main objective is to ensure that both elements are conducted uniformly and at high standards throughout the region. The traditional methods for training drivers had been fairly uniform throughout the world in terms of areas of coverage during training, but with notable variations in training durations, theory tests, and in practical test duration and procedures.

In recent years some developed countries have incorporated computer technology in training and testing of drivers which has reduced the training costs involved and improved the quality and consistency of theoretical testing of candidates. With increasing internet connection in EAC cities and towns computer-based theory testing is feasible and should be considered. The advantages of computer-based theory test include:

- Reducing testing cost
- Improving testing quality and consistency
- Reduced unethical practices by examiners

The approaches to the proposed framework for driver training and structure for testing drivers have been developed based on good practices, studies and approaches which have been taken by various researchers, government and non government institutions and organizations around the world. Among them include:

- Driver and Vehicle Licensing Agency (DVLA) – UK
- K53 Licensing System – South Africa
- Transaid: An independent UK NGO working with ITC (Zambia) and NIT (Tanzania)
- BICO –University of Dar es Salaam, Tanzania

8.6.1 Institutional Arrangement

Traffic legislation in EAC countries is fairly similar in driver training and licensing. In all EAC countries there are regulations which define both the legal authorities that controls driver licensing and specify the requirements for potential license holders. Licensing of drivers include criteria on the minimum driving ages by vehicle type and renewal periods; vehicle classifications; medical limitations; driving test procedures; and provision for training. Similarly, every driver is required by law to have a valid driver's license for each specific category of vehicle for which he/she is qualified, while driving without a valid driver's license is a crime in each country.

The requirement for driver training prior to being issued a license is another common requirement. The EAC countries also have similar conditions to be fulfilled in order to qualify for a public service vehicle, e.g., a minimum age requirement and a specified number of years' driving experience. Driving skills need to be regularly evaluated and upgraded through continuous training and education. None of the EAC country makes it compulsory for drivers to undergo regular updating of driving skills and safety methods through training and increased awareness. It is recommended that legislation be introduced to make such training to be compulsory for PSV and HGV during renewal of driving licenses.

We **recommend** that drivers of PSV and HGV should take a compulsory refresher training course before renewing the license. The refresher training to include advancements on vehicle systems, technological changes on vehicle safety, HIV/AIDS and to road safety awareness to the drivers

In all EAC countries, Traffic Police and Vehicle Inspectors function as both driving examiners and vehicle inspectors although these are two different jobs with differing skills requirements. Driver licensing are carried out by tax collection institutions and law enforcement institutions. The mix of powers and functions contributes to an unstable situation whereby the same institution on one hand tests drivers before they are issued with driving licenses, and on the other hand monitors the traffic violations made by the drivers. This leads to a situation where the same institution in reality monitors its own deeds, which is contrary to all established principles of separation between executive and regulatory powers and ultimately leads to the same institution losing authority, credibility and lack of accountability and responsibility.

Of late some the EAC countries have proposed the Driver and Vehicle Examination and Licensing Agency which will be involved in promoting good driving standards; and ensure the use of road worthy vehicles on the roads. The draft bills states that when the Agencies becomes operational, the Governments shall separate driver testing and licensing from traffic law enforcement and tax collection institutions in order to institute accountability and to comply to requirements of signed regional protocols on road safety, and gazette the establishment an independent Driver and Vehicle Examination and Licensing Agency. It has been proposed in the Tanzania draft bill on Road Traffic and Safety Act that the functions of the Agency shall be to:

i. establish standards and methods for the training and testing of driving instructors and drivers of motor vehicles and riders of motor cycles;
ii. establish standards and methods for the training and testing of vehicle examiners;
iii. provide syllabi for driver training and the training of instructors;
iv. issue driving licences;
v. register and license driving schools;
vi. license driving instructors;
vii. inspect, test and register motor vehicles
viii. issue vehicle registration certificates;
ix. issue vehicle examination certificates;
x. license and regulate private garages to undertake vehicle testing;
xi. maintain registers containing particulars of licensed motor vehicles, driving instructors, driving schools and drivers of motor vehicles;

xii. advise the Minister on policy formulation and development strategy for the achievement of the object of the Agency

xiii. ensure strict compliance with this Act and regulations made under it regarding driver and vehicle examination and licensing;

The new Legislations should, where possible, enable changes in the driving licensing procedures without lengthy delays to cope with technological changes on vehicles, training, and testing of driver candidates.

We **recommend** that the process of transferring the function of driver testing from law enforcement and tax collection institutions to an agency under the responsible ministry of transport be accelerated to facilitate the proposed improvements on the quality of driving schools, instructors, and examiners for driver candidates.

8.6.2 Structure for Drivers Training

The types of drivers training for EAC countries are **recommended** to be categorized as:

- Basic Driver Training for light motor vehicles (LMV)
- Professional Driver Training (PSV and HGV)
- Compulsory Basic Training (motorcycle riders, engine capacity <125cc)

8.6.3 Basic Driver Training for LMV

Driver training and education has been defined as the process of bringing a learner driver to a level of skill and knowledge sufficient for him or her to properly operate a motor vehicle on all roads. Driver training is a structured approach to the learning process that can presumably facilitate and accelerate the needed skill. It has been observed that drivers exposed to this formal instruction have a lower accident rate than those who do not receive such training.

The Basic Driver Training course is for new drivers for light motor vehicles (GVM less than 3.5 tonnes). In the interim period, and in few decades to come there will be a mix of drivers in EAC public roads from different driver training systems of the EAC countries, including those who have never attended driver training. The following minimum requirements for Basic Driver Training are:

- Comply with minimum age limit (18 years)
- Be able to read and write

The Basic Driver Training for LMV will be uniform in all EAC countries and will focus on producing new drivers who are trained according to approved EAC standards. The majority of new drivers will be young, and young drivers are considered to be more at risk of being involved in a car accident than older drivers. This is due to their age and lack of driving experience to handle most road conditions. Keeping young drivers in consideration, the training should also cover physiological and psychological effects of alcohol and drugs, driving attitudes including road rage, and raising the awareness on the effects of environment and lack of experience on accidents.

The Basic Driver Training is designed to produce drivers who have a thorough appreciation of the role they play in keeping the roads safe for themselves and others to use. It focuses on the following five main areas of driver competence:

- Knowledge, driving laws, rules and vehicle operation;
- Control of the vehicle;
- Control in traffic situations;

- Recognising, managing and avoiding risks; and
- Driving in a social context.

We recommend that the syllabus for Basic Driver Training should consist of the following modules:

1. Vehicle Controls and Safety Checks
2. Driving Laws and Rules
3. Pre-Starting Routine
4. Primary Controls for Starting, Moving and Stopping the Vehicle
5. Correct Positioning
6. Changing Direction
7. Progression Management
8. Anticipation and Reaction
9. Sharing the Road
10. Driving Safely Through Traffic
11. Changing Direction in More Complex Situations
12. Speed Management
13. Self-Control and Driving Calmly
14. Night Driving

The proposed syllabus for basic Driver Training is attached in Appendix B.9.1.

8.6.4 Professional Driver Training for PSV and HGV

The proposed driver training for professional drivers is designed for:

- Passenger Service Vehicle (PSV); and
- Heavy Goods Vehicles (HGV) which have GVM more than 3.5 tonnes.

PSV and HGV are mostly driven by drivers whose job is to drive.

We recommend the following minimum requirements for the eligibility to attend the Professional Drivers Training:

- Be able to read and write
- Holder of valid license of at least light motor vehicles (for less than 3.5 tonnes)
- Minimum driving experience of 3 years
- Minimum age requirement for driving PSV and HGV vehicles (21 years)
- Must have attended the *Basic Driver Training* course within the past 5 years

PSV and HVG candidates are assumed to have Basic Driver Training knowledge, the training is supposed not to take much of the training time on teaching road signs, traffic codes, and basic skills. Hence, in addition to the Basic Driver Training knowledge, the Professional Driver Training for PSV and HGV are formulated to train additional knowledge and skills, among other, on:

- Manoeuvring of heavy vehicles
- Driver responsibilities and Road traffic act
- Customer care (PSV)
- Eco-driving
- Procedure for breakdown

- Vehicle loading and overload control regulations
- Vehicle dimension regulations

The programme is divided into 18 modules of various durations. Modules HGV T01 to HGV T11 are classroom based Theory modules. Modules HGV P01 to HGV P07 are vehicle based Practical modules. Trainees for each class of HGV licence (C, C1, C2 or C3) study the same curriculum; however, the practical modules will utilise a HGV of the appropriate class.

A full list of the Theory and Practical modules (including, where appropriate, their component parts) can be found at the Appendix

We recommend that the Syllabus for Professional Driver Training should consist of the following modules:

Module T01:	Philosophy of Driving
Module T02:	Road Signs, Signals & Markings
Module T03:	The Road Traffic Act
Module T04:	Defensive Driving
Module T05:	Basic Mechanical Principles
Module T06:	Fitness to Drive
Module T07:	Transport Documentation & Fuel Consumption
Module T08:	Vehicle Axle Load & Dimension Control
Module T09:	Managing Breakdowns & Accidents
Module T10	Practical Basic First Aid
Module T11:	Customer Care
Module P01:	Driver's Daily Walk Round Check
Module P02:	Start, Move Off, Stop & Park the Vehicle Safely
Module P03:	Coupling & Uncoupling the Tractor Unit and Trailer
Module P04	Reversing with and without the Trailer
Module P05	Drive the Vehicle Safely and Responsibly
Module P06:	Negotiate the Road correctly

The proposed syllabus for PSV and HGV has to a large extent been adopted from the Draft PSV and HGV Driving Curriculum proposed by NIT[32]. The proposed syllabus is attached in Appendix B.9.2.

8.6.5 Compulsory Basic Training for Motorcycle Rider (Engine Capacity < 125 cc)

Motorcycles riders must attend a Compulsory Basic Training (CBT) for light motorcycles (engine capacity less than 125cc) before riding in public roads.

We **recommend the** minimum requirements to attend the CBT as:

- Ability to read and write
- Compliance with minimum age requirement (16 years)

The proposed Compulsory Basic Training for light motorcycle riders has been largely adopted from South Africa K53 driver licensing system[33], and the European Motorcycle Test[34].

[32] National Institute of Transportation. (2011). PSV Driving Curriculum (Draft)
[33] www.foresightpublications.co.za
[34] The new European Motorcycle Test. (2010)

We **recommend** the following modules to be used for training:

1. Legal requirements
2. Safety, equipment, and essential clothing and protection
3. Pre-trip inspection - around the motorcycle
4. Mount - Dismount
5. Pre-trip inspection - on motorcycle
6. Starting and switching off procedures
7. Mirrors - use of
8. Signalling
9. Signalling - hand signal (turning left), hand signal (turning right), hand signal (stop or sudden reduction of speed)
10. Signalling – horn
11. Clutch - use of
12. Moving off
13. Steering
14. Speed management
15. Incline start
16. Turning speed judgment
17. Emergency stop
18. Emergency swerve
19. Speed control
20. Gear changing – up
21. Gear changing – down
22. Following other vehicles
23. Lane changing
24. Stopping - in traffic
25. Stopping - for parking
26. Traffic control signals
27. Intersections - turning left, turning right, proceeding straight, stop signs, yield signs, uncontrolled, traffic lights (flashing red), traffic lights (steady red), traffic lights (green), traffic lights (flashing amber), traffic lights (steady amber), traffic circles
28. Block pedestrian crossing – uncontrolled
29. Level crossing – guarded
30. Level crossing – unguarded
31. Overtaking - to the left of a hazard
32. Overtaking - to the right of a hazard
33. Being overtaken - on the left-hand side
34. Being overtaken - on the right-hand side
35. Freeways – entering
36. Freeways - passing off-and on ramps
37. Freeways - exiting

We further **recommend** that the rider should possess the license for light motorcycles for at least 2 years before he/she can be allowed to ride motorcycle with engine capacity bigger than 125 cc.

8.6.6 Structure for Driver Testing

The proposed tests are designed to examine the driver/rider:

- How he/she inspects and checks the vehicle before starting the engine.
- How he/she controls the vehicle, including smooth operation and correct use of the gears, clutch and brakes.
- How he/she comply with the rules of the road and road signs, signals and markings.
- How he/she perceives upcoming hazards, and driver's awareness and his/her reactions
- How he/she applies defensive driving skills including observations on changing conditions on the road.
- Timely use of signals, brakes and gear changing.
- How to demonstrate courtesy with other road users.
- How he/she ensures good customer care for the passengers (PSV)

We **recommend** that testing for both licenses should include:

- Theory test
- Pre-trip inspection
- Practical test: Yard test and on-road test

The candidate should pass all parts of the above tests to get a license

Theory Test

<u>LMV Candidates</u>

The theory test for LMV driver candidates should be of written type and should contain at least 50 questions. The maximum time to conduct the test should be 50 minutes.

We **recommend** that questions on the theory test should be of **multiple choices**, and the maximum time to conduct the test should be **one hour** and the test should be based on:

- Road signs, signals and markings,
- Defensive driving
- Basic mechanical questions

<u>PSV and HGV Candidates</u>

The theory test for Professional Driver Testing should be of written type and should contain at least 100 questions. The maximum time to conduct the test should be 2 hours.

We **recommend** that the theory test for Professional Driver Testing to be of **written type** and should contain **at least 100 questions of multiple choice type and short questions**. The maximum time to conduct the test should be **2 hours**. Questions on the theory questions should be based on:

- Road signs, signals and markings,
- Defensive driving
- Basic mechanical questions
- Eco-driving
- Vehicle loading and dimensions control
- Vehicle breakdown and emergency

The candidate must pass the theory test before he/she can take the practical test

Motorcycle Rider Candidates (capacity < 125 cc)

The theory test for motorcycle rider candidates should be of written type and should contain at least 50 questions of multiple-choice type. The maximum time to conduct the test should be 50 minutes.

We **recommend** that questions on the theory test should be based on:

- Road signs, signals and markings,
- Defensive driving
- Basic mechanical questions
- Safety and balance

Pre-trip Inspections

Pre-trip Inspections for Light Vehicles, PSV and HGV

The objective of pre-trip inspection is to examine the vehicle for roadworthiness. In this test the examiner will inspect the vehicle for roadworthiness and that the vehicle is fit for the yard test and on-road test. The examiner shall request the applicant to enter the vehicle and operate the lights, direction indicators, wipers, horn, etc. The examiner will also inspect the exterior of the vehicle to inspect tyres and other parts of the vehicle for roadworthiness and security. The examiner should discontinue the test if the vehicle is found to be not roadworthy.

Pre-trip Inspection for Compulsory Basic Training (motorcycles)

The examiner shall instruct the applicant to mount the motorcycle and to start the engine. The examiner shall instruct the applicant to operate the lights, direction indicators and horn. Should any of the items be inoperative, the motorcycle will be considered not roadworthy and the examiner shall discontinue the test.

Practical Test

Practical Test for LMV driver Candidates

The purpose of practical tests is to determine the ability of driver to drive light motor vehicle (not exceeding 3,500kg). The test consists of two separate parts, jointly measuring the proficiency of a driver regarding the handling of the vehicle, obedience of the traffic rules, and coping with traffic problems in the practical driving situation.

a) Yard Test

The yard test should be administered on pre-established open space (yard) which is not used by normal traffic. During the test, the examiner marks the incorrect responses in the appropriate category. The examiner should not pose any questions to the applicant's theoretical knowledge during the practical test. Neither should the examiner comment on, mention or discuss any error made by the applicant during the test before handing the applicant a copy of the Test Report on completion of the test. In the yard test the instructions to carry out a manoeuvre should be only given whilst the vehicle is stationary.

