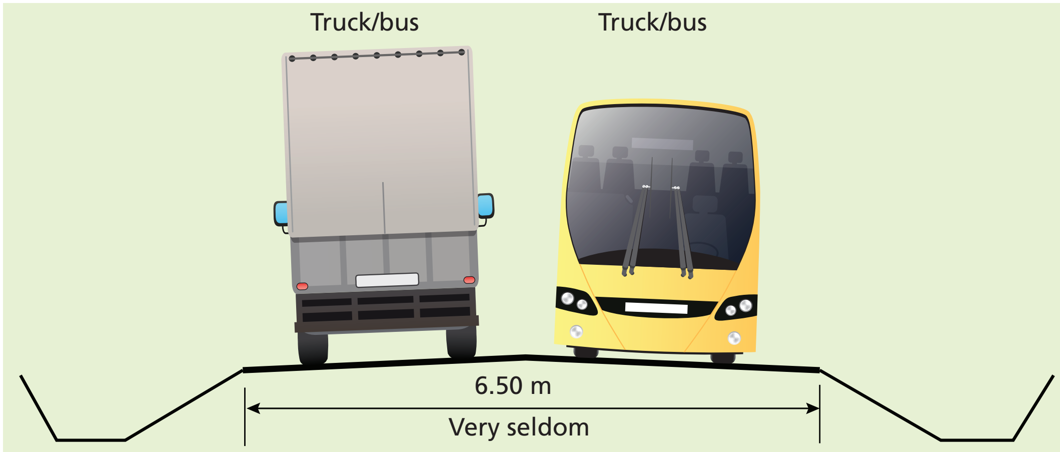

Figure 9-4: Rare traffic occurrence (D5 standard - schematic)

For the lower classes, DC6 to DC8, these potentially hazardous meeting situations becomes even less frequent and the roadway width can be reduced accordingly as shown in Table 9-1.

On sections with typical traffic as illustrated schematically above, additional shoulder width would not normally be required unless there are other compelling reasons to do so, be it for structural or safety reasons (for example, a large number of motor cycles and motor cycle powered vehicles).

Regulating travelling speed

The key to accommodate the various types of road users and maintain a satisfactory level of road safety is to ensure that travelling speeds are kept low. For the communities served by these roads, reliable all-weather access is far more valuable than the slight increase in actual travelling time compared to the theoretical travelling time had the alignment been fully engineered in accordance with the strict geometric design principles.

Research has shown that:

- In terms of geometry, drivers will choose lower speeds on roads that have rough surfaces, are narrow, winding or hilly and where the direction of the road and the lane boundaries are not well delineated. These factors affect drivers' perceptions of speed, and thus drivers will only reduce speeds when they are easily perceived (e.g. a concealed curve will not reduce speed in advance).
- Roadside environment and objects next to the road can also affect speed. Multiple objects next to the road can increase peripheral visual flow and therefore increase perceived speed, which will lead to reduced actual speed.
- Drivers will also slow down if they feel they are too close to objects on the side of the road and they feel they are unable to move away.
- Drivers choose lower speeds on roads with multiple access points to prepare for possible entry of other vehicles and in visual complex environments in order to process the higher levels of visual information.

The recommended LVR design approach of letting the alignment dictate the speed and not vice versa, is commensurate with these research findings. However, one cannot rely upon drivers' perceptions of speed and to expect them to automatically reduce their speed to acceptable levels. The design approach must therefore be coupled with:

- Installation of traffic calming measures where required, particularly in areas with high incidence of non-motorised traffic (NMT), e.g. speed humps, rumble strips, warning and speed limit signs etc.
- Fully engineered solutions on potentially hazardous spots that can be achieved within reasonable costs (e.g. road widening/lane separation over sharp crests, alignment improvement to straighten out blind curves).
- Adequate advance warning to drivers and speed reducing measures where potentially hazardous situations cannot be avoided without incurring prohibitive costs.
- Varying road carriageway width dictated by the amount and mix of traffic and terrain.

Modifications for high proportion of heavy vehicles

Assessing traffic volume and axle loading is discussed in *Chapter 8 – Traffic.* The AADT of motorised vehicles with two or more axles provides the basic means of defining the different geometric road standards but these are modified based on the proportion of heavy vehicles in the traffic stream. For a DC5 road this means that if there are more than 80 heavy vehicles per day, then the road should be designed as a high volume DC4 road in accordance with the specifications in the Road Geometric Desgn Manual (MOW, 2011).

Modifications for high numbers of non-motorised vehicles and motorcycles

Modification of the basic geometric standards may also be required, mainly for safety reason, in areas with a high incidence of motor cycles and motor cycle powered vehicles, non-motorised vehicles including bicycles, and pedestrians. These are assessed in terms of the effective road space that they occupy (dependant on size and speed) measured in terms of PCUs according to Table 9-3. Table 9-4 shows the proposed adjustments of the basic standards.

Table 9-3: PCU values

Vehicle	PCU value	Vehicle	PCU value
Pedestrian	0.15	Bajaj (sometimes called a motorised rickshaw).	0.40
Bicycle	0.20	Motor cycle with trailer.	0.45
Motor cycle	0.25	Small animal-drawn cart.	0.70
Bicycle with trailer	0.35	Large animal drawn cart.	2.00
All based on a passenger car = 1.0			

Note: These values are for LVRs where congestion of motorised vehicles does not occur hence they differ from those used for congestion effects.

Table 9-4: Proposed adjustments for non-motorised and motorcycle PCUs (>300 AADT[(1)])

Standard	AADT	Surface	Proposed modification.
DC 5	200 – 400	Paved	Shoulder width increased to 2.0 m each side.
		Unpaved	Increase carriageway width by 2.0 m.
DC 6	50 – 200	Paved	Shoulder width increased to 2.0 m each side.
		Unpaved	Increase carriageway width by 1.5 m.
DC 7	20 – 50	Paved	None (paved sections will be short).
		Unpaved	Increase carriageway width by 1.25 m.
DC 8	<20	Unpaved	Not required

Note 1: This value may need be modified when further data on road safety becomes available.

The proposed adjustment of shoulder or carriageway width is just one of many possible ways of accommodating mixed road users. Reference is also made to *Chapter 10 – Road Safety* for other solutions. There is no standard way of addressing this issue, and the designer must seek the best design in each case based on the particular circumstances and available budget.

(c) Terrain/Topography

Terrain has a major influence on geometric standards both because of the much higher costs that would be incurred if the same standards were used in hilly and mountainous terrain and because of the additional safety aspects required for roads in such terrain.

The purpose of terrain classification is to help minimise costs by adjusting the geometric standards when large earthworks are required. For a new road the terrain classification is used to give an initial indication of likely costs and should therefore be independent of any road alignment. However, once an alignment has been selected (or already exists) then the terrain class should reflect the conditions along that alignment; even in mountainous terrain a road in a wide river valley may not require expensive earthworks and should be designed based on a flat or rolling terrain class.

There are two methods of defining the terrain class. In the first method the terrain class is determined by the number of 5-metre contours crossed by a straight line connecting the two ends of the road section in question according to the following definitions in Table 9-5.

Table 9-5: Terrain Classes

Classification	Definition
Flat	0 to 10 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally below 3%.
Rolling	11 to 25 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally between 3 and 25%.
Mountainous	26 to 50 five-metre contours per km. The natural ground slopes perpendicular to the ground contours are generally above 25%.
Escarpment	Escarpments are geological features that require special geometric standards because of the engineering risks involved. Typical gradients are greater than those encountered in mountainous terrain.

Source. https://Comparativegeometrics.wordpress.com/2013/.../terrain-classification

In the second method the terrain is classified according to the side slope rather than the gradient as shown in Table 9-6. This is slightly more difficult to calculate from topographical maps but not excessively so and a high degree of accuracy is not essential. The task will always require a degree of engineering judgement.

Table 9-6: Alternative definition of Terrain Classes

Terrain	Transverse Slope
Flat	0% – 10%
Rolling	10% - 25%
Hilly	25% - 60%
Mountainous	Above 60%

Source. https://Comparativegeometrics.wordpress.com/2013/.../terrain-classification

The basic geometric standards are normally applicable to flat and rolling terrain. In difficult hilly or mountainous terrain, the design must be modified to one that can be fitted into the terrain at least cost and still provide a satisfactory service level. This will often result in narrower roads and varying road width. In such cases, alternative designs with associated costs should be worked out during the feasibility study and agreement reached with the Client before the detailed design is carried out.

(d) Nature of Roadside Population

When a road passes through a village, small town or other densely populated or market area, extra width should be provided in each direction for parking and for passenger pick-up. The additional width may be as much as one lane width in each direction, but will depend on the total traffic and PCU number in the area. A pedestrian footpath may also be also specified (refer to Road Geometric Design Manual (MOW, 2011) for cross-section details).