We **recommend** that the maneuvers in the yard test should include at least the following:

- Left turn in the road.
- Alley docking.
- Reverse in a straight line
- Turn in the road
- Parallel parking.
- Incline start.

(i) *Left turn in the road*

The examiner should inform the applicant to steer the vehicle round the bend without stopping, touching the kerb or touching any lines.

(ii) *Alley docking*

Stop to the right of the boundary. Reverse to the right into the demarcated area without touching any obstacle or crossing any barrier line and stop where indicated. When leaving, the examiner should request the applicant to steer out of the demarcated area without touching any obstacles and stop where indicated.

(iii) *Reverse in a straight line*

The examiner should inform the applicant to reverse the vehicle in the demarcated area without stopping until told to do so. The applicant should not touch side boundary lines.

(iv) *Turn in the road (3-point turn)*

The examiner should inform the applicant to stop where indicated and in three movements turn the vehicle around to face in the opposite direction by using two forward and one reverse movements without touching any kerb and end ones turn on the left hand side of the side road, since it would be a violation to stop with on the right hand side.

(v) *Parallel parking*

The examiner should inform the applicant to stop in the demarcated area, and inform that only three movements are permitted: That is reverse movement into the parking bay and two further movements to position the vehicle within the parking bay without touching any obstacle or kerb. The examiner should tell the applicant to repeat the manoeuvre from the opposite side.

(vi) *Incline Start*

The examiner should inform the applicant to stop where indicated without rolling back, and then move off without rolling back.

(b) Road Test

The road test should be conducted for at most 45 minutes on a pre-established route.

We **recommend** that the route should have the following features:

- A tarred multi-lane road containing at least two controlled intersections and clearly demarcated road markings.
- At least one intersection controlled by a four-way stop sign
- At least four intersection controlled by stop signs or robots
- At least two intersections controlled by yield signs, one with the right of the way at least one of the intersection and the other without the right of the way.
- A quiet street where the emergency stops can be executed

- At least two intersections with two way traffic should be crossed with a right turn.

The examiner must inform the applicant well in advance in a clear, concise and audible manner the following;

- To fasten the seatbelt
- Signalling and observing will be required on a public road
- All traffic signs, markings and rules must be obeyed.
- No uncontrolled or dangerous actions may be made
- Giving, as far as possible, one instruction at a time, and well in advance
- Not to give any instruction to carry out any illegal action
- Should the test be terminated at any stage, the full test will have to be repeated
- Hand signals must be executed in a quiet street during the road test.

Practical Test for HGV and PSV Driver Candidates

The practical test for professional driver is similar to that of light motor vehicles. However, parallel parking and 3-point turn should not be tested for HGV and PSV as they are not practical in normal public roads. The sequences for the remaining manoeuvres are similar to those of light motor vehicles.

We recommend that the yard test for HGV and PSV driver candidates should only contain:

- Left turn in the road.
- Alley docking.
- Reverse in a straight line
- Incline start.

Practical Test for Motorcycle Rider Candidates (capacity < 125 cc)

The practical test consists of a yard test, which includes a starting procedure and a driving skill test. The practical test is a measure of the motorcycle rider's ability in respect of the handling of a motorcycle, obedience to traffic rules and the use of motorcycle control.

The test shall be administered on a prescribed track that has no kerbing and which is closed to other traffic and persons, and it should be tarred, paved or other surface which is reasonably level and skid resistant. The test shall commence after the pre-trip inspections has been completed.

We **recommend that** the following maneuvers should be conducted for candidates of motorcycle riders:

- Starting procedure
- Speed management
- Moving off/Turns (left)
- Lane change (right)
- Incline start
- Turning speed judgment (left and right)
- Emergency stops (stop 1 and stop 2)
- Emergency swerve (left and right)

Assessment of Candidates

Theory Test

We **recommend** that the minimum pass marks for theory test for applicants of licenses to drive LMV should be 50%, whereas those for PSV, HGV and motorcycles should be 70%.

Practical Tests

We **recommend** that the assessment of practical driving tests (yard and road tests) should be based on the competency of the candidate to drive/ride independently and the number of faults the candidate makes during the test. Faults should be in two categories:

- Minor Faults
- Serious Faults

We further **recommend** that any single serious fault should lead to a failure in practical test by the candidate, and if the candidate makes more than five minor faults or if repeating the same fault at least 3 times he/she should fail the practical test.

NB: A serious fault is one that immediately endangers the safety of the test vehicle, its passengers or other road users.

In general the candidate will not pass the practical test if he/she does not have the competencies to drive independently, and in a way that is;

- safe
- cooperative
- leads to good traffic flow
- considers health, environment and other people's needs
- corresponds to existing laws and regulations in force when assessed as a whole

We **recommended** in each category of license that the assessment of practical test should be conducted in the same format using standard forms to mark the faults made by the candidates.

The results of practical test should be computerized to enable ease assessment of examiners and areas which need improvement in driver training syllabi.

8.6.7 Quality of Driving Instructors

Driving instructors should have the ability to impart practical and theoretical knowledge to their students, as well as the obvious skills in driving the vehicles and giving demonstrations of techniques where appropriate, they should be conversant and comfortable with classroom procedures and be good communicators. It has been observed that a competent driver does not necessarily make a good driving instructor, but a competent driving instructor must always be a good driver.

We recommend that all driving instructors should be trained and qualified as driving instructors by a recognized government authority. Furthermore there should be a central register of instructors for efficient monitoring which shall ensures that the public can be assured that an acceptable standard of training is offered by all instructors and driving schools. It should be illegal for anyone to conduct driver training unless their name is on the register, and there should be means to ensure that only approved instructors conducts training to the approved standards.

In order to receive a qualification, the driver training instructor must attend and successfully complete training of trainer's course organized by the appropriate government authorities. Potential driving instructors should be required to pass a qualifying examination, and the instructor has to qualify to drive vehicles of the type in which they will train candidates.

We **recommend** that all driving instructors should be trained and qualified as driving instructor and should be centrally registered by a recognized government authority

8.6.8 Quality of Driving Schools

The government authority should set standards for driving schools and all qualified schools should also be centrally registered. This will encourage driving schools to meet criteria, such as to employ registered instructors, adequate numbers of training vehicles, and rooms for theory training. This will ensure better quality control as instructors and schools can be taken off the register or disciplined for any corruption or other deficiencies. Candidates for driver testing should be checked if they have attended registered schools to be given an appointment for testing.

We **recommend** that the government authority should set standards for driving schools and all qualified schools should also be centrally registered

A privatized training system where driving schools operate in a free market environment has been observed to work well in most countries around the world.

We therefore **recommend** that driving schools should be privately owned but closely monitored by the government to ensure that the standards of training are maintained and the school use approved driver training manuals.

8.6.9 Quality of Driving Examiners

Driving examiners should be properly trained and should have adequate time to conduct the test without rushing. Hence adequate number of examiners should be employed to cope with the volume of driver license applicants. All examiners should be subject to the same training and should be qualified to drive all categories of vehicles that they are entitled to examine.

We **recommend** that Driving examiners should be properly trained and all examiners to be subjected to the same training.

Driving examiners should be subject to frequent supervision to ensure that their individual assessment is checked and that the tests are being conducted in accordance with the regulations. Test results of every candidate should be computerized to make assessment of examiner's easier and to notice any individual variation from the average results obtained by other examiners.

It is essential that driving examiners be seen to be fair and impartial and that the test sets a uniform standard wherever it is applied. Examiners should learn to give clear directions and instructions, and should put the candidate at ease. Examiners should assess and mark driving faults as they occur.

We **recommend** that driving examiners be subject to frequent supervision and assessment to ensure that the tests are conducted in accordance with the regulations.

8.7 Further Recommendations

- To consider establishment of Driver Testing Centers which will have full facilities for conducting yard and theory tests
- To look into technical feasibility of establishing **computer-based theory test** to reduce bias, corruption, and testing costs.
- The **driver training syllabus should be revised periodically** to accommodate advancement in technology on vehicle safety features and other vehicle systems
- **Automatic transmission (AT) vehicles** should be allowed for training and testing of LMV driver candidates. This is due to increasing trend in automatic transmission vehicles in the world and to follow the good practice which has been shown by many countries in the world. However, the driver candidates who will pass the test with AT vehicle should be given ***restricted license*** which will not allow him/her to drive manual transmission vehicle until he/she re-take and pass the practical test to change his/her license status to *full license.*

8.8 Impact of Harmonisation of Driver Training and Testing in EAC Member States

Improved harmonized driver training and testing in the EAC will result in improved safety and efficiency across the member states road networks. This is a long term impact that will results from consistent actions by the member states. The actions include implementing the recommendations of this report. Some of the recommendations are pre-requisites for kicking-off the harmonization process in driver training and testing in EAC member states. In that respect, the following need immediate action and have to be implemented as soon as possible:

Transfer the function of driver testing and licensing from law enforcement and tax collection institutions to an agency under the Ministry responsible for Transport.

Development of Training Curriculum for Driver Trainers: A course on training of trainer's should be designed to enable training of instructors. The course should be hosted in an accredited institute and qualified trainers should be awarded certificates as Qualified Driving Instructors. The curriculum for the course should be common throughout the EAC member states

Development of Training Curriculum for Driving Examiners: A course on training of driver examiners should be designed to enable training of instructors. The course should be hosted in an accredited institute and qualified examiners should be awarded certificates as Qualified Driver Examiners. The curriculum for the course should be common throughout the EAC member states

Training of Driver Trainers: All driving instructors should be trained and qualified as driving instructor and should be centrally registered by a recognized government authority.

Training of Driving Examiners: All driving examiners should be properly trained and all examiners to be subjected to the same training.

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Appendix 1: Draft Syllabus for Basic Driver Training

Modules	Objective(S)	To Be Learned
Module 1: **Vehicle Controls and Safety Checks**	To learn how to carry out appropriate pre-start vehicle checks, including checking the roadworthiness of the vehicle; and the vehicle's primary and secondary controls and know how to use them correctly.	• The use of primary and secondary controls; • Vehicle safety; and • Vehicle roadworthiness.
Module 2: **Driving Laws and Rules**	To learn what learner drivers need to know before they begin to drive. This includes: • Traffic act (driving laws and rules) • Road signs and markings	Knowledge on: • the governing laws on regulations and responsibilities of a driver • Administrative aspects of driving • traffic signs and markings
Module 3: **Pre-Starting Routine**	To learn on the necessary procedure before starting the engine	Skill and knowledge on: • correct adjustments to the seat. • use of seat belts • adjusting and use of head restraint • adjust the mirrors, both interior and exterior. • securing doors correctly. • use of the handbrake or parking brake • to ensure that the vehicle is not in gear before starting the vehicle. • to ensure that the steering wheel is correctly adjusted before starting the vehicle. • to ensure that there is sufficient fuel for the journey.
Module 4: **Primary Controls for Starting, Moving and Stopping the Vehicle**	• To learn the knowledge, skills and understanding to operate the primary controls effectively, accurately and without prompting. • to use the primary controls in a timely fashion when it is appropriate	To know the correct procedures to: • to operate the foot brake appropriately with correct pressure • to operate the handbrake or parking brake appropriately • to operate the accelerator appropriately to control the vehicle's progress and engine speed. • to operate the clutch appropriately to ensure smoothness when moving off, stopping and changing gear. • to operate the gears appropriately with proper selection for vehicle and engine speed. • to operate the indicators appropriately to give timely communication to show an intention to change direction. • to steer appropriately and smoothly when changing direction; and, • to steer quickly when necessary to

Modules	Objective(S)	To Be Learned
		take evasive action.
Module 5: **Correct Positioning**	To learn how to correctly position the vehicle on the road for the action about to taken. Actions include: positioning on the straight, cornering, negotiating bends, negotiating junctions, changing lanes, entering and exiting roads, entering and exiting junctions, roundabouts, reversing, turning, parking, stopping, and while progressing through traffic and within traffic lanes.	Skills for driving in traffic in a variety of road conditions and to position vehicle: • on the straight • in traffic lanes • on bends turning left and right • turning left and right • negotiating roundabouts • approaching and emerging from junctions
Module 6: **Changing Direction**	To learn how to use mirrors appropriately; and carry out the mirror-signal-mirror-maneuver (MSMM) routine correctly.	Skill for MSMM routine. when: • turning left and right; • turning at roundabouts; and • when negotiating junctions
Module 7: **Progression Management**	To learn how to regulate and maintain good control over the speed of vehicle in all conditions	Skills for speed management on public roads including: • controlling speed limits; • stopping distances; • effects of road and weather conditions; • driving too fast.
Module 8: **Anticipation and Reaction**	To learn how to scan the environment for potential hazards; and identify and respond appropriately to potential hazards.	Skills for driving in a variety of road conditions including: • junctions • streets with parked cars • streets with substantial numbers of pedestrians • roads with multiple warning signs • roads with poor visibility such as bends with tree cover or sign boards.
Module 9: **Sharing the Road**	To learn how to drive with due care and attention for the road and traffic conditions. This includes taking appropriate actions when identifying hazards.	Skills to: • enter, cross and join roads safely. • park safely on roads where there are other road users. • pass stationary and slow moving vehicles, cyclists and other road users safely and responsibly. • know when and how to give way to other road users including pedestrians. • know how to deal with emergency vehicles.
Module 10: **Driving Safely Through Traffic**	To learn how to drive with due care and attention where there is a significant volume of other traffic.	Skills for: • defensive driving; • recognizing risks to the safety of others; and

Modules	Objective(S)	To Be Learned
		• co-operating with other road users.
Module 11: **Changing Direction in More Complex Situations**	To learn how to: • carry out observational and scanning tasks when turning left and right; • negotiate junctions; and • change direction in more complex and challenging driving situations.	Skills to drive in traffic in a variety of busy road conditions, and have significant interaction with other road users while making independent decisions about which route or position to take.
Module 12: **Speed Management**	To learn how to regulate and maintain good control over the speed of the vehicle in more complex or challenging driving situations.	Skills on how to drive in fast-moving traffic or where speeds and traffic are highly variable.
Module 13: **Self-Control and Driving Calmly**	To learn how to remain in calm control of the vehicle in situations where the driver may be: • frustrated; or • feel threatened.	Skills for self control when experiencing driving situations such as: • joining busy main roads; • stalling and re-starting of the vehicle; and • being tail-gated with a rough driver • parking on a road with significant numbers of other parked cars.
Module 14: **Night Driving**	To learn how to: • use vehicle lighting correctly; • respond correctly to the lights of other road users; • maintain full control of the vehicle while driving through traffic in the dark or in poor lighting conditions.	Skills for: • driving safely in the dark; • drive safely where road lighting is poor; • using vehicle’s lighting correctly; and • appropriate response to other vehicle lighting and emergency vehicles

Appendix 2: Syllabus for Professional Driver Training

Modules	Objectives	To be Learned
Module T01: **Philosophy of Driving**	To understand: • why driving is a profession • the driving environments • drivers responsibilities • principles of good driving	• definition of driving • who is a good driver, and • principles of good driving, • driving environments • driver responsibilities.
Module T02: **Road Signs, Signals & Markings**	To understand: • the purpose of road signs, signals and markings • the consequences of ignoring or disobeying the signs, signals or markings • all road signs, signals and markings	• the purpose and requirements of road signs, signals and markings, • why a driver should comply with all signs, signals or markings
Module T03: **The Road Traffic Act**	To understand: • the purposes of the road Traffic Act • understand all sections of the Road Traffic Act • understand the possible consequences of committing a Road Traffic Offense	• the purposes of the road traffic act, • the requirements of registering motor vehicles, • driving licence requirements, • the appropriate use of a motor vehicle • how traffic regulations are enforced
Module T04: **Defensive Driving**	To understand: • the possible causes of accidents • the Accident Prevention Formula • the need to maintain a safe minimum following distance • understand the effects of adverse conditions • understand the appropriate driver behavior and attitude	• what causes an accident, • the standard accident prevention formula, • the elements of defensive driving, • the need to maintain a safe minimum following distance, • the varying types of accident and their prevention, • how to avoid an intersection collision, • the effects of various adverse conditions on driving, • the correct driver behavior and attitude
Module T05: **Basic Mechanical**	To understand: • the working principles of the internal combustion engine	• the consequences of driving a defective vehicle, • the working principle of an internal combustion engine,

Modules	Objectives	To be Learned
Principles	• the purposes of the various system on the vehicle • the need for proper vehicle maintenance	• the vehicle system for brake, transmission, lighting, steering, fuel • the functions of tyres and wheels • the need for proper vehicle maintenance.
Module T06: **Fitness to Drive**	• To understand the consequences of not being physically or psychologically fit to drive.	• the negative impact of drugs, alcohol and fatique on the ability to drive safely and responsibly, • how emotional states can have a negative impact on the ability to drive safely and responsibly
Module T07: **Transport Documentation & Fuel Consumption**	• To identify all documentation that should be carried on the vehicle • To record the appropriate information on the various documents • To ensure the documents are current and valid, • To understand how and for what purpose fuel consumption is monitored.	• the standard documents to be carried on the vehicle • the purpose, use and validity of the documents, • the monitoring of fuel consumption, • factors contributing to excessive fuel consumption.
Module T08: **Vehicle Axle Load & Dimension Control**	To understand: • why vehicle weight and dimensions are controlled • the maximum weight and dimensions of a particular vehicle • how to calculate the approximate weight of passengers and their baggage	• the reason for the control of axle loads, GVM and vehicle dimension, • the present axle load legislation and maximum dimension limits, • the different vehicle axle configurations,
Module T09: **Managing Breakdowns & Accidents** • Part A – Accidents & Breakdowns • Part B – First Aid	To understand: • Procedures to follow when involved in an accident • What to do in the event of breakdown	• what to do in the event of an accident • how to describe the event with the police • what details to record at the scene of the accident, • not to get involved in any kind of argument with anyone • when to report an accident to the police, • what to do in the event of a breakdown, • how to tow or be towed safely.