In such areas the main running surface should, as a general rule, be paved to minimise nuisance and pollution from dust. Non-bituminous surfacings, e.g. cobble stones, tend to be rougher than bituminous surfacings and will contribute to lower the speed on the main carriageway and will often be the better choice for parking and trading areas.

(e) Road Surface Type

The friction factors for paved and unpaved roads are significantly different and this affects the distance required to stop safely and the safe speed for negotiating curves. This must be taken into account in the geometric design.

(f) Land Use and Physical Features

Land use influences the design of the drainage features of a road and access to the road. Dealing with water run-off and potential erosion, for example, forms an important aspect of design, much of which is geometric in nature. These aspects are dealt with in *Chapter 11 – Hydrology and Drainage Structure.*

(g) Construction Technology

In a labour-abundant economy it is usually beneficial to maximise the use of labour rather than rely predominantly on equipment-based methods of road construction. In such a situation the choice of technology affects the standards that can be achieved, especially in hilly and mountainous areas. This is because:

- maximum cuts and fills need to be small;
- economic haul distances are limited to those achievable using wheel-barrows;
- mass balancing is achieved by transverse rather than longitudinal earth movements;
- maximum gradients follow the natural terrain gradients;
- horizontal alignments may be less direct.

The standards in hilly and mountainous terrain are always lower than in flat terrain but this reduction in standards need not necessarily be greater where labour-based methods are used. Following the contour lines more closely will make the road longer but the gradients can be less severe. Every effort should be made to preserve the same standards in the particular terrain encountered irrespective of construction method, but an economic evaluation using life-cycle costing may strongly favour one method over the other.

(h) Economic and Financial Considerations

Rehabilitation and upgrading of LVRs will sometimes involve widening of the existing roadway to the appropriate width for the projected traffic volume as well as strengthening the pavement to accommodate the design traffic loading. The widening can often be problematic and expensive and may involve expropriation of properties and demolition of houses within the road reserve. In difficult terrain, widening can also involve blasting and may cause destabilisation of steep cut slopes with associated high costs.

The extra width to provide, for example, 1.0 m shoulders on either side of a DC5 road as prescribed by the Road Geometric Design Manual (MOW, 2011), may incur prohibitive costs compared to providing a sealed road width of 6.50 m.

Widening requirements must be assessed on a case-by-case basis and would normally only be required around or within built-up areas with high PCU values.

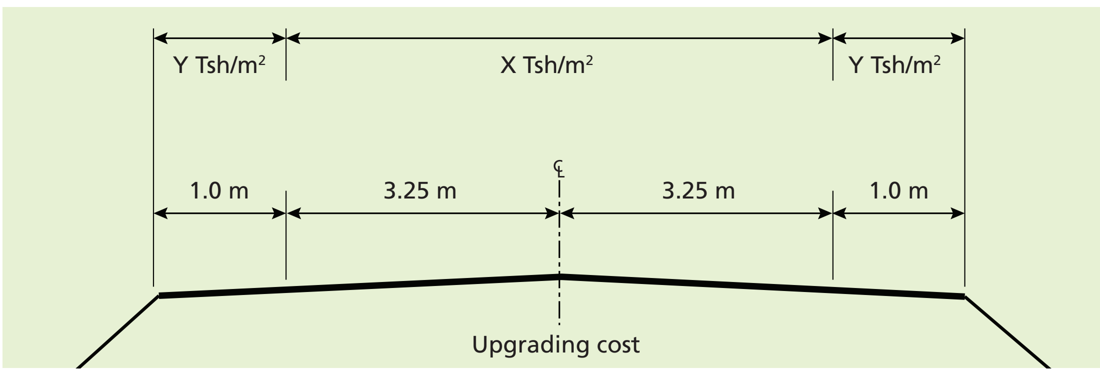

Figure 9-5: Upgrading costs

The unit cost Y Tsh/m^2 for providing extra width for shoulders can be substantially higher than the unit cost X Tsh/m2 for providing the upgraded carriageway.

Ideally a full economic appraisal should be carried out for projects of any considerable size. This may however, not be feasible or required for relatively small LVR upgrading projects and, in the final instance, the available budget will limit what can be achieved.

9.3.3 Procedure for Selection of a Geometric Standard

The procedure for selecting the appropriate design standard is shown in the flow chart in Figure 9-6. This procedure should be applied for the Option B approach to design, as discussed in Section 9.2.

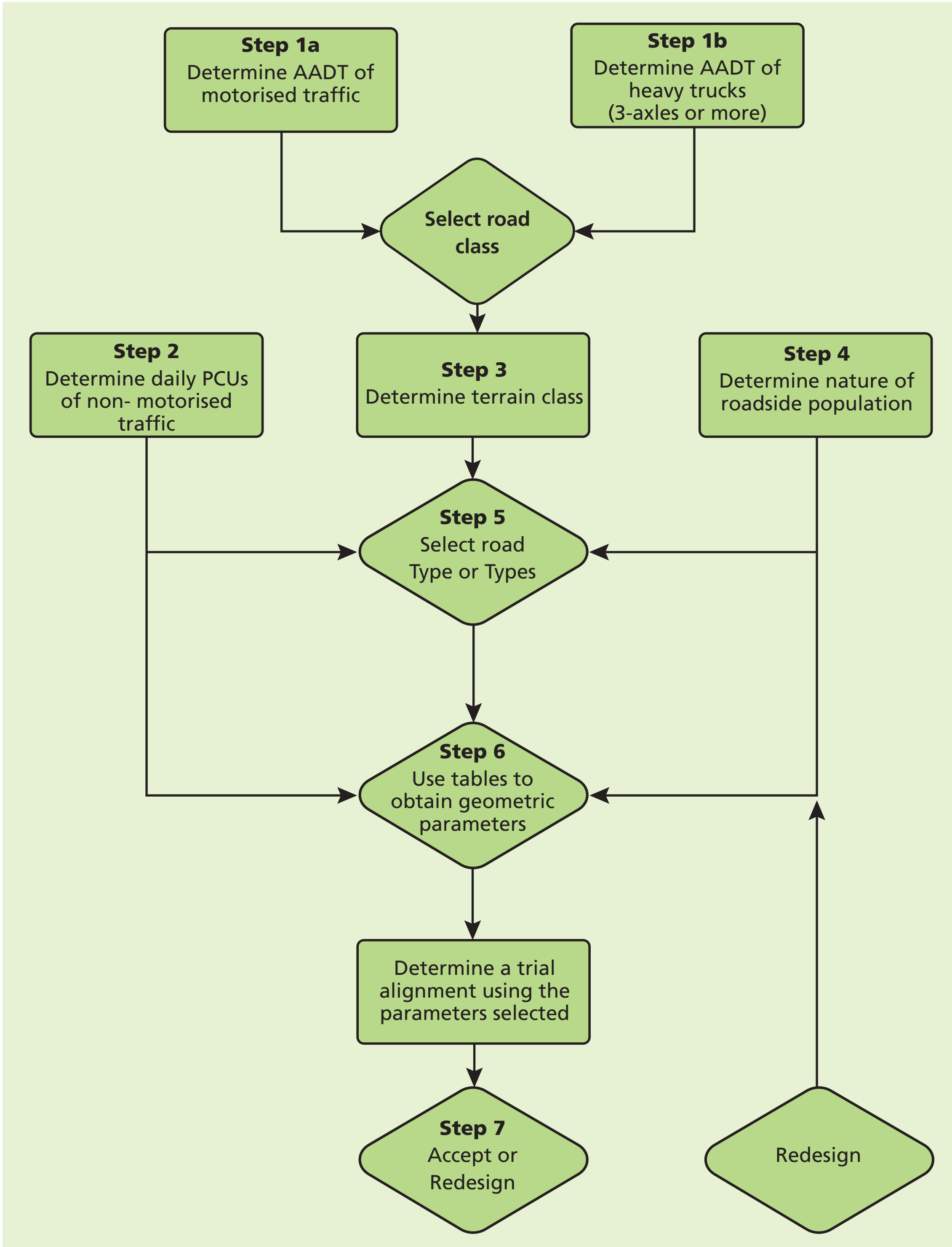

Figure 9-6: Flow chart for the selection of geometric standards for new roads

9.4 OTHER GEOMETRIC DESIGN ELEMENTS

9.4.1 General

There are a number of other geometric design elements that need to be considered in the design process and depend, to some extent, on whether Option A or Option B , as described in Section 9.2.3, is being used for the design.