Modules	Objectives	To be Learned
		• the consequences of a broken down vehicle remaining on the road
Module T10: **Practical Basic First Aid**	To understand: • the roles and responsibilities of a first aid provider • the requirement for carrying a First Aid kit and what it should contain • basics for life support	• the roles and responsibilities of a first aid provider, • the barriers to action, • scene size-up and safety: (initial assessment of the casualty, • body substance isolation, • emergency moves and back safety: (when alone, with one helper, with two or more helpers, using a stretcher). • why a First Aid kit should be carried, • what a First Aid kit should contain.
Module T11: **Customer Care (for PSV)**	• To understand good customer care • understand the importance of driver in good customer care, • understand how to create a good first impression.	• what is customer care/service, • the importance of driver to customers and to the business success • managing and exceeding customer expectations, • how to create a good first impression.
Module :P01 **Driver's Daily Walk Round Check**	• To identify all the components of the vehicle that must be checked • To check the vehicle for defects • To record correctly the defects found	• accurately identify defects, recording correctly and reporting to the relevant person, • understand why checks should be carried out by a competent person • How to check all fluid levels, the correct air pressure, the horn, all lights, and reflectors are clean, electrical equipment is in good working order, • check there any damage that would impair ability to drive the vehicle safely, • check all wheel nuts, all controls mirrors • make adjustments to ensure a safe and comfortable driving position which enables driver to maintain good all-round visibility and control of the vehicle and minimizes fatigue, • check if there is sufficient fuel • check fire extinguisher for security, accessibility and

Modules	Objectives	To be Learned
		operational status, • check First Aid kit for contents and accessibility, • check that the emergency exits and their operation are clearly identified,
Module P02: **Start, Move Off, Stop & Park the Vehicle Safely** • Part A - Starting the engine • Part B - Moving off • Part C - Stopping and Parking	• To understand the appropriate way of starting the vehicle safely • To understand the appropriate way to move off safely and to stop and park the vehicle safely and responsibly	• How to make sure the vehicle transmission is disengaged, • How to monitor instrumentation and gauges throughout engine start up and respond correctly to the information provided, • How to start the engine using the specified method, • How to select the appropriate gear in preparation for moving off, • How to carry out all-round visual checks, including blind spots, to ensure that it is safe to move-off, • How to signal for intention to move off to other road users; • How to move off straight-ahead, on the level and on gradients, safely and smoothly, maintaining control of the vehicle at all times, • How to move off around a parked vehicle or obstruction, safely and smoothly maintaining control of the vehicle at all times, • How to recover quickly and safely if the vehicle stalls, • How to select a safe, legal and convenient place to stop and park, • How to use the accelerator and brakes appropriately to regulate speed and bring the vehicle to a stop safely, • How to ensure the vehicle is secure when parked or stopped, • How to ensure that neutral is selected when stopped or parked, • how to select a gear (vehicle dependent) and position the steering wheels to hold the vehicle safely when parked on a gradient, • How to switch the engine off
Module P03: **Coupling and Uncoupling**	To Understand all procedures on how to position the tractor unit	• how to reverse the tractor unit into position in front of the trailer

Modules	Objectives	To be Learned
of Trailers: • **Part A: Coupling the trailer** • **Part B: Uncoupling the Trailer**	for coupling and uncoupling the trailer.	in preparation for coupling the combination, • brakes application during coupling and uncoupling, • heights adjustments • how to check and open the fifth wheel jaws, • how to test if the kingpin locking mechanism is secure, • what the dog clip is and its purpose, • the various types of couplings and the need to ensure the tractor unit susies are compatible with the trailer couplings, • how to operating the landing gear mechanism safely and correctly, a. why the handle must be stowed safely b. how to check the roadworthiness of the vehicle combination by carrying out a driver's walk-round check,
Module P04: **Reversing With and Without Trailer**	To understand all procedures for reversing the vehicle with and without trailer	•
Module P05: **Drive the Vehicle Safely and Responsibly** • Part A - Monitor and respond to information from instrumentation, driving aids and the environment • Part B - Operate the Major Controls effectively	To understand: • How to react appropriately to information from instruments, driving aids and the environment • to operate the major controls of the vehicle effectively • to maintain control of the vehicle while making effective use of mirrors and other aids • how to monitor other	• How to monitor and respond appropriately to gauges, warning lights and other aids when driving, • How to make effective use of mirrors and other aids to monitor other road users and hazards, • How to monitor and respond to changes in the environment, • How to judge speed and distance correctly and effectively, • How to signal intentions correctly to other road users within a safe, systematic routine, • How to operate the lights, indicators, horn, windows, wipers, demisters, ventilation controls

Modules	Objectives	To be Learned
	road users and hazards	• How to use the accelerator smoothly, apply the brakes smoothly and progressively to control the vehicle • How to change gear smoothly and in good time, • how to select the appropriate gear for the road speed of the vehicle, • How to coordinate the use of gears with braking and acceleration, • How to steer the vehicle safely and responsibly in all road and traffic conditions, • How to continue to steer the vehicle while operating other controls, • How to coordinate the operation of all controls to maneuver the vehicle safely and responsibly in all road and weather conditions in forward and reverse gear, • How to continue to make effective observations, including checks of blind spots, while undertaking a maneuver,
Module P06: **Negotiate the Road correctly** • Part A - Maintain an appropriate position on the road • Part B - Negotiate bends safely • Part C - Negotiate all types of junctions, including roundabouts, and all types of crossings • Part F - Cooperate with other road users • Part G - Identify and respond to hazards • Part H - Drive Defensively • Part I - Follow the principles of ecologically responsible driving (Eco-	To understand: • To maintain an appropriate position on the road • To negotiate bends safely • To negotiate all types of junctions, including roundabouts, and all types of crossings safely • To comply with signals, signs and road markings • To communicate intentions to other road users • To cooperate with other road users • To identify and respond to hazards • To drive defensively • To follow the principles of	To learn how to: • select and maintain an appropriate position on the road, • change lanes safely and responsibly, • overtake other road users legally, safely and responsibly • assess bends correctly on approach, • select a safe position to enter the bend at an appropriate speed, • maintain safe speed and control throughout a bend • apply an appropriate safe and systematic procedure to negotiate all types of junctions and roundabouts • turn left/right • emerge safely into streams of traffic, • cross the path of traffic safely when turning right, • respond appropriately to all permanent and temporary traffic signals, signs and road markings, • respond appropriately to signals

Modules	Objectives	To be Learned
safe)	ecologically responsible driving	given by authorized persons, • respond appropriately to signals given by other road users, • drive at an appropriate speed, • use indicators and arm signals to signal intentions in a timely fashion and where appropriate, • reinforce the use of any signals given by positioning the vehicle correctly and safely, • use horn and lights as a means of communication to other road users where appropriate, • use hazard lights correctly and safely. • be aware of and anticipate the likely actions of other road users, • give other road users sufficient time to perform maneuvers, • monitor and manage own reaction to other road users, • respond to emergency vehicles and official motorcades appropriately, • progress in the traffic stream and overtake with due consideration for other road users. • use appropriate techniques to continually scan the driving space close to the vehicle and into the distance, • use visual clues and precursors to predict possible hazards and anticipate situations which may arise, • judge the significance of potential hazards and prioritize responses, • maintain concentration when faced with distractions. • scan and check surroundings with particular reference to blind spots, • position vehicle to maximize visibility to other road uses, • drive at such a speed that you can always stop safely in the distance you can see to be clear. • accelerate and decelerate smoothly and progressively, • anticipate the need to stop, using timely and smooth deceleration, thereby reducing fuel consumption and general vehicle

Modules	Objectives	To be Learned
		wear and tear, • drive in the highest responsive gear while maintaining full control and avoiding labouring the engine, • remove excess load from the vehicle when not required, • ensure tyres are correctly inflated,

FINAL REPORT

ANNEX A.9

HARMONISATION OF VEHICLE DIMENSIONS AND COMBINATIONS

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EXECUTIVE SUMMARY

Standards and regulations on vehicle dimensions and combinations are essential from the safety aspects of vehicle and of other road users. It is important to ensure that characteristics of all vehicles using public roads are compatible with the geometric design of the road network. Non-regulated standards within a country or between countries which have cross-border freight transport are a potential hazard for safety of the vehicles, road infrastructure, and all road users.

COMESA, EAC and SADC have started the process of harmonization of vehicle dimensions and have agreed on maximum vehicle dimensions, in terms of height, width and length of vehicles. In principle the tripartite partner states have agreed to these vehicle dimensions limits at the regional level. However, few countries have passed national legislations to enforce these changes. The recommended standards and regulations for EAC in this report do not only look within EAC countries but also as an integral part of tripartite process of harmonization of transportation instruments.

The objectives under the component reported in this chapter were to:

- Review standards and regulations on vehicle dimensions and combination in the partner states, SADC, and COMESA regions, and identify dimensional parameters and vehicle combinations which are regulated.
- Identify areas of differences and commonality within the partner states and in SADC and COMESA.
- Propose harmonized standards and regulations to be used in all the EAC partner states.

Standards which are recommended for harmonization include:

- ***Vehicle main dimensions:*** overall length, width, and height for a rigid single vehicle (including trailer), articulate vehicle (or truck tractor), and for (any) combination of vehicles
- ***Vehicle control dimension***: Minimum turning radius, wheelbase, front and rear overhangs, and maximum projections of carried loads at front and rear, and at the sides of the vehicle
- ***Warning requirements for projecting loads***
- ***Restrictions on vehicle combinations***
- ***Vehicles exempted from the provisions*** and their regulations when using public roads: These include agricultural and construction machinery.

The existing vehicle dimensions standards in the EAC countries and the respective recommended standards for harmonization are as follows.

Standard	Existing	Recommended
Maximum overall length of vehicles (m)	Single Unit Vehicle (SU) Tanzania (12.5), Kenya (11.0), Uganda (12.5), Rwanda (11.0), Burundi (12.0), Zanzibar (12.5)	12.5
	Articulated Vehicle(semi-trailer & SU + trailer) Tanzania (17.0), Kenya (17.4), Uganda (17.0), Rwanda (17.4), Burundi (17.4), Zanzibar (17.0)	17.4
	Combination Vehicles (Interlinks) Tanzania (22.0), Kenya (22.0), Uganda (22.0), Rwanda (18.0), Burundi (18.0), Zanzibar (22.0)	22.0
Maximum overall width of vehicles (m)	Bus (wheel track >1.9 m) and Goods Vehicle Tanzania (2.6), Zanzibar (2.6) Refrigerated Truck Burundi (2.65) Any Other Vehicle Tanzania (2.6), Kenya (2.65), Uganda (2.5), Rwanda (2.65), Burundi (2.5), Zanzibar (2.6)	2.65
Maximum overall height (m)	Any Vehicle Tanzania (4.6), Kenya (4.2), Uganda (4.0), Rwanda (4.2), Burundi (4.2), Zanzibar (4.6)	4.6
Minimum Turning Radius (m)	Any Other Vehicle Tanzania (12.65), Zanzibar (12.65) Kenya (12.5)	12.65
Maximum Wheel Base (m)	Semi-trailer Tanzania (10.0), Zanzibar (10.0)	10.0
	Any Other Vehicle Tanzania (8.5), Zanzibar (8.5)	8.5
Maximum Front Overhang (m)	Semi-Trailer Tanzania (1.8), Zanzibar (1.8)	1.8
	Any Other Vehicle [1]Tanzania and Zanzibar [(smaller of 60% of WB or 5.8 – (0.5xWB))];	Small of 60% of WB or 5.8 – (0.5xWB))

	Kenya (= 60% of WB); Rwanda (smaller of 2.7m or 55% of WB) Where: WB = Wheel base	
Maximum Rear Overhang (m)	Any Other Vehicle Tanzania and Zanzibar (60% of WB) Kenya (60% of WB) Rwanda (smaller of 3.5 m or 0.65 x WB)	60% of WB
	[2]Trailer Tanzania and Zanzibar (50% of trailer body)	50% of trailer body
Projecting load limits (front and rear)	Bus and Goods Vehicle Tanzania (1.3), Zanzibar (1.30) Any Other Vehicle Tanzania (1.25), Kenya (1.8), Uganda (1.25), Rwanda (2.7F, 3.5R), Burundi (-), Zanzibar (1.25)	1.25
Projecting load limits (sides)	Tanzania (0.15), Kenya (0.15), Uganda (0.15), Rwanda (-), Burundi (-), Zanzibar (0.15)	0.15

[1]Except semi-trailer or trailer with one axle or one axle unit
[2]Trailer other than semi-trailer with single axle or two axles with axle distance < 1.2m

On the warning requirements for protruding loads carried by a vehicle to warn other road users the study is recommending adopting the SADC regulations which was developed for use in its partner states. The SADC regulation is detailed, easier to apply and is much safer compared to the regulations which are used in the EAC countries. Tanzania has already adopted the SADC regulations. We thus recommend that this provision be adopted as the EAC standard.

SADC has also developed regulations on restrictions of certain combinations of vehicles to be used in the member states and the same regulations are in the ongoing process to be harmonized in the entire EAC-SADC-COMESA Region. In this regard we also recommend adopting the SADC regulation on restrictions on vehicle combinations. Furthermore, the Study conducted by PADECO[1] offered recommendations to consider technological advances on vehicle combination such as *interlinks* which are widely used in some Southern African SADC countries and include them in the current regulations in the EAC countries, provided the combination do not exceed dimensional, load or manoeuvrability limits.