9.4.2 Alignment

Applying Option A, the geometric design elements such as sight and stopping distances, minimum curve radii, k-values etc. are dealt with in a different manner than for higher classes of road that are designed for a specific design speed. By allowing for varying travelling speed dictated by the alignment, the comfort and safety of the road users can normally be achieved through appropriate traffic calming measures and widening where so required.

The alignment is basically determined using a "design-by-eye" approach. The designer may want to do simple surveys and cross-check perceived problematic spots against the standard design criteria in the RGDM, and there is no harm in doing so before the final decision is taken. The extra cost of satisfying the standard design criteria must then be weighed against the cost and service level that can be achieved with alternative solutions. Minor adjustments may also be done at the time of construction.

The advantages of a design by eye approach are:

- It enables the engineer to fit the alignment to the terrain so that it causes minimum disturbance to any existing facilities and the adjacent physical environment.
- Only a basic centre line topographic survey of the existing road is required to estimate quantities and produce tender drawings.
- The geometry of the road may be described on a simple strip map showing the existing horizontal alignment with kilometre stationing. Possible improvements, such as improved sight distances where necessary, are easily indicated.

9.4.3 Camber and Crossfall

Achieving proper camber and crossfall is essential to ensure rapid shedding of water off the carriageway and to prevent moisture ingress into the pavement from the top, as discussed in Section 12.3.3.

On paved roads a camber of 3.5% is recommended, as shown in Figure 9-7. Although steeper than traditional specifications, it does not cause problems for drivers in a low speed environment. It also accommodates reasonable construction tolerance of +/- 0.5% (taking into account the skills and experience of small scale contractors and LBM of construction), and provides an additional factor of safety against water ingress into the pavement should slight rutting occur after some time of trafficking.

On curves, a reversed camber in the outer lane will normally provide sufficient super-elevation if speeds are kept low. Otherwise, the design of super-elevation in curves should be in accordance with the specifications in the Road Geometric Design Manual (MOW, 2011).

Figure 9-7: Recommended camber on paved roads

9.4.4 Side Slopes and Low Embankments

Side slopes should be designed to ensure the stability of the roadway and, on low embankments, to provide a reasonable opportunity for recovery if a vehicle goes out of control across the shoulders.

Figure 9-8 illustrates the general cross section and defines the various elements. The position of the side drain invert should be a reasonable distance away from the road to minimise the risk of infiltration of water into the road pavement structure when the drain is full for any length of time.

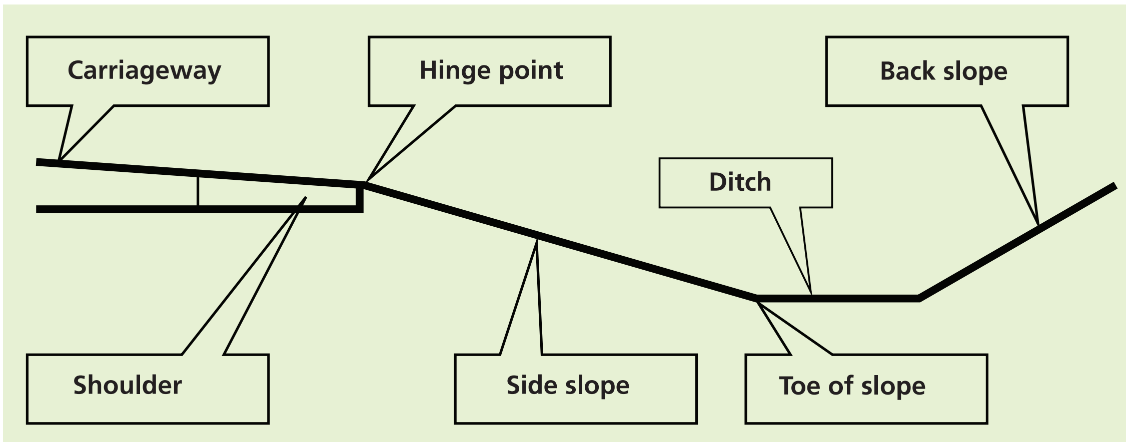

Figure 9-8: Details of the road edge elements

The side slope is defined as ‘recoverable’ when drivers can generally recover control of their vehicles should they encroach over the edge of the shoulder. Side slopes of 1:4 or flatter are recoverable. Research has also shown that rounding at the hinge point and at the toe of the slope is also beneficial.

A non-recoverable slope is defined as one that is traversable but from which most drivers will be unable to stop safely or return to the roadway easily. Vehicles on such slopes can be expected to reach the bottom. Slopes of between 1:3 and 1:4 fall into this category.

A critical slope is one on which the vehicle is likely to overturn and these will have slopes of greater than 1:3.

The selection of side slope and back slope is often constrained by topography, embankment height, height of cuts, drainage considerations, right of way limits and economic considerations. For rehabilitation and upgrading projects, additional constraints may be present such that it may be very expensive to comply fully with the recommendations provided in this manual.

Slope dimensions for the various conditions are summarised in Table 9-7.

Table 9-7: Slope dimensions for cross-sections (ratios are vertical:horizontal)

Material	Height of slope (m)	Side slope		Back slope	Safety classification
		Cut	Fill		
Earth	0.0-1.0	1:4	1:4	1:3	Recoverable
	1.0-2.0	1:3	1:3	1:2	Not recoverable
	>2.0	1:2(1)	1:2(1)	1:1.5	Critical
Rock	Any height	Dependent on costs			Critical
Expansive clays(2)	0-2.0	n/a	1:6		Recoverable
	>2.0	n/a	1:4		

Notes:
- Certain soils may be unstable at slopes of 1:2. Geotechnical advice required.
- The side drains should be moved away from the embankment.
- If critical and non-recoverable side slopes cannot be avoided, it is often appropriate to install ‘guard posts’ at critical locations.

9.4.5 Side drains

Detailed information concerning side drains is provided in Chapter 11. Substantial side drains should be avoided when the road traverses areas of expansive clays, with preference for the roadway to be constructed on a low embankment. Water should be discharged uniformly along the road. Where side drains cannot be avoided they should be a minimum distance of 4 m from the toe of the embankment and should be shallow and trapezoidal in shape.

9.4.6 Right-of-way

Right-of-way (or the road reserve) is provided to accommodate road width and the drainage requirements; to enhance safety; to improve the appearance of the road; to provide space for non-road travellers; and to provide space for upgrading and widening in the future. The width of the right-of-way depends on the cross-sectional elements of the highway, topography and other physical controls; plus economic considerations. Although extended rights-of-way are convenient, right-of-way widths should be limited to a practical minimum because of their effect on local economies.

Rights-of-way are measured equally each side of the centre line. Recommended Road Reserve widths applicable for the different road classes are shown in Table 9-1.

9.4.7 Shoulders, flush kerbs and edge beams

The functions of shoulders include:

- Giving structural support to the carriageway.
- Allowing wide vehicles to pass one another without causing damage to the carriageway or shoulder.
- Providing extra room for temporarily stopped or broken down vehicles.
- Allowing pedestrians, cyclists and other vulnerable road users to travel in safety.
- Limiting the penetration of water into the pavement.

On paved roads the whole roadway width should normally be sealed, whether shoulders are provided or not.

Gravel shoulders tend to be badly maintained and can pose serious danger to traffic (edge drop) and trap water that will penetrate into the pavement layers.

As much as provision of shoulders may be desirable, it may not always be economically feasible and strictly warranted in very low traffic situations. Where shoulders are not provided, there are other means of providing extra lateral support and to protect the edge of the sealed surface.

It is good practice to ensure even and proper compaction right to the edge of the roadway, to initially construct it slightly wider than the specified width and trim the side slope off with a rounded shape over, say, the first 0.3 to 0.5 m from the edge of the seal.

Other means of giving lateral support and protecting the edge of the surfaced area include a method that has been used extensively in Tanzania in the past, namely to provide flush kerb stones which may be set in mortar or just properly embedded in the top of the side slope (Figure 9-9).

Where minor accesses join the road or on sections with frequent “off carriageway” driving or parking occurs, concrete edge beams or flush kerb stones should be used to protect the edge of the seal.

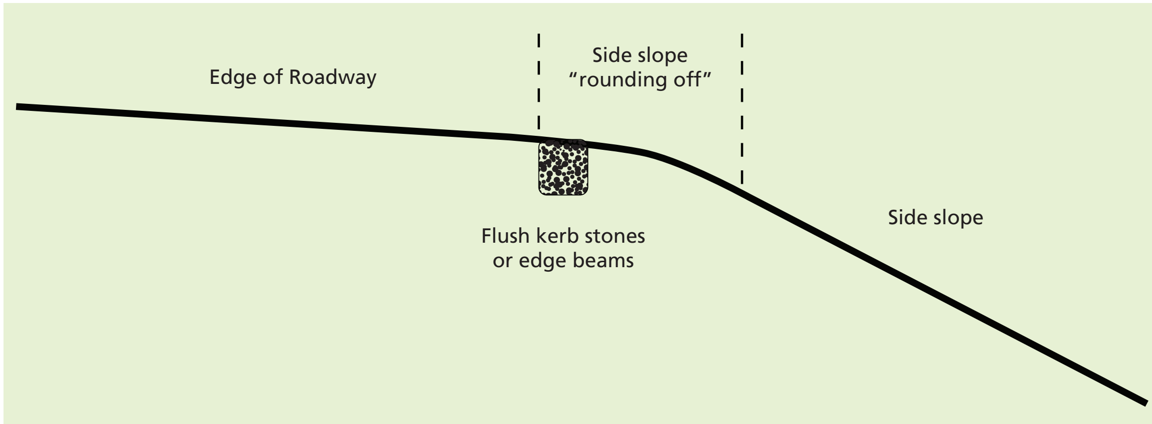

Figure 9-9: Side slope rounding off and location of flush kerb stones or edge beams

9.4.8 Single Lane Roads and Passing Bays

There is good agreement internationally about the recommended carriageway width for single-lane roads, namely 3.5 to 4.0 m, depending on traffic volume, mix and terrain. Passing bays maybe required, depending on the traffic level, and provision for other traffic and pedestrians will need to be introduced (e.g. wider shoulders) if the numbers of other road users exceed specified levels. The increased width should allow two vehicles to pass at slow speed.

Passing bays should normally be provided every 300 m to 500 m depending on the terrain and geometric conditions. Care is required to ensure good sight distances and the ease of reversing to the nearest passing bay, if required. Passing bays should be built at the most economic places rather than at precise intervals provided that the distance between them does not exceed the recommended maximum. Ideally, the next passing bay should be visible from its neighbour.

The length of passing bays is dictated by the maximum length of vehicles expected to use the road, indicating the need to define a design vehicle. A typical vehicle for LVRs is about 13 m long therefore passing bays of twice this length should be provided. In most cases a length of 25 m will be sufficient for LVRs.

A suitable width depends upon the width of the road itself. The criterion is to provide enough overall width for two design vehicles to pass each other safely at low speed. Therefore, a total trafficable minimum width of 6.3 m is required (providing a minimum of 1.1 m between passing vehicles). Allowing for vehicle overhang when entering the passing bay, a total road width of 7.0 m is suitable.

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CONTENTS

ROAD SAFETY

10.1 INTRODUCTION

10.1.1 Background

Road safety is of prime importance for all road users in Tanzania. However, the safety concerns of road users on low volume roads (LVR) tend to be quite different to those on high volume roads (HVR) and warrant special consideration. This is largely because there tends to be a much higher number of vulnerable road users on LVRs, including pedestrians, bicycles, motorcycles and animals, than on HVRs.

The challenge faced in a rural road environment is to ensure that the speed of motorized traffic is restrained to relatively low levels, particularly within villages, as this will go a long way to reducing crashes. However, this is not easily achieved because the roads serving these villages often serve two conflicting functions in that they cater for both inter- and intra-village traffic. As a result, a variety of traffic calming measures are required to minimize traffic crashes coupled with proactive strategies that address the causes of such crashes before they occur. A balance also needs to be struck between the costs of implementing such measures and the resulting benefits – an equation which is very difficult to quantify.

10.1.2 Purpose and Scope

The main purpose of this chapter is to raise awareness of the types of road safety problems that occur on LVRs located in Tanzania and to highlight typical measures that may be undertaken to improve road safety. Other general aspects of road safety that can be enhanced by proper design controls and enforcement of these controls are addressed in the Road Geometric Design Manual, Chapter 4: Design Control and Criteria (MOW, 2011).

The chapter highlights the importance of planning for road safety coupled with appropriate design, construction and maintenance of LVRs as well as through road safety education and enforcement. Special attention is paid to speed reduction measures in villages where there is a high incidence of vulnerable road users.

10.2 IMPORTANCE OF ROAD SAFETY

10.2.1 General

Because of its multi-dimensional nature, road safety cannot be discussed in isolation of a number of related factors. As illustrated in Figure 10-1, road safety is linked to, and influenced by, various elements of the road system, including road design, environmental conditions, road maintenance and related pavement condition.

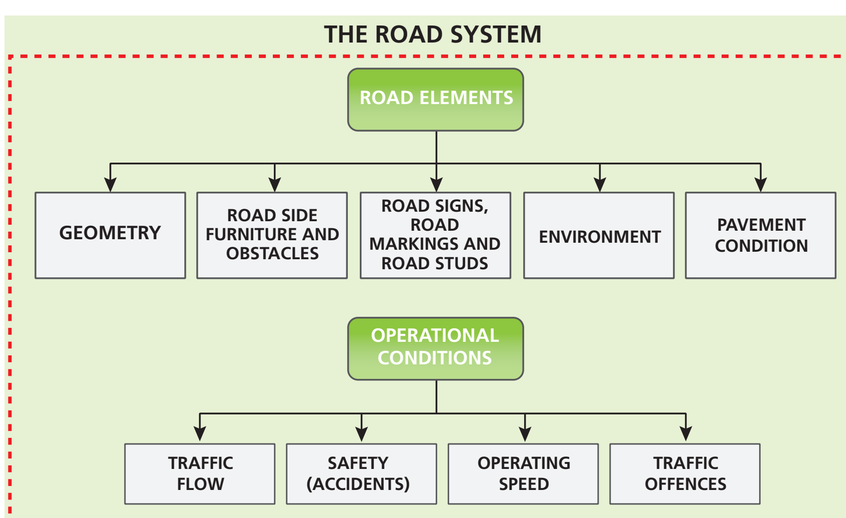


Figure 10-1: Elements of the road systems and operational condition

Many of the road elements and operational conditions are also interrelated. For example, as illustrated in Figure 10-2, overloading has an important influence on pavement condition and is influenced by police surveillance, while speeding is also influenced by the road geometry and police surveillance. Both overloading and speeding, in turn, have an influence on safety in terms of crashes whilst the environment, coupled with the maintenance standard applied, affects pavement condition and, in turn, road safety.

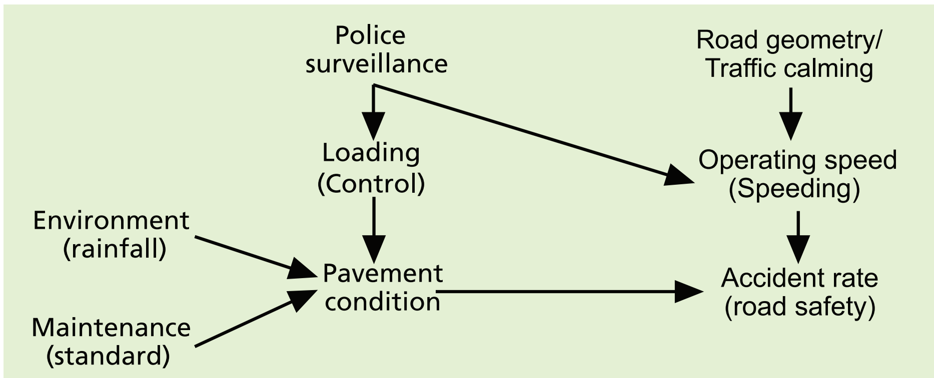


Figure 10-2: Interrelationship between road elements and operational conditions

Thus, although the importance of designing for safety on LVRs is now widely recognised, the actual process of identifying key design features and resolving the conflict of safety and other considerations is complex (interrelated) and requires tackling in a holistic manner.

10.2.2 The Nature of Crashes

Traffic crashes on LVRs tend to be multi-causal in nature, involving human factors, the road environment and vehicle factors. They are more often caused by a combination of these factors, with human factors typically contributing to an estimated 95% of all crashes, the road environment about 28%, and vehicles about 8%, as shown in Figure 10-3.