The *interlink vehicle combinations* have two significant advantages over the equivalent 56-ton truck and trailer combination due to their two rotating connections being further apart, firstly they are more stable on the road and therefore improve road safety. And secondly, they are flexible and convenient because they can be unhooked and pre-loaded, and one tractor can operate several

[1]PADECO (2011). Study for the Harmonization of Vehicle Overload Control in the East African Community. Draft Final Report

semi-trailers and therefore fleet utilization can be improved, whereas, the drawing vehicle in a truck-trailer combination cannot be utilized in such a manner.

9 HARMONISATION OF VEHICLE DIMENSIONS AND COMBINATIONS

9.1 Introduction

9.1.1 Background

Standards and regulations on vehicle dimensions and combinations are essential from the safety aspects of vehicle and of other road users. It is important to ensure that characteristics of all vehicles using public roads are compatible with the geometric design of the road network. Non-regulated standards within a country or between countries which have cross-border freight transport are a potential hazard for safety of the vehicles, road infrastructure, and all road users. Therefore, the process to harmonize the physical standards and configuration of vehicles between countries which have different road infrastructure design need a careful approach in considering the technical impacts and consequences which might arise from the changed standard in a particular country for the purpose of harmonization.

The infrastructure design standards are not uniform in all EAC member countries. In addition to that, vehicle owners in EAC Partner States imports vehicles from different countries in the world, mostly from developed countries, which also have different manufacturing standards and infrastructure design standards. Therefore, the process of harmonizing standards should address road infrastructure, regulatory, and institutional issues, without marginalizing the full benefits of inter-state freight movement by trucks and the primary objective for harmonization.

COMESA, EAC and SADC have started the process of harmonization of vehicle dimensions and have agreed on maximum vehicle dimensions, in terms of height, width and length of vehicles. In principle tripartite Partner States have agreed to these vehicle dimensions limits at the regional level. However, few countries have passed national legislation to enforce these changes. The recommended standards and regulations for EAC in this report do not only look within EAC countries but also as an integral part of tripartite process of harmonization of transportation instruments.

9.1.2 Objectives and Study Methodology

The objectives of this task were:

- To review standards and regulations on vehicles dimensions and combination in the EAC Partner States, SADC, and COMESA regions and identify dimensional parameters and vehicle combinations which are regulated
- Identify areas of differences and commonality within the Partner States and in SADC and COMESA.
- To propose harmonized standards and regulations to be used in all EAC Partner States.

To achieve the objectives we adopted a broad methodology which included review of the relevant Partner States documents and best practices literature, interviewing experts and interacting in workshops organized in each Partner State.

9.2 Overview of Vehicle Dimensions and Combinations

During the review of standards and literature it was evident that there is confusion and mix-up of terminologies and the way various dimensions of the vehicle are defined. This has necessitated to first considering consistency (or harmonization) in terminologies and definitions in vehicle dimensions and also in vehicle combinations in order to avoid confusion. This is important for the purpose of regulatory control in the application of dimensional limits and combination of vehicles in the EAC Partner States.

9.2.1 Definitions and Terminologies[2],[3],[4]

Trailer:	Any vehicle without a live axle, and is designed to be drawn by a power-driven vehicle. It includes a semi-trailer, a full trailer or a pony trailer.
Full trailer:	*Or dog trailer* (Australia) is a vehicle designed and used so that its weight and load is carried on its own axles, and includes a combination of vehicles consisting of a semi-trailer and a converter dolly;
Pony trailer:	A vehicle with one axle group close to the centre of its chassis that carries the preponderance of its weight and load, and that is towed by a drawbar that is rigidly attached to the structure of the trailer.
Semi-trailer:	Is a trailer without a front axle. Any trailer designed to be coupled to a special motor vehicle called a *semi-truck* or *road tractor* (UK) or *prime mover (Australia)* in such a way that part of it rests on the motor vehicle and that a substantial part of its weight and of the weight of its load is borne by the road tractor. A semi-trailer can also be coupled to a *dolly* which is normally equipped with landing gear (legs which can be lowered) to support it when it is uncoupled from the road tractor.
Trailer wheelbase:	Is the longitudinal distance from the centre of the kingpin of a semi-trailer the turntable of a full trailer, or the hitching device on a pony trailer to the trailer turn centre
Drawing vehicle:	*Or prime mover:* Is the vehicle which pulls another vehicle (normally a trailer)
Truck:	Is a motor vehicle, other than a bus, that is designed and normally used to carry cargo, and that may operate as a single unit or may pull a trailer other than a semi-trailer.
Tractor:	*Or tractor truck or horse: Is* a motor vehicle designed and normally used to pull a semi-trailer, a semi-trailer and a full trailer or a semi-trailer and another semi-trailer
Tractor wheelbase:	The longitudinal distance from the centre of the steering axle to the geometric centre of the drive axle unit
Combination of vehicles:	Coupled vehicles which travel on the road as a unit.

[2] Weights and Dimensions of Vehicles Regulations made under Section 191 of the Motor Vehicle Act R.S.N.S. 1989, c. 293 O.I.C. 2001-526 (November 15, 2001), N.S. Reg. 137/2001 as amended up to O.I.C. 2010-5 (January 12, 2010), N.S. Reg. 4/2010. CANADA

[3] Convention on Road Traffic, Vienna, 1968.

[4] Vehicle Standards Information for Vehicle Owners & Operators in New South Wales. No. 5. Revision 3, May 1998

Articulated vehicle:	Also "*Tractor semi-trailer" or "semi-trailer truck":* Combination of vehicles comprising a special motor vehicle (road tractor) and a coupled semi-trailer
Converter dolly:	Is a small trailer that can be coupled to a truck or trailer so as to support a semi-trailer. The dolly is equipped with a *fifth wheel* to which the semi-trailer is coupled.
A-dolly:	A converter dolly that is towed from one hitch on the centre line of the towing vehicle
C-dolly:	Means a converter dolly with a frame rigid in the horizontal plane that is towed from 2 hitches located in a horizontal transverse line on the towing unit that precludes any rotation in the horizontal plane about the hitch points.
Fifth wheel assembly:	A coupling device for the purpose of supporting and towing a semi-trailer, whose lower half is composed of a plate and locking jaws and is mounted on the rear portion of a vehicle frame or the frame of a converter dolly and whose upper half is composed of a plate and a kingpin fastened to the underside of the forward portion of the semi-trailer;
A-train double:	A combination of vehicles composed of a road tractor, a semi-trailer and either an A-dolly and a semi-trailer or a full trailer attached to the foremost semi-trailer in a like manner as if an A-dolly were used
B-train double:	A combination of vehicles composed of a road tractor, a semi-trailer, and a second semi-trailer towed by the lower half of a fifth wheel assembly mounted on the rear of the foremost semi-trailer;
C-train double:	A combination of vehicles composed of a road tractor, a semi-trailer, a C-dolly and a second semi-trailer
Road train	A road train consists of a conventional tractor unit, but instead of pulling one trailer or semi-trailer, the road train pulls two or more of them. It is a trucking concept used in some parts of the world (Argentina, Australia, Mexico, the United States and Canada) to move freight efficiently.
Axle:	An assembly of 2 or more wheels whose centres are in 1 transverse vertical plane and that transmit weight to the road
Axle group	A single axle, a tandem axle, tridem axle, or a self-steering quad axle
Tandem axle:	An axle group containing 2 consecutive axles that does not include any liftable or self-steering axles, and that has the same number and size of tires on each axle
Drive axle:	*Or live axle:* The axle or axle group that is connected to the power source of a motor vehicle and that transmits power to the wheels
Steering axle:	An axle whose steering is controlled by the driver of a vehicle in order to control the direction of travel of the vehicle;

Inter-axle spacing:	The longitudinal distance between the centre of the rearmost axle of an axle group and the centre of the foremost axle of the next axle group to the rear, within a vehicle or combination of vehicles;
Drawbar:	A structure connected to the chassis frame of a trailer or converter dolly that includes a device for coupling to a hitch on a towing vehicle
Drawbar length:	For a pony trailer, a full trailer or a converter dolly, means the longitudinal distance from the centre of a device that attaches to a hitch on the towing vehicle to the centre of the lower half of a fifth wheel assembly or turntable
Rear overhang:	Is the longitudinal distance from the centre of the rear axle or axle group on a truck or the geometric centre of the rear axle or axle group on a trailer to the rearmost point of the truck or trailer.
Front overhang:	Is the longitudinal distance from the centre of the front axle to the front most point of the vehicle.
Overall height:	The vertical distance between the highest point on the vehicle or combination of vehicles, including cargo, and the surface of the road
Overall length:	The longitudinal distance from the foremost point of a vehicle or a combination of vehicles to the rearmost point of a vehicle or combination of vehicles, including cargo, exclusive of any extension in the distance caused by an aerodynamic device at the rear
Overall width:	The greatest transverse dimension of a vehicle or combination of vehicles, including cargo, excluding side mirrors.
Track width:	The overall width of an axle across the outside faces of the tires
Minimum turning radius:	Is the size of the smallest circular turn that the vehicle is capable of making

9.2.2 Impact of Vehicle Dimensions and Combinations on Safety, Performance, and Geometric Design

The compatibility of heavy vehicles when turning and manoeuvring with the physical constraints of road geometry is an important component of vehicle dimension regulations. The limits on the *main dimensions* and *control dimensions* for rigid vehicles and combination of vehicles are required to ensure safety and more importantly the compatibility of the road geometry with the turning and manoeuvring characteristics of vehicles. The characteristics of vehicles and combinations of vehicles which are of primary concern in assessing compatibility with highway geometry include:

- Low speed off-tracking performance
- High speed off-tracking performance
- Swept path when turning
- Swing out of the front and rear of the vehicle
- Swing out of protruding load when turning, and
- Vertical clearance.

The configurations and combinations of heavy vehicles have markedly different characteristics than smaller vehicles in a number of areas including: acceleration performance, braking performance, stopping distance, rollover threshold, and turning characteristics.

When changes are made to the highway system it is often made to better accommodate the characteristics of large trucks such as passing and climbing lanes and pavement widening on curves. In addition, the size difference between small cars and heavy trucks has an influence on traffic safety in a number of areas including:

- positioning and visibility of road signs,
- passing zone striping,
- traffic signal timing, and
- intersections and ramp designs.

As the size and weight of vehicles increases the stability and control characteristics of the vehicles also become affected, and virtually every regulation imposed on the size or weight of heavy trucks have somehow an influence on the stability, handling or turning characteristics of the vehicle.

The most influential variable affecting the rollover resistance of heavy trucks is the height of the centre of gravity of the vehicle. The higher the centre of gravity, the lower the rollover threshold will be. Rolling stability of vehicle generally improves as the wheelbase increases, the increase in wheel track, and with fewer trailers and coupling points.

Furthermore, it has been reported that[5,6] the roll stability and handling characteristics of vehicle combinations are subject to a phenomenon called rearward amplification (RA), where the forces resulting from action taken by the driver such as swerving to avoid a pothole, or rapid lane changing are amplified as they move through the rear trailers with a consequence of producing higher rollover forces at the rear trailer than at the drawing vehicle at the front. RA is usually measured in terms of yaw velocity or lateral acceleration gain. The rearward amplification increases when the velocity increases. In addition, for some types of vehicle combinations it is relevant to use the yaw damping

[5] Harmonization of Vehicle Weight and Dimension Regulations within the NAFTA Partnership. North American Free Trade Agreement Land Transportation Standards Subcommittee. Working Group 2 - Vehicle Weights and Dimensions. Report to the Land Transportation Standards Subcommittee *October 1997*

[6] Vehicle combinations based on the modular concept. Background and analysis. *John Aurell & Thomas Wadman, Volvo Trucks.* Report no. 1/2007 Committee 54: Vehicles and Transports

of the lightest damped mode during free oscillations as a stability criterion. The damping always becomes lower for increased speed. The yaw damping rate describes how quickly the amplitudes of the oscillations are attenuated. RA may be determined in different manoeuvres. The result depends on the type of manoeuvre.

The turning radius and space required by vehicles and vehicle combinations increases as the wheelbases of rigid vehicle or coupled vehicles increases, and they decrease as the number of connection points increases. In combinations of vehicles, turning performance is a function of the length of the wheelbases of each unit including any converter dollies. Hence, the overall length of the combination of vehicles is not directly related to the turning space. It is known that a typical triple trailer combination can turn in about the same space as a conventional tractor semi-trailer, even through the triple combination may be 10 meters longer than the tractor semi-trailer.

Vehicle dimensions are categorized as ***main dimensions*** and ***dimension controls:***

- **Main Dimensions** include: overall length, width, and height for a rigid single vehicle (including trailer), articulate vehicle (or truck tractor), and for (any) combination of vehicles.
- **Dimension Controls** include: the minimum turning radius, wheelbase, front and rear overhangs, and projections of carried loads at front and rear, and at the sides of the vehicle:
- **Vehicle Main Dimensions (Overall length, width and Height)**

9.2.2.1 Main dimensions

a) Overall Length of the Vehicle

Overall length of the vehicle or combination of vehicles is the longitudinal distance from the foremost point of a vehicle to the rearmost point of a vehicle or combination of vehicles, including cargo, exclusive of any extension in the distance caused by an aerodynamic device at the rear. Limiting factors for a rigid vehicles or combination of vehicles length are:

- **Safety:** Passing or overtaking safely within the passing sight distance and passing zone striping practices
- **Road Geometry**: Compatibility of the road geometry with the turning and manoeuvring characteristics of vehicles at the length limit selected.

Figure 1 illustrates the overall length for vehicles or combination of vehicles.

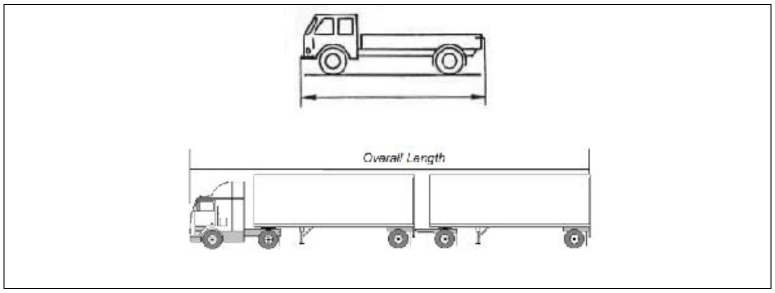


Figure 1: Overall length for vehicles or combination of vehicles

b) Overall Width of Vehicle

Overall width of vehicles is the largest transverse dimension of a vehicle or combination of vehicles, including cargo, excluding side mirrors. A limit on overall width of vehicles is needed to ensure that vehicles are compatible with the space available within a single lane in the class of roads which that particular type of vehicle is allowed to operate. The rolling stability of the vehicles, also improves as the width of the wheel track increases. Figure 2 illustrates the width of the vehicle.

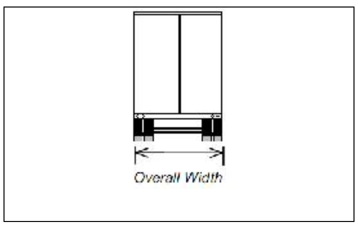


Figure 2: Overall width of a vehicle or combination of vehicles

c) Overall Height of the Vehicle

Overall vehicle height is the vertical distance between the highest point on the vehicle or combination of vehicles (including cargo), and the surface of the road. The most influential variable affecting the rollover resistance of vehicles is the height of the centre of gravity of the payload; the higher the centre of gravity, the lower the rollover threshold will be. When loaded, a truck with higher body structure will have its centre of gravity raised depending on how the cargo body would allow.