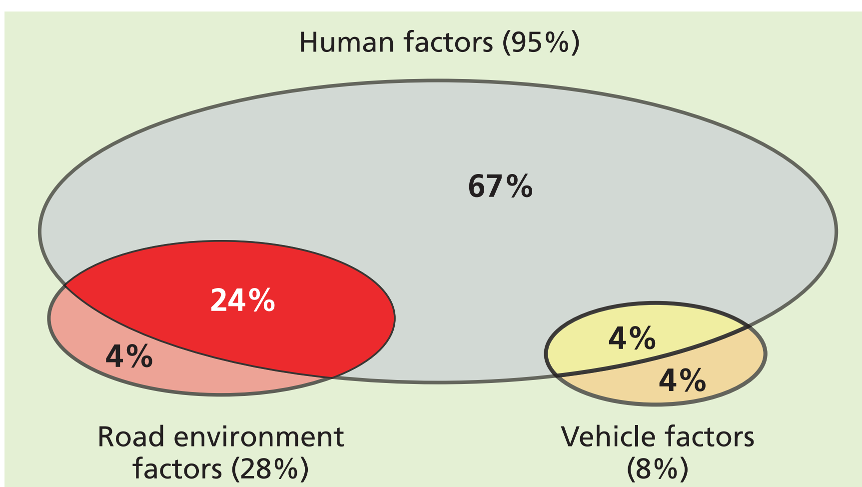


Figure 10-3: Factors contributing to road crashes

It is apparent from Figure 10-3 that confronting the challenge of road safety requires proactive strategies that treat the root cause of crashes – human factors – and, to a lesser extent, road environment and vehicle factors - a major challenge that confronts many countries.

Human factors

Human factors are associated largely with human error of one kind or another and include:

- Misjudgment/overtaking/inattention/distraction.
  - Use of mobile phones.
  - “Distracted“ walking.
- Speeding.
  - Inappropriate/excessive speed.
- Drink-driving.
  - Driving under the influence of alcohol/drugs.
- Negligence by drivers, pedestrians or cyclists.
  - Non-use of seatbelts, child restraints or helmets.
  - Sheer disregard/lack of knowledge of road traffic rules and regulations.
- Fatigue.
  - Driving for excessively long periods without adequate rest.

Road environment

Road environment factors include:

- Poor road design.
  - Inadequate road capacity.
  - Failure to separate pedestrians/NMTs from vehicular traffic.
  - Blind crest curves.
  - Sharp horizontal curve immediately after a sharp crest.
  - Sharp curve requiring a reduction in speed of more than 20 km/h.
  - Road geometry that does not encourage and enforce a reduction of speed on hazardous sections.
  - Inadequate road signage and markings.
  - Hazards close to the edge of the road.
  - Inappropriate choice of road surfacing.
  - Unmarked changes in road surfaces (particulary form paved to unpaved).
- Poor maintenance.
  - Deterioration of surfacing and pavement structure.

Vehicle factors

- Un-roadworthy vehicles.
  - Poor brakes/no horn/no lights.

10.2.3 Typical Causes of Road Crashes

The following are typical causes of crashes that occur on LVRs roads where many vulnerable road users, particularly motorcycles and NMT, are put in a high risk situation:

- Inadequate planning and designing for road safety due in part to the non- inclusion of pedestrian and slow-moving traffic in traffic surveys, and consequent failure to take proper account of the operational environment.
- The provision of relatively steep cambers, typically 5-7% on gravel roads, in order to shed water off the road. This camber may provide little difficulty to motorized vehicles which tend to travel along the centre of the road. However, it can be very dangerous for cyclists and motorcyclists who often carry very large/heavy loads and, as a result, are unable to easily manoeuvre out of the way of fast-approaching traffic. This often results in them falling off their bicycles, damaging both themselves and their goods.
- The roadway outside the longitudinal drainage ditch is seldom cleared by more than a few m, if at all, and, in addition, people often tend to build houses quite close to the road. In these situations, there is potentially a considerably increased risk to pedestrians, particularly young children, as there is little warning to motorised traffic when pedestrians or animals decide to cross the road.
- A combination of poor motorcycle driver behaviour, such as use of inappropriate speed, and poor road condition, such as a potholed or slippery surface, causing the motorcyclist to lose control.
- Relatively fast-moving motorised traffic competing for limited road space with much slower-moving non-motorized modes of traffic and pedestrians, as shown in Figure 10-4.

Figure 10-4: Problems encountered on unpaved road

Figure 10-5: Problems encountered on concrete strip road

10.3 APPROACH TO ROAD SAFETY

10.3.1 General

Despite the fact that human error is the chief causal factor in most road crashes, there is little doubt that proper planning, design and implementation of appropriate road safety measures can affect road user behaviour and reduce the frequency with which errors and crashes occur. Thus, it would be sensible and easier to make the road simpler to use and more forgiving than it is to improve the skill and behavior of the road user.

Pedestrians, motor bike riders and cyclists have been found to be the most vulnerable road users on all roads, not just LVRs. Unfortunately, however, they tend to be treated as obstacles to motorized traffic, even when it is obvious that they are the majority of road users, particularly on rural access roads. Thus, they should be viewed not as a peripheral concern but, rather, as being central to addressing the road safety problem in Tanzania.

10.3.2 Planning for Road Safety

It is often possible to improve road safety characteristics at markedly little or no extra cost, provided the road safety implications of design are considered at the planning stage. This requires adherence to a number of key principles that include:

- ***Catering for all road users***
  - Includes non-motorized vehicles, pedestrians, cyclists, motorcyclists, disabled persons, etc.
  - Has implications for almost all aspects of road design, including carriageway width, shoulder design, side slopes and side drains.
- ***Providing a clear and consistent message to the driver***
  - Roads should be easily "read" and understood by drivers and should not present them with any sudden surprises which should be addressed by appropriate signage or other measures.
- ***Encouraging appropriate speeds and behavior***
  - Traffic speed can be influenced by altering the "look" of the road, for example by providing clear visual clues such as changing the shoulder treatment or installing prominent signing.
- ***Reducing conflicts***
  - Cannot be avoided entirely but can be reduced by design, including staggering junctions or using guard rails to channel pedestrians to safer crossing points.
- ***Creating a forgiving road environment***
  - Forgives a driver's mistakes or vehicle failure to the extent that this is possible without significantly increasing costs.
  - Ensures that demands are not placed upon the driver, which are beyond his or her ability to manage.
- ***Undertaking appropriate traffic counts***
  - Ensuring that traffic counts also include pedestrians, bicycles, motorcycles and other forms of NMT. Such information will be influential in planning aspects of the geometric layout of the road, such as shoulder widths.
- ***Undertaking road safety audits***

  Because of the paramount importance of road safety on LVRs, a road safety audit should be undertaken at the planning stage of a new project or before upgrading of an existing project. Such an audit should systematically identify hazardous features, including crash "black spots" and the crash potential related to the improvement/upgrading of the road, and should propose treatments that will reduce crash risk to road users. Site specific remedial treatments should be identified and prioritized for early implementation, based on the risks identified at the audit stage.

10.3.3 Designing for Road Safety

Segregating motorized traffic from NMT

Appropriate design implies designing for all road users and has implications for almost all aspects of LVR design, including carriageway width, shoulder design, side slopes and side drains. Specific measures or combinations of measures include:

- Providing segregated footpaths on narrow bridges, as shown in Figure 10-6, or adjacent to the carriageway in rural areas, as shown in Figure 10-7, and peri-urban areas, as shown in Figure 10-8, to offer safe access for pedestrians.
- Providing sealed shoulders where a segregated footpath is not possible, as shown in Figure 10-9.
- Providing bicycle lanes by allocating part of a road to bicycles or by building off-road paths, on-road or off-road bicycle lanes.

Figure 10-6: Segregated footway on narrow bridge
(Provides effective protection for pedestrians)

Figure 10-7: Segregated footpath off road
(Cost effective means of catering for high volumes of pedestrian traffic)

Figure 10-8: Simple poles used to increase safety for pedestrians

Figure 10-9: Sealed shoulder used to cater for pedestrian or NMT

Junctions

Conflicts may be reduced by staggering junctions or using guard rails to channel pedestrians to safer crossing points. Figure 10-10 illustrates some good and poor practices.

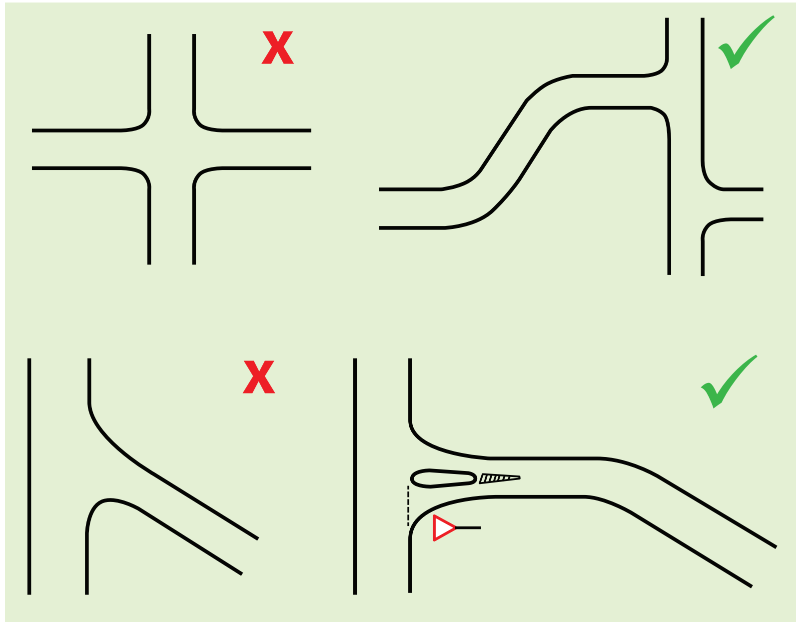


Figure 10-10: Good and bad junction designs

Roadside access

- Road safety in areas that provide roadside access to a variety of facilities may be improved by:
- Prohibiting direct frontal access to major routes and instead use of service roads.
- Using lay-bys or widened shoulders to allow villagers to sell produce.
- Using lay-bys for buses or taxis to avoid restriction and improve visibility.

Examples of the above measures are illustrated in Figure 10-11.

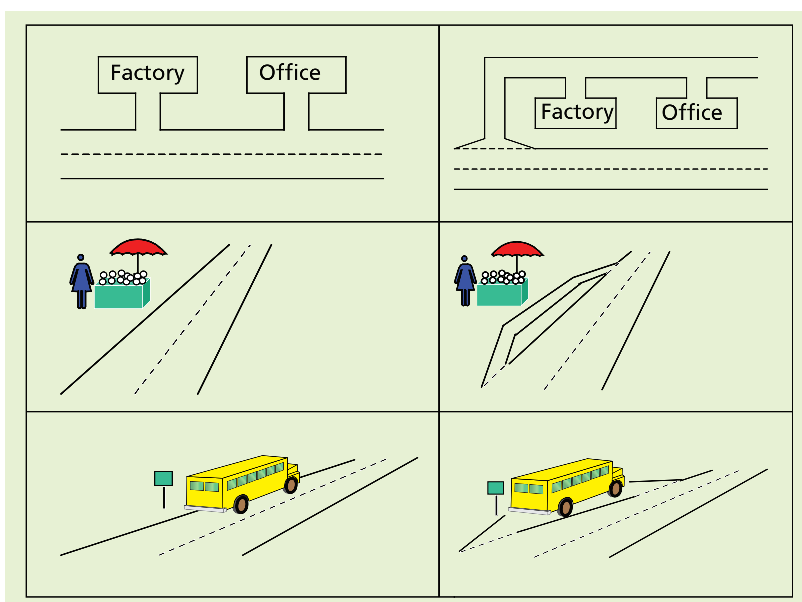


Figure 10-11: Examples of improving roadside access to reduce crashes

Roadside Ditches/Side Drains

The type of drainage ditches/side drains that are provided within the road cross section can have a significant effect on road safety in terms of the ability of the drive to "recover" from running off the road. Careful design and location of such facilities are required to provide a "forgiving road environment". Figures 10-12 and 10-13 illustrate examples of good and poor design of roadside drains/ditches.

Figure 10-12: Well designed roadside drain
(Shallow side slopes improve safety)

Figure 10-13: Inappropriate choice of roadside drain
(Leaves no room for recovery if a vehicle runs off the road)

Clear Zone

Many crashes are made more severe because of obstacles that an out-of-control vehicle may collide with. The concept of clear zones identifies these obstacles and attempts to eliminate such hazards. The aim is to provide a roadside that is forgiving. In other words it enables the driver to avoid colliding with anything, and recover control. This means that there must be an obstacle-free strip – the Clear Zone (Safety Zone or Recovery Zone) – on both sides of the road because vehicles run off the road to both the left and to the right. Shoulders are usually classed as part of the Clear Zone.

Features that are regarded as obstacles are:

- Embankment side slopes steeper than 1V:4H. Slopes as steep as 1V:3H may be acceptable provided that there is a clear run-out area at the bottom of the embankment).
- Back slopes steeper than 1V:2H.
- Non-deformable rigid obstacles such as concrete guard posts, bridge piers and abutments, retaining walls, rock cuttings, walls, culvert headwalls, and rigid supports for sign gantries and large signs.Obstacles such as trees, lighting posts, and supports for signs.
- Ditches and open drains (unless designed to be traversable).
- Fences.

Where obstacles in the Clear Zone cannot be avoided, it will be necessary to consider whether they should be shielded by a safety barrier or marked with guard posts.

Figure 10-14 provides guidance on dealing with roadside hazards in order to provide a "forgiving roadside".

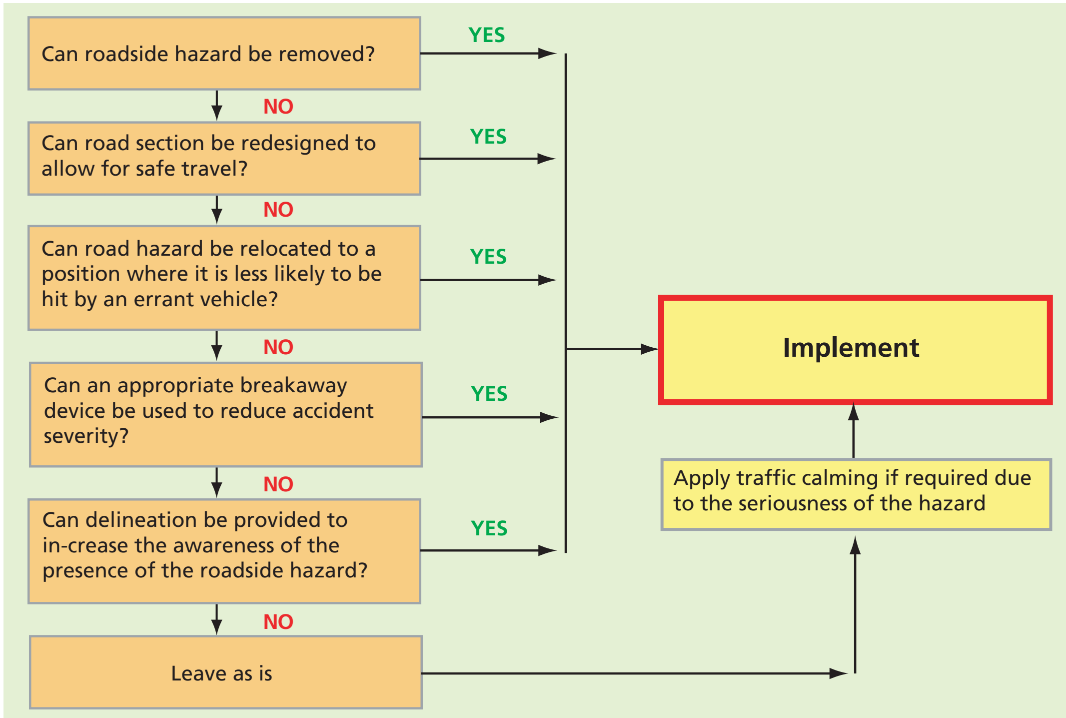


Figure 10-14: Procedure for dealing with roadside hazards

10.3.4 Traffic Calming

Traffic calming typically comprises a combination of mainly physical features that can reduce the negative effects of motor vehicle use, alter driver behavior, and improve conditions for NMT users. Such measures focus on reducing speeds through the use of self-enforcing traffic engineering methods or through road design.

There are a number of relatively low-cost traffic calming measures that can be introduced, particularly within villages, to reduce vehicle speed and thus improve the safety of road users, especially NMT. Specific measures include:

- Encouraging police to enforce local speed limits.
- Providing regulatory traffic signs of local speed limits.
- Calming traffic with speed humps, rumble strips, road narrowing, pedestrian crossings and specially demarcated low speed zones.

Traffic calming measures in villages require special attention. This is because the roads serving these villages are often required to serve two conflicting functions in that they must cater for both inter- and intra-village traffic. As a result, traffic entering the village often does so at speeds that are much too high for a village environment where there is slow moving turning traffic, parking outside shops and stalls and the needs of pedestrians who require to move along or across the road. Such a situation requires the need for a comprehensive “village treatment” which will induce a driver to reduce speed significantly as he or she passes through a village.