A limit on overall height limit on vehicles is also required to ensure there is adequate clearance between vehicles and overhead structures on the roadway. Figure 3 illustrates the overall height of the vehicle.

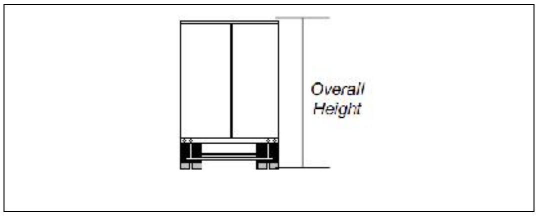


Figure 3: Overall height of the vehicle

9.2.2.2 Dimension Controls (Turning radius, wheelbase, front/rear overhang, and Projection of Loads)

a) Minimum Turning Radius and Swept Path

Minimum turning radius is the size of the smallest circular turn that the vehicle is capable of manoeuvring. Whereas, when a vehicle makes a turn, especially a sharp turn to the left or right, the rear wheels do not follow in the same path (track) of the front wheels. This is similar to off-tracking. This characteristic increases with increasing wheelbase of the vehicle. The limiting factor for vehicle minimum turning radius and swept path is compatibility of vehicle with the roadway geometry. Figure 4 shows the minimum turning radius and swept path.

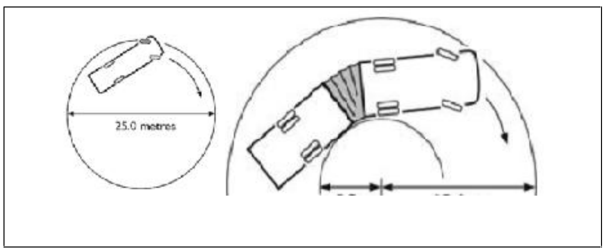


Figure 4: Minimum turning radius (left) and swept path (right). Swept path = Ro – Ri

b) Wheelbase

For a trailer, wheelbase is the longitudinal distance from the centre of the kingpin of a semi-trailer, the turntable of a full trailer, or the hitching device on a pony trailer to the trailer turning centre. For a rigid vehicle and tractor, it is the longitudinal distance from the centre of the steering axle to the geometric centre of the rear axle unit. The length of a vehicle wheelbase is a major factor in the manoeuvrability of the vehicle. The longer the wheelbase, the less manoeuvrable the vehicle will be. Hence, the limiting factor for vehicle wheelbase is road geometry. Figure 5 illustrates the wheelbase for tractor and semi-trailer.

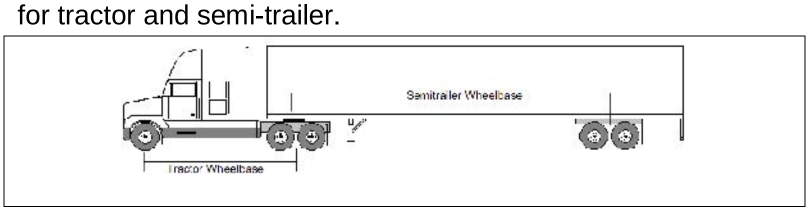


Figure 5: Wheelbase for road tractor and semi-trailer

c) Front and Rear overhang

Rear overhang is the longitudinal distance from the centre of the rear axle or axle group on a truck or the geometric centre of the rear axle or axle group on a trailer to the rearmost point of the truck or trailer. Whereas, front overhang is the longitudinal distance from the centre of the front axle to the front most point of the vehicle.

Overhang in either the front or rear also affect the manoeuvrability of the vehicle. When front overhang is excessive, the front corner of the vehicle will travel in a wider arc (swing out) and may come into contact with a vehicle in the other driving lane or in the opposite corner, road signs and pedestrians on the other side of the road. The affect on the manoeuvrability of a vehicle with excessive rear overhang is similar to that with excessive front overhang, that is, the rear part of the vehicle will swing wide and may come into contact with an object on the other side of the road or a vehicle travelling in the another lane. Therefore, limiting factor for vehicle front and rear overhangs is the road geometry. Figure 6 illustrates the front and rear overhangs of vehicle.

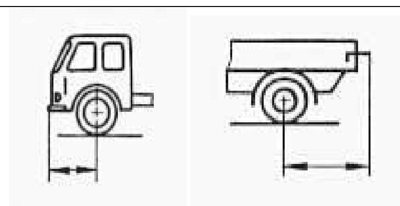


Figure 6: Front overhang (left) and rear overhang (right)

d) Projecting Loads

The effects of projecting load at the front and rear of the vehicle are the same as excessive front and rear overhangs, respectively. Hence, the reasons for their limits are the same. For the side projections the reason for limiting is the road lane width, which is an important road geometric parameter. Figures 7 and 8 illustrate projecting loads on the vehicle.

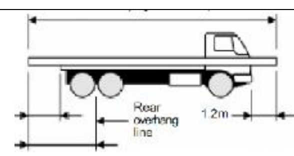


Figure 7: Projecting load at the front and rear of the vehicle

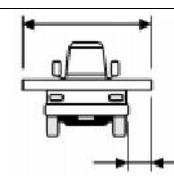


Figure 8: Projecting load at the sides of the vehicle

9.2.2.3 Vehicles Combination

In both developed and developing countries there are many vehicle combinations which are operating in public roads. The diversity of vehicle combinations makes regulations to vary in content and detail between countries depending on economic and safety priorities and the parent acts from which the regulations are derived. However, from the technical point of view, the major limitations for vehicle combinations are compatibility with road geometry and safety performance of the combinations.

Vehicle parameters which are commonly regulated in combination of vehicles are maximum length; axle weights, gross vehicle weight, number and spacing of axles, and number of vehicles (including trailers) in the combination. These parameters affect pavement and bridges loading, traffic operations on highway geometry, highway capacity, safety, and transport productivity.

Economies of scale have been of increasing importance in transportation industries, including vehicle manufacturing and transportation, and have thus contributed to the development of modern transport systems. In road transport, the infrastructure is difficult to change to accommodate bigger vehicles, hence vehicle width and height are almost fixed by the road systems, and effort is only directed to design longer vehicles which can manoeuvre in the existing roads. Vehicles combination or vehicles with extended length and weight limits have the potential to make freight transport more efficient. A number of countries, including Australia, Brazil, Canada, Finland, Sweden and the United States of America, have successfully demonstrated the use of road vehicles with extended dimensions for freight transport.

9.3 Vehicle Dimensions and Combination Standards in EAC Countries

Tanzania (Mainland and Zanzibar) has changed the regulations and standards for vehicle dimensions and combinations to follow SADC regulations. Tanzania has fully adopted the SADC regulations and has been included in the parent act, whereas Zanzibar is in the process of changing the regulations in the parent act. Therefore the standards for vehicle dimensions and combinations presented for Tanzania are the same as those of SADC.[7,8]

Table 1 indicates the source(s) which were reviewed for each EAC country and from which the standards and regulations for vehicles dimensions and combinations were extracted for comparison with other EAC countries in this task.

[7] Road Traffic Act, 1973 (Amended 1996). National Road Traffic Regulations, 1999. Tanzania

[8] Guidelines on Harmonization of Vehicle Regulations Dimensions of Vehicles. SADC. 2009.

Table 1: Source(s) for data and information on vehicles dimensions and combinations for each EAC country

Country	Status
Tanzania (Mainland & Zanzibar)	• Road Traffic Act 1973 (amendment of 1996). National Road Traffic Regulations, 1999. • Report of the Meeting of the Technical Committee on Axle Load Limits Implementation in the East African Community. EAC Secretariat. August 2007 • Guidelines on Harmonization of Vehicle Regulations Dimensions of Vehicles. SADC. 2009
Kenya	• Report of the Meeting of the Technical Committee on Axle Load Limits Implementation in the East African Community. EAC Secretariat. August 2007 • Kenya Traffic Act. Chapter 403. Revised Edition 2009 (1993) • KS 372:2011. Road Vehicles – Passenger Vehicle Body Construction-Specifications. KEBS 2010. 3rd Edition.
Uganda	• The Traffic and Road Safety (weighbridge) Act, 2004 Supplement No. 21. Uganda Gazette No.35, Vol. XCVII July 2004 • Report of the Meeting of the Technical Committee on Axle Load Limits Implementation in the East African Community. EAC Secretariat. August 2007
Rwanda	• Road traffic and Transport Act. Official Gazette of the Republic of Rwanda. Kigali, on 02/09/2002 • Report of the Meeting of the Technical Committee on Axle Load Limits Implementation in the East African Community. EAC Secretariat. August 2007
Burundi	• Regulation of the Maximum Axle Load of Vehicles Circulating in the Territory of Burundi. Bujumbura, August 1993. Ministerial Order No.720/70. Minister Of Public Works And Equipment & Minister Of Transports, Posts And Communications (translated)

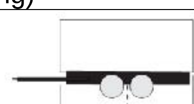
9.3.1 Maximum Overall length of the Vehicle (meters)

Tables 2 to 6 summarize the maximum overall lengths for different type of vehicles and combination of vehicles in EAC Partner States.

Table 2: Maximum length for a rigid vehicle (Excluding semi-trailer, including drawbars and coupling)

Tanzania	Kenya	Uganda	Rwanda	Burundi
12.5	11.0	12.5	11.0	12.0

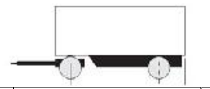
Table 3: Maximum length for a trailer with one axle or Axle unit (Excluding semi-trailer, excluding drawbars and coupling)



Tanzania	Kenya	Uganda	Rwanda	Burundi
[1]8.0 [2]11.3	[3]11.0	12.5	7.0	

[1]GVM <12,000 kg; *[2]GVM>12,000 kg* *[3]Including drawbar in horizontal position*

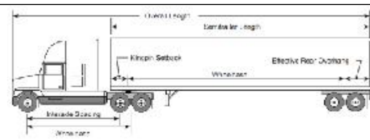
Table 4: Maximum length for other Trailers not referred in Table 1B with GVM>12,000 kg, (excluding semi-trailer, excluding drawbars and coupling)



Tanzania	Kenya	Uganda	Rwanda	Burundi
12.5	[1]11.0	12.5	11.0	

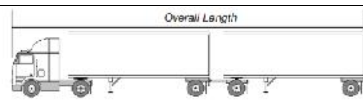
[1]Including drawbar in horizontal position

Table 5: Maximum overall length for articulate Vehicle or Tractor Semi-trailer (including drawbars and coupling)



Tanzania	Kenya	Uganda	Rwanda	Burundi
18.5	17.4	17.0	17.4	17.4

Table 6: Maximum overall length for any combination of vehicles including bus-train (including drawbars and coupling)



Tanzania	Kenya	Uganda	Rwanda	Burundi
22.0	22.0	22.0	18.0	18.0

9.3.2 Maximum Overall Width of the Vehicle (meters)

Table 7 summarizes the maximum overall width for different categories of vehicles in EAC Partner States.

Table 7: Maximum Overall Width (Excluding side mirrors) for different categories of vehicles

Tanzania	Kenya	Uganda	Rwanda	Burundi
2.5 [1]2.6	2.65	2.5	2.65	2.5 [2]2.65

[1] A bus: Wheel track > 1.9 m or Goods Vehicle: GVM>12,000 kg[2] Refrigerated and container vehicles

9.3.3 Maximum Overall Height of the Vehicle (meters)

Table 8 displays the maximum overall height for different categories of vehicles in EAC Partner States.

Table 8: Maximum overall height for different categories of vehicles

Tanzania	Kenya	Uganda	Rwanda	Burundi
4.6 [1]4.65	4.2	4.0	4.2	4.2

[1]Double decker bus

9.3.4 Minimum Turning Radius (meters)

Table 9 shows the minimum turning radius for different categories of vehicles in EAC Partner States.

Table 9: Minimum Turning Radius for different categories of vehicles

Tanzania	Kenya	Uganda	Rwanda	Burundi
13.1	12.5			

9.3.5 Maximum Wheelbase (meters)

Maximum wheelbase information is available from Tanzania (which conforms to SADC) for:

- rigid except bus-train (8.5 m)
- semi-trailer (10.0 m)
- Bus-train 15.0 m

No information was available from the other Partner States.

9.3.6 Maximum Front and Rear overhangs (meters)

Table 10 illustrates front and rear overhangs for different categories of vehicles

Table 10: Maximum Front and rear overhangs for different categories of vehicles

OVERHANG	Tanzania	Kenya	Uganda	Rwanda	Burundi
FRONT	[1]a [2]1.8	e		b	
REAR	e [3]c	e		d	

[1]For any other vehicle,
a: For vehicles where the distance from the front end of the vehicle to the backrest of the driver's seat at rearmost position is less than 1.7 m: smaller of 60% of WB or [6.2 – (0.5xWB)];
For Driver's seat position more than 1.7 m: smaller of 60% of WB or [5.8 – (0.5xWB)];
[2]Semi-trailer
[3]Trailer other than semi-trailer with single axle or two axles with axle distance < 1.2m, c = 0.5 x length of trailer body
b = smaller of 2.7m or 55% of WB
d = smaller of 3.5 m or 0.65 x WB
e = 60% of WB

9.3.7 Projecting Load Limits (meters)

Projecting loads are not allowed to:

- Exceed the overall length prescribed in section 4.1 (Maximum overall length of the vehicle) for that particular category of vehicle.
- Exceed the allowed front overhang limit as provided in that particular category of vehicle as provided in section 4.6 (Front and Rear Overhangs)
- Exceed 300 mm from the front end of the vehicle

Table 11 shows the standard for the limits on projecting loads in EAC Partner States.

Table 11: Maximum limits on projecting loads at the front, rear, and sides for different categories of vehicles

PROJECTION	TANZANIA	KENYA	UGANDA	RWANDA	BURUNDI
FRONT (F) and REAR (R)	1.25 [1]1.3	1.8	1.25	2.7 (F) 3.5 (R)	
SIDE	0.15	0.15	0.15		

[1]Bus and goods vehicle

9.3.8 Warning Requirements for Projecting Loads

9.3.8.1 Tanzania

The warning requirements for projecting loads used by Tanzania are those which have been developed by SADC. The regulation is detailed in content and specifications on the marking colours, dimensions of markings used to indicate projecting loads, and the time of operation of vehicle with such carried loads. The SADC regulation[9] is reproduced below as follows:

- *No person shall operate a vehicle on a public road if the load on such vehicle projects more than 150 millimetres beyond the side thereof, unless -*

  (a) during the period between sunset and sunrise and at any other time when, due to insufficient light or unfavourable weather conditions, persons and vehicles upon the public road would not be clearly visible at a distance of 150 metres, the extent of such projection is indicated -

    (i) by means of either a white retro-reflector or a lamp emitting a white light, fitted at the outer edge of the front of such load; and

    (ii) by means of either a red retro-reflector or a lamp emitting a red light, fitted at the outer edge of the rear of such load; and

  (b) during any other period, the extent of such projection is indicated by means of flags of red cloth, not less than 300 millimetres by 300 millimetres, suspended by two adjacent corners thereof transversely to the direction in which the vehicle is travelling, from the front and rear of such projection.

- *No person shall operate a vehicle on a public road if the load on such vehicle projects more than 300 millimetres beyond the rear thereof, unless -*

  (a) during the period between sunset and sunrise and at any other time when, due to insufficient light or unfavourable weather conditions, persons and vehicles upon the public road would not be clearly visible at a distance of 150 metres -

    (i) the width of such projection is indicated by means of red retro-reflectors or lamps emitting a red light fitted on the end of such projection: Provided that where the width of any such projection is less than 600 millimetres it shall be sufficient for the purpose of indicating such width to fit one retro-reflector or lamp on the end thereof; and

    (ii) the length of such projection is indicated by means of yellow retro-reflectors or lamps emitting a yellow light fitted on both sides of such projection at the end thereof; and

  (b) during any other period, the length of such projection is indicated by means of a red flag or red cloth, not less than 300 millimetres by 300 millimetres, suspended by two adjacent corners thereof transversely to the direction in which the vehicle is travelling, from the end of such projection, and the width of such projection is

[9] *Guidelines on Harmonization of Vehicle Regulations Dimensions of Vehicles. SADC. 2009*

indicated by means of such flags suspended by two adjacent corners thereof parallel to the direction in which the vehicle is travelling, from both sides of such projection at the end thereof: Provided that where the width of such projection is less than 600 millimetres it shall be sufficient for the purposes of indicating such projection to suspend one such flag from the end thereof.

- *For the purposes of this regulation, the light of any lamp shall comply with the provisions of the visibility distance requirements of lights.*

9.3.8.2 Kenya

Warning is mandatory for loads which projects by more than 60 cm at the rear. The regulation is given in the Traffic Act[10] as follows:

No load shall be carried on a vehicle –

- *which projects beyond the maximum overall length of the vehicle by more than 1.8 metres;*
- *which projects to the rear beyond the maximum overall length of the vehicle by more than 60 cm., but a load may project not more than 1.8 metres where the rear extremity of the load is plainly indicated by a conspicuous red marker during the day and by a red light at night.*

9.3.8.3 Uganda

The regulation in Uganda stipulates that a vehicle, trailer or engineering plant shall not carry a load if the load projects beyond 0.15 m at the sides, and 1.25 m at the rear. Except that if the load exceed 1.25 and not more 1.8 m it has to clearly be indicated by a conspicuous red marker during the day, and red light at night.

9.3.9 Combination of Vehicles

9.3.9.1 Tanzania

Tanzania follows SADC regulations[11] for combination of vehicles. SADC have developed regulations to restrict certain combination of vehicles to be used in the Partner States. The regulations are as summarized below:

- Subject to mass regulation restriction, no person shall operate on a public road any combination of motor vehicles -
  - (a) other than a drawing vehicle and one or two trailers;
  - (b) other than a motor vehicle drawing one other motor vehicle which is not a trailer; and
  - (c) other than a motor vehicle drawing another motor vehicle which is not a trailer, and a trailer, in the case of an emergency or a breakdown,
- Subject to mass regulation restriction, no person shall operate on a public road a combination of motor vehicles -
  - (a) consisting of a trailer attached to a drawing vehicle in such a manner that the combination of trailer and drawing vehicle cannot bend in a horizontal plane, if the combined length of such trailer, including any drawbar or coupling and the rear overhang of the drawing vehicle, exceeds three comma one metres;

[10] *Kenya Traffic Act. Chapter 403. Revised Edition 2009 (1993)*

[11] *Guidelines on Harmonization of Vehicle Regulations. Dimensions of Vehicles. SADC. 2009*

(b) consisting of a trailer or trailers attached to a drawing vehicle if the length of the drawbar of any trailer in such combination, where such trailer has more than one axle, exceeds 2 metres: Provided that in the case of an underslung coupling, the drawbar may exceed two metres if the distance between the two vehicles does not exceed 2.5 metres.

- Subject to mass regulation restrictions, any combination of vehicles may be towed by a breakdown vehicle if the brake system of the towed combination, excluding the drawing vehicle of such combination, is functional and coordinated with the working of the brakes of the breakdown vehicle.
- When a drawing vehicle is pulling two trailers, a converter dolly shall, when used in combination with a semi-trailer, be deemed not to be a trailer.
- Notwithstanding anything contained in this regulation, no person shall use a breakdown vehicle or motor vehicle, that must display the gross combination mass in kilograms (denoted as GCM), to tow or draw another vehicle on a public road, unless such information is displayed on such breakdown vehicle or motor vehicle.

9.3.9.2 Kenya

The regulation for vehicles combination is contained in the National Traffic Act[12] and it states as follows:

- No motor vehicle shall tow more than one trailer or other towed vehicle on a road; but a semi-trailer directly superimposed on the drawing vehicle shall not be counted as a trailer.
- Where a trailer or vehicle is towed on a road by a motor vehicle the towing vehicle shall not be more than 4.5 metres long.
- No trailer shall be used on a paved or all weather road if the weight or dimensions of the trailer exceed those laid down for a motor vehicle having the same number of axles and axle configuration
- No vehicle with a rigid chassis shall have more than three axles except for a vehicle with two steering axles and two rear axles.
- The maximum number of axles which may be fitted on any combination of a motor vehicle and a semi-trailer or motor vehicle and drawbar trailer shall be seven.
- The maximum number of axles in any axle group fitted on any trailer shall not exceed (a) on a drawbar trailer, 2 axles: and (b) on a semi-trailer, 3 axles.

9.3.9.3 Rwanda

The provision for Rwanda states that[13]; a self propelling vehicle and an animal draught vehicle cannot pull more than two vehicles. However, a *mobylette* with a side-car is not allowed to pull any trailer.

9.3.9.4 Burundi

The restrictions for vehicle combinations for Burundi are summarized as follows[14]

[12] *Kenya Traffic Act. Chapter 403. Revised Edition 2009 (1993)*

[13] Rwanda Road traffic and Transport Act. Official Gazette of the Republic of Rwanda. Kigali, on 02/09/2002

[14] Regulation of the Maximum Axle Load of Vehicles Circulating in The Territory of Burundi. Bujumbura, August 1993. Ministerial Order No.720/70. Minister Of Public Works And Equipment & Minister Of Transports, Posts And Communications (translated)

- Any combination has to abide by the limits on overall length, width, and height.
- A motor vehicle can draw only two vehicles.
- With exceptions, train of four vehicles is subject to authorization by the services of the Ministry in charge of road transport and the Ministry in charge of the construction of roads.

9.3.10 Vehicles Exempted from the Provisions

Agricultural and construction machinery are exempted from provisions on vehicle dimensions and combinations in most EAC countries. Provisions for exempted vehicles for Tanzania, Kenya and Burundi are presented below. Provisions for Uganda and Rwanda were not obtained.

9.3.10.1 Tanzania

Tanzania follows SADC regulations[15] for exemption on provisions on vehicle dimensions and combinations for certain types of vehicles including agricultural and construction machinery. The highlights on the provision are as follows:

- Except for a breakdown vehicle, any vehicle, which is a drilling machine, a mobile crane, a fork lift, a straddle truck, a road making machine, an earthmoving machine, an excavation machine, a construction machine or a loading machine is exempt from the provisions of this Part, except for the provisions of regulations on *maximum overhang* and *overall height* of vehicle: Provided that:
  -no such vehicle shall be operated on the roadway of a public road during the period between sunset and sunrise and at any other time when, due to insufficient light or unfavourable weather conditions, persons and vehicles upon the public road would not be clearly discernible at a distance of 150 metres;
  -the overall width of any such vehicle shall not exceed 3.5 metres;
  -the driver of any such vehicle on the roadway of a public road shall stop such vehicle, and where possible, drive it off the roadway if it be necessary in order to allow other vehicular traffic to pass;
  -any such vehicle exceeding the maximum overall length or the maximum overall width shall display two flags of red cloth not less than 600 millimetres by 600 millimetres, in such manner as to indicate its abnormal length or width and such flags shall be suspended from the vehicle transversely to the direction of travel; and
  -no such vehicle, other than a mobile crane which is operated for the purpose of removing any hazard or obstruction on a freeway, shall be operated on a freeway.
- Any vehicle, including a tractor, which is not a goods vehicle and which is used solely for bona fide agricultural, and when operated on a public road, is exempt from the provisions of this Part, except for the provisions maximum overall height: Provided that:
  -no such vehicle shall be operated on the roadway of a public road during the period between sunset and sunrise and at any other time when, due to insufficient light or unfavourable weather conditions, persons and vehicles upon the public road would not be clearly discernible at a distance of 150 metres;
  -the driver of any such vehicle on the roadway of a public road shall stop such vehicle and, where possible, drive it off the roadway if it be necessary in order to allow other vehicular traffic to pass;

[15] *Guidelines on Harmonization of Vehicle Regulations Dimensions of Vehicles. SADC. 2009*

-where such vehicle: exceeds the maximum overall length, encroaches beyond half the width of the roadway except when overtaking other vehicles or crossing bridges; or is more than 2.6 metres but less than 3.5 metres wide, two flags of red cloth shall be displayed not less than 600 millimetres by 600 millimetres in such manner as to indicate its abnormal length or width and such flags shall be suspended from the vehicle transversely to the direction of travel, but the vehicle may be fitted with amber flashing lights in lieu of such flags; and

-the overall width of any such vehicle shall not exceed 4.5 metres, provided further that when the overall width exceeds 3.5 metres -such vehicle shall not normally encroach beyond half the width of the roadway except when overtaking other vehicles or crossing bridges; and if such vehicle does encroach beyond half the width of the roadway, two escort vehicles with the headlamps switched on and displaying red flags not less than 600 millimetres by 600 millimetres or amber flashing lights shall be provided, one travelling in front and one to the rear of such vehicle.

9.3.10.2 Kenya

Kenya Bureau of Standard develops regulations[16] for operating agricultural machinery in public roads. The code is comprehensive it specifies general safety and environmental operational requirements on public roads for agricultural tractors, trailers, trailed appliances and mounted equipments/implements. The Standard applies to all categories of agricultural tractors and their trailers as well as related agricultural machinery and it refers to the legal requirements as per the Traffic Act, Cap. 403 of the Laws of Kenya. However, the standard does not apply to industrial tractors, earthmoving tractors, purpose-built forestry tractors and fork lift trucks.

With respect to vehicle dimensions limits and vehicle combinations the standard is as follows:

- If the overall width of an agricultural vehicle, trailer, implement or appliance exceed 2.65 m, the agricultural vehicle owner shall apply for exemption from the Highway Authority stating the time, date and route of the journey and information on the weight, height and width of the vehicle.
- For vehicles, trailers, appliances, implements or combinations exceeding 2.65 m wide and up to 3.5m wide the speed limit should be kept to 32 km/h.
- If the overall width of an agricultural motor vehicle, an agricultural trailed appliance, mounted implement or combination exceeds 2.65 m, at least one person other than the driver of the drawing vehicle shall be employed or an escort van at the front to warn other road users of any danger.
- If the width of a vehicle, appliance, implement or trailer exceeds 3.5m then a speed limit of 20 km/h is recommended. Exemption shall be sought from the ministry of transport and the vehicle shall be accompanied by a second person/vehicle at the front (not carried on the vehicle) to warn other road users.
- Agricultural tractors, trailed appliances or drawbar trailers shall not individually exceed 11m in length including drawbar. However, a weight transfer tractor trailer combination shall be classified as an articulated vehicle and shall be subject to the length rules applicable to the Traffic Act for such a vehicle which shall be 18 m
- The limit of the overhang from an agricultural motor vehicle shall not exceed 60% of the wheel base.

[16] Code of Practice for Operation of Agricultural Tractors and Trailers on Public Roads. Public Review Draft, First Edition. September 2010

- The extremities of the vehicle or implement (including any blade or spike) should be made clearly visible at a reasonable distance to any person on the road even in reduced visibility.
- Mounted non-articulated and semi-mounted implements (for example ploughs, sprayers and haymaking equipment), which cannot be classified as agricultural trailed appliances, also fall within the scope of this code of practice and the end of each projection shall be made clearly visible at a reasonable distance to any person.
- An agricultural tractor shall tow not more than one trailer or other towed vehicle, unless there is an effective braking system installed.
- The overall maximum length of an agricultural trailer including its drawbar shall not exceed 11 m irrespective of the number of axles.
- Where an agricultural tractor is towing a trailer or an appliance, the overall length of the combination shall not exceed 18 m. However, it is recommended that tandem trailers up to a total length of 25 m may be allowed for agricultural purposes when drawn by an agricultural tractor not capable of speeds exceeding 40 K/hr.

9.3.10.3 Burundi

The provision for Burundi is brief and lack details compared to those of Tanzania, and Kenya. It states that[17]:

- Agricultural vehicles of up to 3 m width moving to/from the farm are required to travel at a speed not more than 20 km per hour.
- The external components and attachments which are easily detachable should be folded or removed to reduce the width while driving on public roads.
- The exemption is also applicable to the mobile equipment used by contractors which move between the yard and the construction site at a speed not exceeding 20 km per hour.
- The following vehicles or combination of vehicles are also exempted provided that they do not travel at a speed more than 20 km per hour: vehicle trains used to transport of among others tree trunks; trains of vehicles used by businesses and moving either between the garage and the yard, or from a yard to another, provided that such sites are not within a distance of more than 10 km, in which case a special permit must be requested.

9.3.11 Recommendations from PADECO Study on Combination of Vehicles

The Study conducted by PADECO[18] offered recommendations to consider technological advances on vehicle combination types and include them in the current regulations in EAC countries, provided the combination do not exceed dimensional, load or manoeuvrability limits. The interlink vehicle combination which is widely used in Southern Africa, have two significant advantages over the equivalent 56-ton truck and trailer combination due to their two rotating connections being further apart, they are more stable on the road and therefore improve road safety. And secondly, they are flexible and convenient because they can be unhooked and pre-loaded, and one tractor can operate several semi-trailers and therefore fleet utilization can be improved, whereas, the drawing vehicle in a truck-trailer combination cannot be utilized in such a manner.

[17] Regulation of the Maximum Axle Load of Vehicles Circulating in The Territory of Burundi. Bujumbura, August 1993. Ministerial Order No.720/70. Minister Of Public Works And Equipment & Minister Of Transports, Posts And Communications (translated)

[18] PADECO (2011). Study for the Harmonization of Vehicle Overload Control in the East African Community. Draft Final Report

9.4 Vehicle Dimension Limits from Other Countries in the World

Table 12 shows maximum vehicle main dimensions from selected countries in the world. It can be seen that the maximum limits for EAC countries are within the range of most other countries in the world.

Table 12: Vehicles weight and dimension limits for selected countries in the world

	Permissible single axle weight *[tonnes]*[19]	***Permissible gross weight tractor + semi-trailer*** *[tonnes]*	***Maximum vehicle dimensions*** *[meters]*		
			length	*Height*	*width*
Australia	9.0	45.5 (125.2)	19.0 (53.5)	4.3	2.5
Brazil[20]	10.0	45.0 (74.0)	22.40 (30.00)	4.40	2.6
Canada	9.1	62.5	25.0 (38.1)	4.12	2.6
China	10.0	40.0	18.0	4.2	2.5
Germany[21]	10.0	44.0	18.75	4.0	2.6
India	10.2	44.0	18.0	4.2	2.7
Japan	10.0	36.0	18.0	3.8	2.5
Russia	10.0	44.0	20.0	4.0	2.55
Sweden	10.0	44.0 (60.0)	18.75 (25.25)	4.0	2.55
Thailand	9.1	37.4	10.0	4.0	2.5
UK	10.0	44.0	18.75	4.0	2.55
USA[22]	9.1	36.3 (59.45)	19.8 (35.20)	4.1	2.6

9.5 Areas of Commonality and Differences

From the reviewed standards and regulations in EAC Partner States it was evident that some standards and regulations are not available in some countries. Hence it is recommended that the countries should develop the missing standards and regulations with consideration for harmonizing with other countries in EAC and to the fact that, COMESA, EAC and SADC have signed an agreement on harmonization of vehicle dimensions. Table 13 shows the status on vehicle dimensions and combinations between EAC Partner States and potential areas to be harmonized.