Village treatment – paved roads

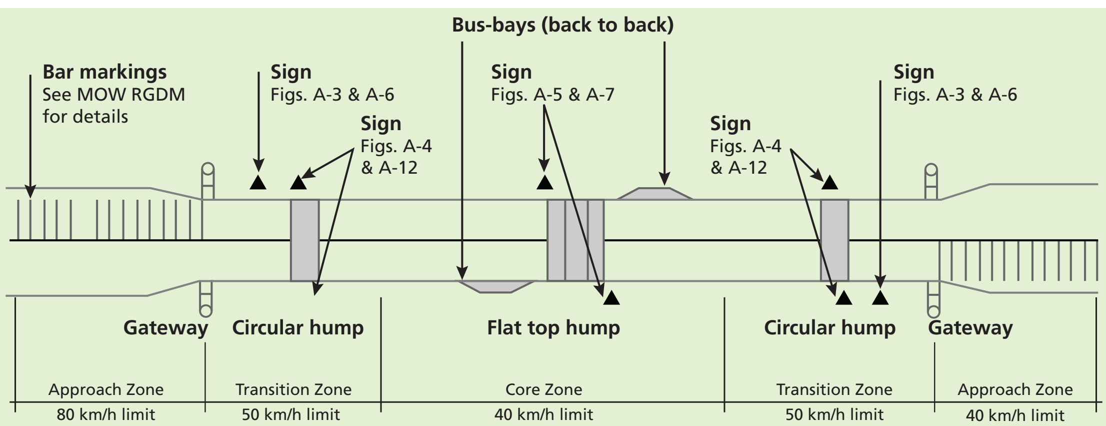
The objective of the “village treatment” approach to traffic calming is to develop a perception that the village is a low-speed environment and to encourage the driver to reduce speed as a result of this perception. To this end, the road through the village is divided in three zones, namely:

(i) The approach zone.

(ii) The transition zone.

(iii) The core zone.

- **Approach zone:** This is the section of road prior to entry into the village, where the driver needs to be made aware that the open road speed is no longer appropriate. This is the section of road where speed should be reduced typically from above 60 km/h down to about 50 km/h, before entering the village. The village entry should be marked by a Gateway as described later in this chapter.
- **Transition zone:** This is the section of road between the village entrance, or Gateway, and the core zone of the village. The target speed, and posted speed limit in this zone would be maintained at typically 50 km/h. The first road hump or humps in a series of humps will be sited in this zone. In this context, with adequate advance warning provided by the approach zone and Gateway, road humps are quite safe.
- **Core zone:** This is the section identified as being in the center of the village, where most of vehicle/ pedestrian conflicts would be expected to take place. This would normally be where the majority of shops, bus-bays or other pedestrian generating activities are located. This is the section where pedestrian crossing facilities are most likely to be established and where the target speed, and posted speed limit, should typically be reduced further to 40 km/h. Road humps would normally be provided within this zone with advisory speed limits of 20 km/h in order to enforce the lower speed environment required.



Notes:
1. Layout not scale.
2. Road cross-section dimensions and speed limits are indicative only and should be amended to suit site conditions and traffic regulations.
3. RGDM - Road Geometric Design Manual (2011)

Figure 10-15: Village treatment - typical layout

The elements which make up the village treatment are as follows:

- **Roadway bar markings:** These are used in the approach zone as the initial warning to the driver that a speed reduction is required. They are painted on the carriageway immediately in advance of the village entry point or Gateway (see Road geometric Design Manual (MOW, 2011) for further details).
- **The Gateway:** This marks the main entrance to the village and is a clear indication to the driver that the road is now changing in character at this point as an additional encouragement to reduce speed. It is also important to ensure that devices used as part of a gateway treatment are: (1) crashworthy if placed within the clear zone and (2) do not obstruct sight distance. Details of the gateway are shown in Figure A-1.

- **Rumble strips:** These are used as warning devices to drivers to reinforce the fact that the driver is approaching a village environment as shown in Figure A-4. The distance between strips should be shortened on the approach to the village so as to give an audible change in the pitch of the sound which is likely to impact on the driver driver (see Road Geometric Design Manual (MOW, 2011) for further details).
- **Road humps:** These are the main self-enforcing means of producing a speed reduction. There are two types of humps as follows:
  - **Circular profile hump** which has been designed to provide the required reduction in speed while at the same time providing a reasonably comfortable ride for passengers and the least damaging effect on vehicles when travelling at the advisory speed.

    The specific purpose of the Circular profile hump is to lower traffic speeds. As such, it is most useful as a back-up to the gateway so that drivers have little option but to slow down before reaching the core zone. For this reason, the first hump in a series of humps should always be a Circular profile hump, and should always be sited in the transition zone, i.e. between the Gateway and the core zone (see the Road Geometric Design Manual, MOW, 2011) for further details).
  - Flat-top hump: of which the top portion of this hump is flat with a ramp on either side (see the Road Geometric Design Manual (MOW, 2011) for further details).

The flat-top hump will generally be used at locations within the core zone of the village where there is a need for zebra crossings on popular pedestrian routes (ussualy near schools, bus stops and markets). In this situation, the hump may be combined with a pedestrian crossing, which would be sited on the flat part of the hump.

Figure 10-16: Circular profile hump

Figure 10-17: Appropriate signage for pedestrian crossing on flat top hump

The recommended sacing and combination of road humps is shown in the annex to this chapter.

- **Pedestrian crossings:** Such crossings can be expediently combined with flat-top humps by locating them on the flat top of the hump which itself is located in the core zone of the village and has been designed to provide a safe crossing place for pedestrians, as well as functioning as a speed retarder. These crossings are usually sited at the busiest crossing points in the village.
- **Traffic signs and road markings:** These are used to provide warning information to motorists at all elements of the village treatment, such as at entry to the Gateway, at the start and exit of the core zone, in advance of road humps, advising speeds, etc.

- **Bus-bays and shelters:** These should generally be provided in the core section of the village and should be in pairs. They should be located back-to-back, i.e. when there is a bus in each bay, they should be facing away from each other so that passengers leaving a bus and then crossing the road, will be behind a bus parked on the opposite side, and will not be crossing in front of it (see the Road Geometric Design Manual (MOW, 2011) for further details).
- **Pedestrian routes:** The aim should be to identify the major pedestrian routes within the village, to determine at what point they join the road, and whether any realignment is necessary to ensure that pedestrians are led to appropriate crossing places. The main pedestrian route within a village should always join the road within the core zone.

The recommended spacing and combination of road humps is shown in Figure A-2.

Notwithstanding the potential road safety benefits to be derived from the installation of appriopriate traffic calming measures, certain precautions should always be observed when implementing them. For example, speed humps or raised pedestrian crossings should generally not be implemented in the following situations:

- In front of, or very near to, driveway locations.
- Near roadway intersections (should be placed at least 15-20 m away from the intersection).
- Near shade of trees and other physical objects which might obscure them during the day time.
- Where they are not illuminated by street lighting at night.
- On gradients in excess of 8 %.
- On curves.

Unpaved roads

Although traffic levels on unpaved roads in villages generally tend to be lower than on paved roads, traffic speeding, combined with dust emissions, are a major problem for which appropriate traffic calming measures are also required. Such measures are, in principle, similar to those for paved roads in terms of signage. However, special measures need to be taken to embed road humps in the gravel substrate so as to anchor them and minimize their horizontal movement under the action of traffic (see typical layout in Figure 10-18).

Various materials may be used for constructing gravel road humps including cement-mortared brick or stone masonry and stabilized gravel. In addition, where required for cross drainage purposes, the use of raised culverts and drifts, by virtue of their natural profile, can also act as traffic calming devices. In all cases, their location should be well signed which is very important to avoid serious accidents. It should also be noted that speed humps very often lead to accumulation of water and measures need to be in place to avoid this.

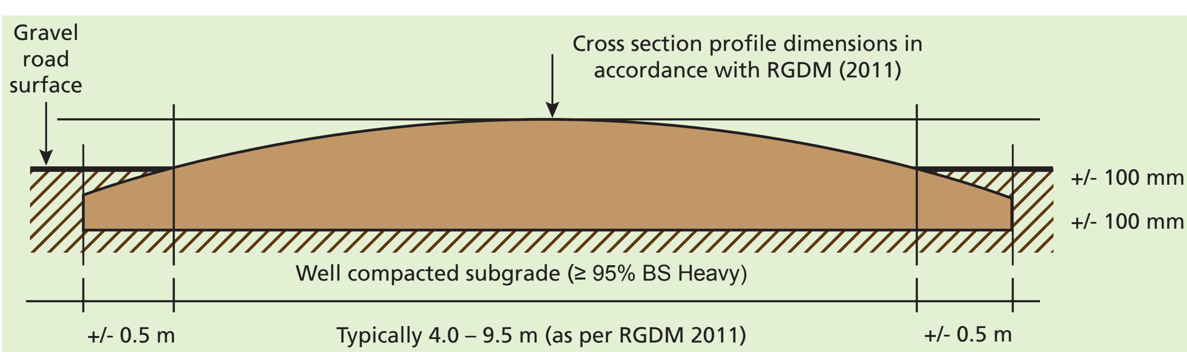


Figure 10-18: Longitudinal profile of gravel road hump (circular)

10.3.5 Regular Maintenance

It is the function of the local authority to ensure that safety hazards are minimized by carrying out regular maintenance of road safety infrastructure. Such measures include:

- Vegetation growth which can obscure visibility, for example, at sharp curves.
- Potholes in the road surface.
- Flooding resulting from blocked culverts.
- Dirty, damaged or missing traffic signs.
- Faint road markings.
- Damaged bridges and guardrails.
- Scoured road shoulders.