Table 13: Status of standards and regulations and potential for harmonization

Standard/ Provision	Status and Comments	Potential for Harmonization and/or Comments
Maximum overall length of vehicles	Rigid vehicle: Range 11.0 – 12.5. Can be considered to be within agreement range. Trailer: There is mixed categorization. Some countries categorize trailers by GVM, others by number of axles, and still others give dimensional limit without any categorization. Articulate vehicle: Range 17.0 – 18.5. Can be considered within agreement range Combination of vehicles: 18.0 – 22.0. Can be considered to be within agreement range	• Yes for -Rigid vehicle -Articulate vehicle -Combination of vehicles • Categorization of trailers need to be harmonized first
Maximum overall width of vehicles	Range 2.5 – 2.65. Can be considered to be within agreement range	Yes
Maximum overall height of vehicles	Range 4.0 – 4.6. Can be considered to be within agreement range Double Deck bus: Only regulated in Tanzania	• Double deck bus not to be considered in this exercise, and can be regulated within each country • Yes –for other vehicles
Maximum wheelbase	Standards from Kenya and Tanzania are within agreement range.	Yes
Front and rear overhangs	• Standards from the four countries suggest that the regulation is within agreement range.	• Yes
Projecting load limits	• Range 1.25 – 3.5. for front/rear projection • Regulation for Rwanda seems way too high compared to other countries. • 0.15 for side projection	• Yes for front and rear projection: Rwanda to reduce the limit • Yes, for side projection
Warning requirement for projecting loads	Provision by SADC which Tanzania have adopted seem to adequately cover the safety requirements for projecting loads	• Yes
Restrictions on vehicle combinations	• All countries prohibit a combination of more than three vehicles.	• Need to accommodate new technologies which are safer but do not comply with the current regulations in all EAC Partner States.
Vehicles exempted from the provisions	• The SADC regulation which Tanzania have adopted seem to adequately cover the safety requirements for projecting loads • Regulations from Kenya and Burundi impose speed limits to vehicles exempted from the provisions	• Yes

9.6 Recommendations for EAC Standards and Regulations

9.6.1 Vehicle Physical Dimensions

Table 14 shows recommended dimension limits for harmonization by EAC Partner States.

Table 14: Existing and Recommended Vehicle Dimensions Standards for Harmonization. (All dimensions in Meters)

Standard/provision	Existing Standards	Recommended Harmonized Dimensions
Maximum overall length of vehicles (m)	Rigid vehicle Tanzania (12.5), Kenya (11.0), Uganda (12.5), Rwanda (11.0), Burundi (12.0).	12.5
	Articulate vehicle Tanzania (18.5), Kenya (17.4), Uganda (17.0), Rwanda (17.4), Burundi (17.4).	18.5
	Combination vehicles Tanzania (22.0), Kenya (22.0), Uganda (22.0), Rwanda (18.0), Burundi (18.0).	22.0
Maximum overall width of vehicles (m)	Bus (wheel track >1.9 m) and Goods vehicle Tanzania (2.6) Refrigerated truck Burundi (2.65) Any other vehicle Tanzania (2.6), Kenya (2.65), Uganda (2.5), Rwanda (2.65), Burundi (2.5),	2.65 2.5
Maximum overall height of vehicles	Any vehicle Tanzania (4.6), Kenya (4.2), Uganda (4.0), Rwanda (4.2), Burundi (4.2).	4.6
Minimum Turning Radius	Any other vehicle Tanzania (13.1), Kenya (12.5)	13.1
Maximum Wheel Base	Semi-trailer Tanzania (10.0),	10
	Any Other Vehicle Tanzania (8.5),	8.5
Maximum Front Overhang	Semi-Trailer Tanzania (1.8)	1.8
	Any other Vehicle Tanzania [1]a Kenya (= 60% of WB); Rwanda (smaller of 2.7m or 55% of WB) Where: WB = Wheel base	[1]a
Maximum Rear Overhang	Any Other Vehicle Tanzania (60% of WB) Kenya (60% of WB) Rwanda (smaller of 3.5 m or 0.65 x WB)	60% of WB
	[2]Trailer Tanzania (50% of trailer body)	50% of trailer body
Projecting load limits (front and rear)	Bus and goods vehicle Tanzania (1.3)	

	Any other vehicle Tanzania (1.25), Kenya (1.8), Uganda (1.25), Rwanda (2.7F, 3.5R), Burundi (-)	1.25
Projecting load limits (sides)	Tanzania (0.15), Kenya (0.15), Uganda (0.15), Rwanda (-), Burundi (-)	0.15

[1]*For vehicles where the distance from the front end of the vehicle to the backrest of the driver's seat at rearmost position is less than 1.7 m:*
a = smaller of 60% of WB or [6.2 – (0.5xWB)];
For Driver's seat position more than 1.7 m:
a = smaller of 60% of WB or [5.8 – (0.5xWB)];

9.6.2 Warning Requirements for Projecting Loads

All EAC countries have a regulation on the warning required for protruding loads carried by a vehicle to warn other road users. SADC developed a regulation on the same for its Partner States to adopt. The SADC regulation is detailed, easier to apply and is much safer compared to the regulations which are used in EAC countries.

Recommendation on projecting loads
The SADC provision on Warning Requirements for Projecting Loads on the vehicle is much safer for other road users and is much clear and has technical details which are easier for the drivers to apply. Hence it is recommended that this provision to be adopted as EAC standard.

9.6.3 Restrictions on Vehicles Combinations

SADC have developed regulations to be used in the Partner States, and the regulations are also in the ongoing process to be harmonized in the entire EAC-SADC-COMESA region. Hence it is recommended that in the interim period the SADC regulation for combination of vehicle be adopted as EAC standard.

Recommendation on vehicle combinations

- The SADC regulation on restrictions on vehicle combinations to be adopted as EAC regulation.
- Longer Combination Vehicle (LCV) which exceeds the current *maximum length of combination of vehicles* should be evaluated against proposed geometric standards and adopted if viable.

9.6.4 Vehicles Exempted from the Provisions

Agricultural and construction machinery are exempted from provisions on vehicle dimensions and combinations in most EAC countries.

Recommendation for agricultural and other exempted vehicles

- The SADC regulation on vehicles which are exempted from the provisions on dimension limits and restrictions on vehicle combinations to be adopted as EAC regulation.
- Speed limits to be included in the regulations to increase safe operation of agricultural and construction machinery when using public roads.

9.7 Impacts of Harmonisation of Vehicle Dimensions and Combinations in EAC Member States

The standard which needs immediate attention is the maximum overall height of the vehicle. The recommended EAC standard for maximum height is 4.3 m. Hence with exception of Tanzania, other EAC countries whose standard is lower than 4.3 m have to check and take measures to provide the required minimum vertical clearances of overhead structures such as bridges, pedestrian overpasses, and utility lines clearance of 4.3 m plus recommended allowances along the regional trunk road networks in their countries.

The increase of transportation of high-cube ISO shipping containers in the region has posed a problem that the maximum height shoots to 4.5 m when the high-cube container is loaded to a standard semi-trailer. Tanzania and some SADC countries have changed the standard for maximum height to 4.6 m to accommodate standard semi-trailers loaded with high-cube ISO containers. South Africa and the Task Force panel of experts for the tripartite (COMESA-EAC-SADC) is studying the effect of raising maximum height on vehicle stability before considering raising the standard height. Hence, the exercise of checking the overhead structures in regional truck roads in EAC countries should also consider future integration of the tripartite regions should the standard be raised to 4.6 m.

The implications for the vehicle dimensions affecting the horizontal alignment of roadways are outlined under the section dealing with geometric design standards.

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FINAL REPORT

ANNEX A 10

HARMONISATION OF TRANSPORTATION OF ABNORMAL LOADS AND DANGEROUS GOODS

TABLE OF CONTENTS

LIST OF TABLES

GLOSSARY AND ACRONYMS

AU	African Union
BICO	Bureau for Industrial Cooperation
COMESA	Common Market for Eastern and Southern Africa
EAC	East African Community
GVM	Gross Vehicle Mass
LEFs	Load Equivalent Factors
RECs	Regional Economic Communities
RUF	Road Usage Factor
SADC	Southern African Development Community
SATCC	Southern Africa Transport and Communications Commission
UN	United Nations

EXECUTIVE SUMMARY

The five EAC partner states, Kenya, Uganda, Tanzania, Rwanda and Burundi had agreed to standardize road transport laws and regulations in the EAC in order to improve the efficiency and safety of domestic, transit and cross-border road traffic. This report presents an overview of the current situation of recommend a harmonization process on the transportation of abnormal loads and dangerous goods in the EAC region.

Review of EAC countries policies, regulations and procedures for the conveyance of abnormal loads and dangerous goods showed variations across the EAC member states. The classification of such goods in terms of type, hazardousness and abnormality also differ. During the course of this task we became aware of the SADC and COMESA harmonization processes in road transport infrastructure standards and institution arrangements in a number of areas including conveyance of abnormal and dangerous goods which started in 2010. Since all EAC countries have multiple memberships within the three RECs (Tanzania being a member of SADC and Kenya, Uganda, Rwanda, and Burundi being members of COMESA), it became obvious that this study adopts the draft tripartite Task Force study to smoothen and speed up the harmonisation process in the three RECs.

With the objective of speeding harmonization process in the three RECs and the fact that SADC and later COMESA/EAC/SADC Task force have already put substantial efforts to develop the same regulations we recommend that this study adopt the COMESA/EAC/SADC Draft Proposals on:

- Standards for the Conveyance of Abnormal Loads, and
- Administrative Guidelines for Granting of Exemption Permits for the Conveyance of Abnormal Loads
- COMESA/EAC/SADC Dangerous Goods Legislation

Dangerous chemicals and products are traded throughout the World by land, sea, and air transportation systems. It is important that classification, general packing requirements, marking, labelling, etc should not only comply on the regulations within a certain region by land transport, but globally and by all modes of transportation. The UN Recommendations on the Transport of Dangerous Goods are internationally recognized standard guidelines and were prepared to ease trade of chemicals throughout the world in a safe and efficient manner. Many countries and economic communities around the World have prepared their National regulations by using UN Recommendations.

We therefore recommend using UN Recommendation Guidelines for harmonisation on transportation of dangerous goods in EAC countries. The tripartite Task Force has also recommended using UN Recommendations for the three RECs. The Task Force are preparing the administrative guidelines for the transportation of dangerous goods, which we have also recommended to adopt.

10 HARMONISATION OF TRANSPORTATION OF ABNORMAL LOADS AND DANGEROUS GOODS

10.1 Introduction

The objectives of this component of the study were to review and subsequently propose a harmonized regime on the transportation of abnormal loads and dangerous goods in the EAC region. To complete this task the following tasks were required as per ToR:

- Review of existing documents/statutes and proposals for improvements to the same,
- Identification of areas of commonality which lend themselves to harmonization
- Propose and implement the incorporation of areas unique to particular countries into the harmonized regimes
- An indication of the impact of harmonization
- Conduct of a stakeholder workshops to gain consensus on the harmonization of different regulations and standards

In order to adequately address the scope of work, the methodology adopted for the project entailed the following activities:

- Visit EAC Partner States for the purpose of collecting documents from each State and to make initial contacts with the responsible officials. The visits involved one member of the Consultant's team visiting the contact person in the respective Partner State to identify and collect documents relevant to all thematic areas.
- Preparation of an Inception Report and submission of the same to the EAC Secretariat.
- Detailed documents review, situational analysis, and preparation of Working Papers.
- Preparation and presentation of the Working Papers and Draft Final Report at the EAC Task Force meetings which were held in Dar es Salaam and Mwanza to review the reports.
- Preparation of the Final Report.

10.2 Current Situation

10.2.1 EAC Countries

Review of EAC countries policies and procedures for the conveyance of abnormal loads and dangerous goods showed that there are no common and detailed procedures across the EAC member states. The classification of such goods in terms of type, hazardousness and abnormality differ. The common approach taken in the conveyance of these goods involves reporting to mandated authorities, which differ from country to country, for the abnormal load to be assessed for transporter to be charged, and markings and escorting requirements. Table 1 outlines the legal framework where abnormal loads and dangerous goods are administered.

Table 1: The Legal Framework in the EAC Member States

COUNTRY	LAW/ACT	REGULATION	REMARKS/ RECOMMENDATIONS
Tanzania	The Transport Licensing Act, 1973	Transport licensing for both passengers and cargo	Covers all types of cargoes
	Atomic Energy Act no. 7 (2003)	Safe Transportation of Radioactive Ores	Covers Radioactive materials

Kenya	Traffic Act (Cap 403)	Traffic and transportation licensing	Cargoes not clearly defined
Uganda	Traffic and Road Safety Act, 1998 (Cap 361)	Traffic and transportation licensing	Cargoes not clearly defined
Rwanda	Law no. 39/2001 of 13th Sept. 2001, Rwanda Regulatory Agency, Article 8. Carriage of Dangerous Goods.	Transport of cargo by road	

10.2.2 SADC and COMESA

In the Meeting at a Tripartite Summit on 22 October 2008 in Kampala, Uganda[1], leaders of Member States of SADC, the Common Market for Eastern and Southern Africa (COMESA) and the East African Community (EAC) agreed on important milestone towards continental integration as envisaged by the African Union (AU). According to the final communiqué, the historic Tripartite Summit agreed on a programme of harmonization of inter-regional infrastructure programmes as well as institutional arrangements on the basis of which the three RECs would foster cooperation. One of the components of joint implementation of inter-regional infrastructure program as well as institutional arrangements is on the conveyance of abnormal and dangerous loads within the three RECs.

The Summit resolved to immediately start working towards merging the three trading blocs into a single REC with the objective of fast-tracking the attainment of the African Economic Community. In that regard, the Summit directed the Tripartite Task Force composed of the Secretariats of the three RECs to develop a roadmap for the implementation. The agreed roadmap included an initial assessment of current regime of bilateral transport agreements to be followed by development of Harmonisation Proposals.

During the course of this task we became aware of the SADC and COMESA harmonization processes in road transport infrastructure standards and institution arrangements in a number of areas including conveyance of abnormal and dangerous goods which started in 2010. Since all EAC countries have multiple memberships within the three RECs (Tanzania being a member of SADC and Kenya, Uganda, Rwanda, and Burundi being members of COMESA), it became obvious that this study adopts the draft tripartite Task Force study to smoothen and speed up the harmonisation process in the three RECs.

The first Working Group and Experts meeting took place in Gaborone, Botswana in October 2010. In this meeting the Working Group discussed on the proposals for the Guidelines on Harmonization of Vehicle Regulations and Standards in respect of Fitness of Vehicles (the Testing of Vehicles for Roadworthiness and Evaluation of Vehicle Test Stations); and Transportation of Abnormal and Dangerous Goods

The second meeting of the Working Group which included experts from COMESA and SADC countries was held in Kampala, Uganda on 4-5 July 2011. The same representatives also met in Kigali, Rwanda on 29-31 May 2012 to discuss on the draft standards and guidelines for conveyance of abnormal loads and dangerous goods, among other things. In view of harmonising the EAC and the tripartite proposals, BICO was invited to attend Kampala and Kigali meetings.

[1] Final Communique of the COMESA-EAC-SADC Tripartite Summit of Heads of State And Government. 22nd October 2008. Kampala, Uganda

From the draft standards, regulations and guidelines we noted that substantial progress has been made by the tripartite Task Force to come up with proposals for harmonized standards and regulations for the transportation of abnormal loads and dangerous goods. The draft guidelines[2] for transportation of abnormal loads were initially adopted from South Africa by SADC and are now used as draft guidelines by the tripartite Task Force for member states to comment and contribute in order to get harmonized guidelines throughout the three RECs. On transportation of Dangerous goods the tripartite Task Force have adopted the use of UN model Recommendations on Transportation of Dangerous Goods in the three RECs. The Task Force is currently preparing the administrative guidelines for the transportation of dangerous goods in the tripartite region.

In the Kigali meeting the Working Group recommended to engage a Consultant to develop a single regional permit (multi-country) with a clear institutional framework and approach for the purpose of issuing Abnormal Load permits. The study to develop single permit should include formula and methods of sharing of permit fees (between countries), route inspections and escorts. The study should map current country based institutional frameworks, formulae and methods used by COMESA/EAC/SADC member states in managing Abnormal Loads Permits. The study should also develop a harmonised methodology for determining Load Equivalency Factors (LEFs) for determining mass fees for Abnormal Loads.

10.2.3 UN Recommendations on Transportation of Dangerous Goods

United Nations (UN) recognized that with different regulations in every country and for different modes of transport, international trade in chemicals and dangerous products would be seriously impeded, if not made impossible and unsafe. The UN Sub-Committee of Experts on the Transport of Dangerous Goods (UNSCETDG) is responsible for the ***UN Recommendations on the Transport of Dangerous Goods - Model Regulation***[3], which are internationally accepted as the principal technical standards underpinning the air and sea dangerous goods codes, and are also used by many countries as the basis for their road and rail dangerous goods transport codes.

Moreover, dangerous goods are also subject to other kinds of regulations, e.g. work safety regulations, consumer protection regulations, storage regulations, environment protection regulations. The model regulations cover the transport of dangerous goods by all modes of transport except by bulk tanker. They are not obligatory or legally binding on individual countries, but have gained a wide degree of international acceptance. Over and above these regulations, countries can apply additional provisions in their territory in case of special risks.

Dangerous goods may be pure chemical substance or manufactured articles (eg, ammunition, fireworks). The transport hazards that they pose are grouped into nine classes, which may be subdivided into divisions and/or packing groups. The most common dangerous goods are assigned a UN number, a four digit code which identifies it internationally: less common substances are transported under generic codes such as "UN1993: flammable liquid, not otherwise specified". So every country in the world uses the same number for that specific consignment of dangerous goods. These can be found in dangerous goods publications volumes for road and rail and the IATA for movement by air.

UNSCETDG prepared a "Model Regulations on the Transport of Dangerous Goods", which aim at presenting a basic scheme of provisions that will allow uniform development of national and

[2] COMESA/EAC/SADC STANDARD. The Conveyance of Abnormal Loads. Updated to include amendments agreed to at the Working Group of the Panel of Experts meeting, held on 28 – 29 October 2010.
[3] http://www.unece.org

international regulations governing the various modes of transport; yet they remain flexible enough to accommodate any special requirements that might have to be met. UN expects that governments, intergovernmental organizations and other international organizations, when revising or developing regulations for which they are responsible, will conform to the principles laid down in these Model Regulations, thus contributing to worldwide harmonization in this field.

Furthermore, the new structure, format and content should be followed to the greatest extent possible in order to create a more user-friendly approach, to facilitate the work of enforcement bodies and to reduce the administrative burden. Although only a recommendation, the Model Regulations have been drafted in the mandatory sense in order to facilitate direct use of the Model Regulations as a basis for national and international transport regulations.

Amongst other aspects, the Model Regulations cover principles of classification and definition of classes, listing of the principal dangerous goods, general packing requirements, testing procedures, marking, labelling or placarding, and transport documents. There are, in addition, special requirements related to particular classes of goods. With this system of classification, listing, packing, marking, labelling, placarding and documentation in general use, carriers, consignors and inspecting authorities will benefit from simplified transport, handling and control and from a reduction in time-consuming formalities. In general, their task will be facilitated and obstacles to the international transport of such goods reduced accordingly.

10.3 Recommendations

10.3.1 Adoption of COMESA/EAC/SADC Tripartite Task Force Draft Proposal on Conveyance of Abnormal Loads and Dangerous Goods

With the objective of speeding harmonization process in the three RECs and the fact that SADC and later COMESA/EAC/SADC Task force have already put substantial efforts to develop the same regulations we recommend that this study adopt the COMESA/EAC/SADC Draft Proposals on:

- Standards for the Conveyance of Abnormal Loads, and
- Administrative Guidelines for Granting of Exemption Permits for the Conveyance of Abnormal Loads
- COMESA/EAC/SADC Dangerous Goods Legislation

We recommend that harmonisation of conveyance of abnormal loads and dangerous goods in EAC countries adopt the COMESA/EAC/SADC Draft Proposals on:

- Standards for the Conveyance of Abnormal Loads, and
- Administrative Guidelines for Granting of Exemption Permits for the Conveyance of Abnormal Loads
- COMESA/EAC/SADC Dangerous Goods Legislation

10.3.2 Adoption of UN Recommendation Guidelines on Transportation of Dangerous

Dangerous chemicals and products are traded throughout the World by land, sea, and air transportation systems. It is important that classification, general packing requirements, marking, labelling, etc should not only comply on the regulations within a certain region by land transport, but globally and by all modes of transportation. The UN Recommendations on the Transport of Dangerous Goods are internationally recognized standard guidelines and were prepared ease trade of chemicals throughout the world in a safe and efficient manner. Many countries and economic communities around the World have prepared their National regulations by using UN Recommendations.

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We recommend using UN Recommendation Guidelines for harmonisation on transportation of dangerous goods in EAC countries.

10.4 Overview of COMESA/EAC/SADC Draft Guidelines on Transportation of Abnormal Loads

This section highlights the COMESA/EAC/SADC proposed guidelines for transportation of abnormal loads. The full proposal is contained at the Appendix.

10.4.1 Guiding Principles

The fundamental principles guiding the process for transportation of abnormal loads are:

- An exemption permit for an abnormal load will only be considered for an indivisible load, abnormal in dimension and/or mass, where there is no possibility of transporting the load in a legal manner;
- The damage to the road infrastructure by an abnormal vehicle has to be recovered from the carrier;
- The risks to other road users must be reduced to a level equivalent to a situation without the presence of the abnormal vehicle on the road; and
- The conditions imposed must take into account the economic and/or social interest of the country and public at large.
- The purpose of the exemption permit system is not to undermine or circumvent SADC regulations.
- This document contains recommendations that are generally applicable, but the issuing authority can deviate from these recommendations and/or impose additional requirements when taking the circumstances applicable to each application into account.

10.4.2 Types of Abnormalities and Classifications

In this section types and classification of abnormality are described. A vehicle or a vehicle with its load that is considered to be indivisible can be abnormal either in terms of dimension or mass or both. The extents of exceeding dimension or mass from the legal limits are classified for indivisible loads.

10.4.3 Dimensional Limits

Exceeding dimensions on vehicles or loads and conditions which are allowed under permit are described in this section in terms of the following dimensions.

- Length
- Width
- Height
- Overhangs
- Load projections
- Wheelbase
- Turning radius
- Height to wheel track ratio

10.4.3.1 Length

Table 2 shows maximum length of different categories of vehicles which are allowed under permit.

Table 2: Maximum Overall Length per Vehicle Type (including load projections)

Vehicle Type	Overall Length (m)	Comments
Rigid vehicles	20	Including mobile cranes
Foundation diggers	23	
Articulated vehicles	26	Truck-tractor & semi-trailer
Combinations of vehicles	28	Truck, dolly & semi-trailer

For the transportation of long loads of up to 20.0 m in length:

- Vehicles of a conventional type without steerable rear axles, or vehicles incorporating non-steerable dollies or extendible trailers may be used.
- In the case of non-steerable axle units, the longitudinal distance between the extreme axle centres of any axle unit may not exceed 4.2 m.

For the transportation of loads from 20.0 m to 25.0 m in length or for wheelbases exceeding 14.5 m, steerable rear axles or steerable dollies must be used.

For loads longer than 25.0 m all rear axle units must be fully steerable. Alternatively, a steerable dolly (fully steerable axles in all conditions, both static and dynamic) with a turntable capable of 180 degree rotation may be used. With this type of vehicle, a rear projection of the load is not desirable and loads should be supported near the end.

10.4.3.2 Width

Allowed limits under permit depend on factors such as topography, road width, traffic volumes and obstructions. Special provision must be made in terms of markings and escorting if the vehicle width exceeds 3.5 m.

10.4.3.3 Height

The principal factors limiting the permissible height of abnormal loads are the clearances under any overhead bridges or overhead lines on the route, and the stability of the vehicle and the load. It is the responsibility of the carrier to identify a suitable route and to substantiate the suitability of the route with the application. Actions which are required to be done by the carrier and the authorities of power and telecommunication utilities are outlined for different height of loads or vehicles.

10.4.3.4 Overhangs for Load-Carrying Vehicles

The front overhang for load-carrying abnormal vehicles must comply with the requirements for maximum front overhang. However, for rear overhang it may not exceed 2 m or 70 per cent of the wheelbase, whichever is the greater.

10.4.3.5 Overhangs for Non Load-Carrying Vehicles

In the case of non-load carrying vehicles, such as mobile cranes and foundation diggers, the actual front or rear overhang shall not exceed the values given in Table 3.

Table 3: Front/Rear Overhang under Permit for Non-load Carrying Vehicles.

Wheelbase (m)	3 m	4 m	5 m	6 m or more
Allowable front or rear overhang (m)*	3.9	4.6	5.2	6.0

*From the centre of the front or rear axle to the furthest point of the overhang section of the vehicle.

The above-mentioned overhang values may be increased by 50% if the increased overhang section is at a height not less than 2.5 m above the road surface. In the case of special vehicles such as mobile cranes and drilling rigs, the front overhang of the furthest point of the central unit (the boom) must always be within the outer minimum turning circle of the vehicle.

10.4.3.6 Front Load Projections

No abnormal load shall project more than 1.0 m beyond the front end of the drawing vehicle.

10.4.3.7 Rear Load Projections

Regulation requires that no load shall project more than 1.8 m and that the combined length of the load and the vehicle shall not exceed the limits on overall vehicle length in that class of vehicle.

The following should be allowed under permit:

(i) For vehicles with a rear overhang of more than 50 per cent of the wheelbase: rear projection of the load may not exceed 0.5 m.

(ii) For vehicles with a rear overhang of less than 50 per cent of the wheelbase : Rear projection of the load measured (a) behind the centre of the rearmost axle unit, and (b) behind the rear end of the vehicle, may not exceed the values given in the Table 4.

Table 4: Allowable Rear Load Projections

Wheelbase (m)	Beyond centre of rearmost axle unit (a)	Beyond rear end of vehicle (b)
3	3.9	2.6
4	4.6	2.9
5	5.2	3.2
6	6.0	3.5
7	6.0	3.7
8	6.0	4.0
9	6.0	4.3
10 and more	6.0	4.5

Where the height of the load above the road surface is not less than 2.5 m, the projection may be increased by 20 per cent or 750 mm whichever is the lesser. A rear own escort should be provided where the projection exceeds 1.8 m.

10.4.3.8 Wheelbase

No wheelbase shall exceed 14.5 m unless steerable rear axles or steerable dollies are used.

10.4.3.9 Turning Radius

The maximum turning radii of vehicles will be determined by the geometry of the routes along which these vehicles travel. Traffic officer escort(s) may be required in cases where both the width and turning radius of a vehicle are excessive.

10.4.3.10 Height/Wheel Track Ratio (Stability)

In the case of a loaded vehicle with a height/wheel track ratio greater than 2 or a load width/wheel track ratio greater than 1.8, or a load that is asymmetrically loaded, the carrier may be required to produce calculations performed by a professional engineer showing that the loaded vehicle is adequately stable under all conditions of road travel.

10.4.3.11 Ground Clearance

A vehicle or combination must have a ground clearance of not less than 150 mm under laden mass conditions. Trailers with less than 150 mm clearance must be able to lift to 150 mm when required to negotiate an undulation in the road surface.

10.4.4 Mass Limits

This part describes the permissible maximum vehicle or combination mass of a road vehicle or combination of road vehicles that is allowed to operate under permit on a public road. The factors that determine the limits are:

- The capacity of the vehicles as rated by the manufacturer;
- The load which may be carried by the tyres;
- The damaging effect on road pavements;
- The structural capacity of bridges and culverts;
- The power of the prime mover(s) to vehicle mass ratio;
- The load imposed on the driving axles; and
- The load imposed on the steering axles.

The description and calculations used to evaluate the limit of each of the above factor is described in details in the Appendix.

10.4.5 Categories of Routes

Road sections (width) are categorized and posted speed limits are recommended. This schedule is designed to assist route selection for abnormal loads of different width.

10.4.6 Marking and Escorting

10.4.6.1 Exceeding Length and Width

In this section the type of warning devices and escorting requirements for exceeding length and width, and when travelling during the night are illustrated. In each type of marking detail specification of location to be displayed on the vehicle, dimension of the mark, speed limit, etc, are given. The types of warning marks include:

- Flags
- “Abnormal vehicle” warning board
- Escort vehicle warning board
- Speed restriction board
- Amber flashing lights
- Marker lamps and retro-reflectors

For long and wide loads a Road Usage Factor (RUF), which is proportional to vehicle length and width, is calculated to determine the type of warning device required as shown in the 5.

Table 5: Warning Apparatus Requirements for Long and Wide Loads

RUF Limit	Warning Apparatus Required
RUF < 0,54	Flags
	Abnormal vehicle warning board
RUF >= 0,54	Flags
	Abnormal vehicle warning board
	Amber Flashing Lights
	Marker lamps and retro-reflectors

Furthermore, the class and number of escort required in terms of dimension class and route category are summarized in Table 6. Table 7 shows the escort requirement in terms of RUF for the case where the route category is not defined.

Table 6: Escort Requirements for Wide and Long Loads in terms of Dimension Class and Route Category

Dimension Class	Route Category			
	A	B	C	D
D1	No escort	No escort	No escort	No escort
D2	No escort	No escort	No escort	One own escort vehicle
D3	No escort	No escort	One own escort vehicle	Two own escort vehicles
D4	No escort	One own escort vehicle	Two own escort vehicles	Two traffic officer escort vehicles
D5	One own escort vehicle	Two own escort vehicles	Two traffic officer escort vehicles	Two traffic officer escort vehicles
D6	Two traffic officer escort vehicles	Two traffic officer escort vehicles	Two traffic officer escort vehicles	Two traffic officer escort vehicles

Table 7: Escort Requirements for Wide and Long Loads in terms of the RUF

RUF Range	Escort Requirements
RUF ≤ 0,54	No escort required.
0,54 < RUF ≤ 0,94	One own escort vehicle required.
0,94 < RUF ≤ 2,73	Two own escort vehicles required.
RUF > 2,73	Two traffic officer escort vehicles or one traffic officer escort vehicle and one own escort vehicle.

10.4.6.2 Other Exceeding Dimensions

Apart from length and width, marking and escorting requirements for other vehicle dimensions which when exceeded could increase traffic friction or interfere with road furniture and utility structures. These are:

- Height
- Overhangs
- Load projections.

REFERENCES

1 COMESA/EAC/SADC STANDARD. The Conveyance of Abnormal Loads, Updated to include amendments agreed to at the Working Group of the Panel of Experts meeting, held on 28 – 29 October 2010.

2 Final Communique of the COMESA-EAC-SADC Tripartite Summit of Heads of State And Government. 22nd October 2008. Kampala, Uganda

3 http://www.unece.org

4 Kenya: Traffic Act (CAP 403). 1993.

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6 Tanzania: The Road Traffic Act, 1973.

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8 Uganda: Traffic and Road Safety Act, 1998 (Cap 361).