As illustrated in Figure 10-19, failure to carry out adequate road maintenance can impact adversely on road safety in that it prevents motorcycles and non-motorized users from using the shoulders when required to move off the concrete strips due to on-coming motorized traffic.

Figure 10-19: Example of poorly maintained concrete strip road

10.3.6 Road Safety Education

Road safety education (RSE) is an important tool to raise awareness of problems and behaviour related to traffic and road safety. It involves teaching children, who are often the most vulnerable group of affected road users, and adults to be safer road users. It does so by developing:

- Knowledge and understanding of road traffic.
- Behavioural skills necessary to survive in the presence of road traffic.
- An understanding of their own responsibilities for keeping themselves safe.
- Knowledge of the causes and consequences of road crashes.
- A responsible attitude to their own safety and to the safety of others.

Figure 10-20: Road safety education for school children

10.3.7 Publicity campaigns

Road safety publicity campaigns can also raise the awareness of problems and behaviour in addition to improving knowledge, shaping attitudes and behaviour, as well as stimulating discussion and debate. These publicity campaigns can include local drama performances in which local languages are used in order to reach all members of the community. Community workshops, radio & television broadcasts and cinema can also be a useful means of tools in promoting road safety.

Figure 10-21: Publicity campaign on road safety at local level

10.3.8 Law enforcement

Traffic law enforcement is meant to achieve the safe and efficient movement of all road users including non-motorised traffic and pedestrians. In this regard, enforcement of traffic rules (such as speed limits, stop signs and rules at pedestrian crossing facilities) can be used to significantly improve road user behaviour and safety. This situation highlights the need to promote traffic law enforcement more vigorously, including the use of well mounted campaigns which, ideally, should be accompanied by education and publicity.

Annex A – Key elements of the Village Treatment approach to road safety

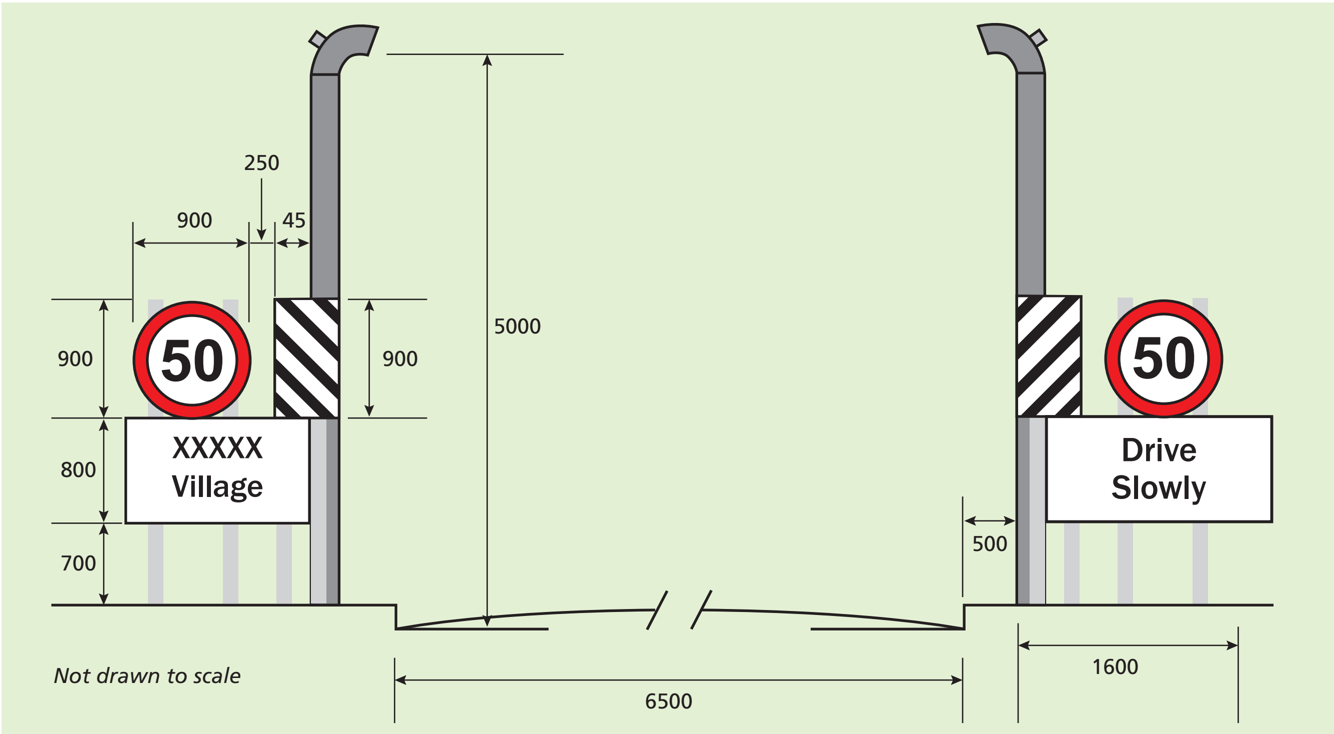


Figure A-1: Village safety measures - Gateway approach to village

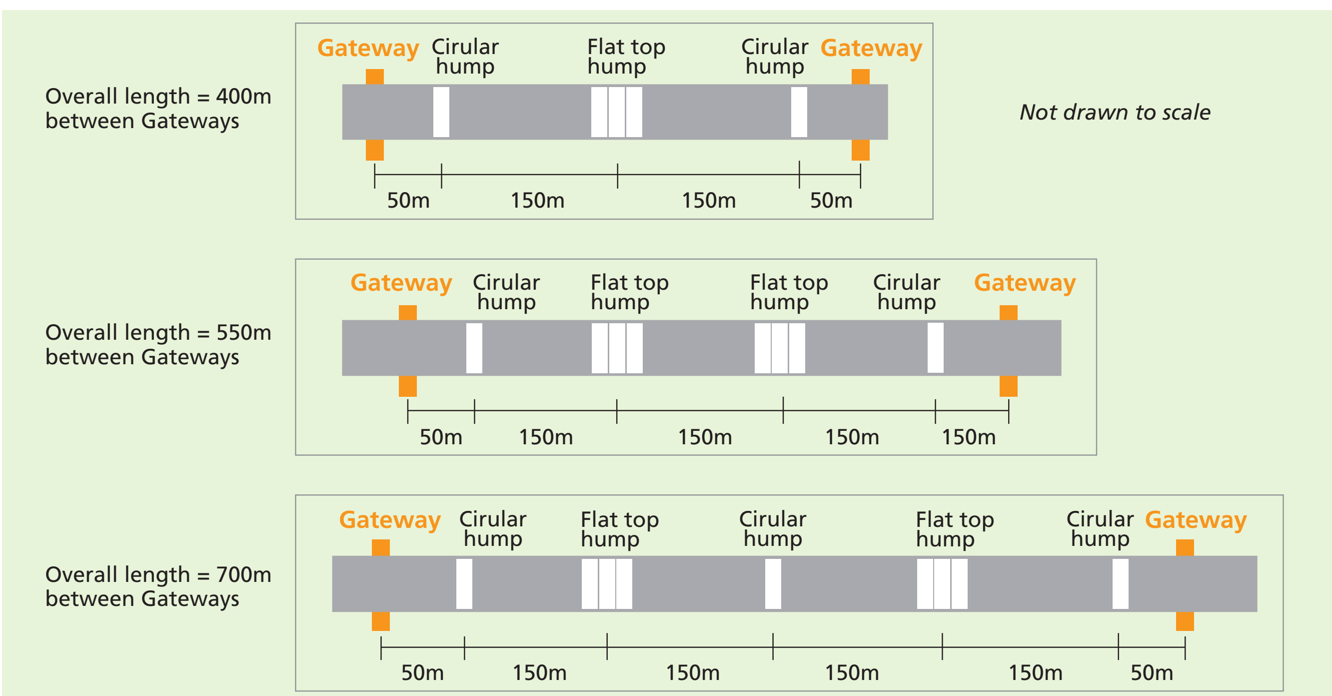


Figure A-2: Village treatment - Typical spacing and combination of road humps

Figure A-3 *

Figure A-4* (W332)

Figure A-5

Figure A-6*

Figure A-7*

Notes: * Denotes non-standard sign - Note: Sign A-4 may be used instead.
** Denotes National sign.

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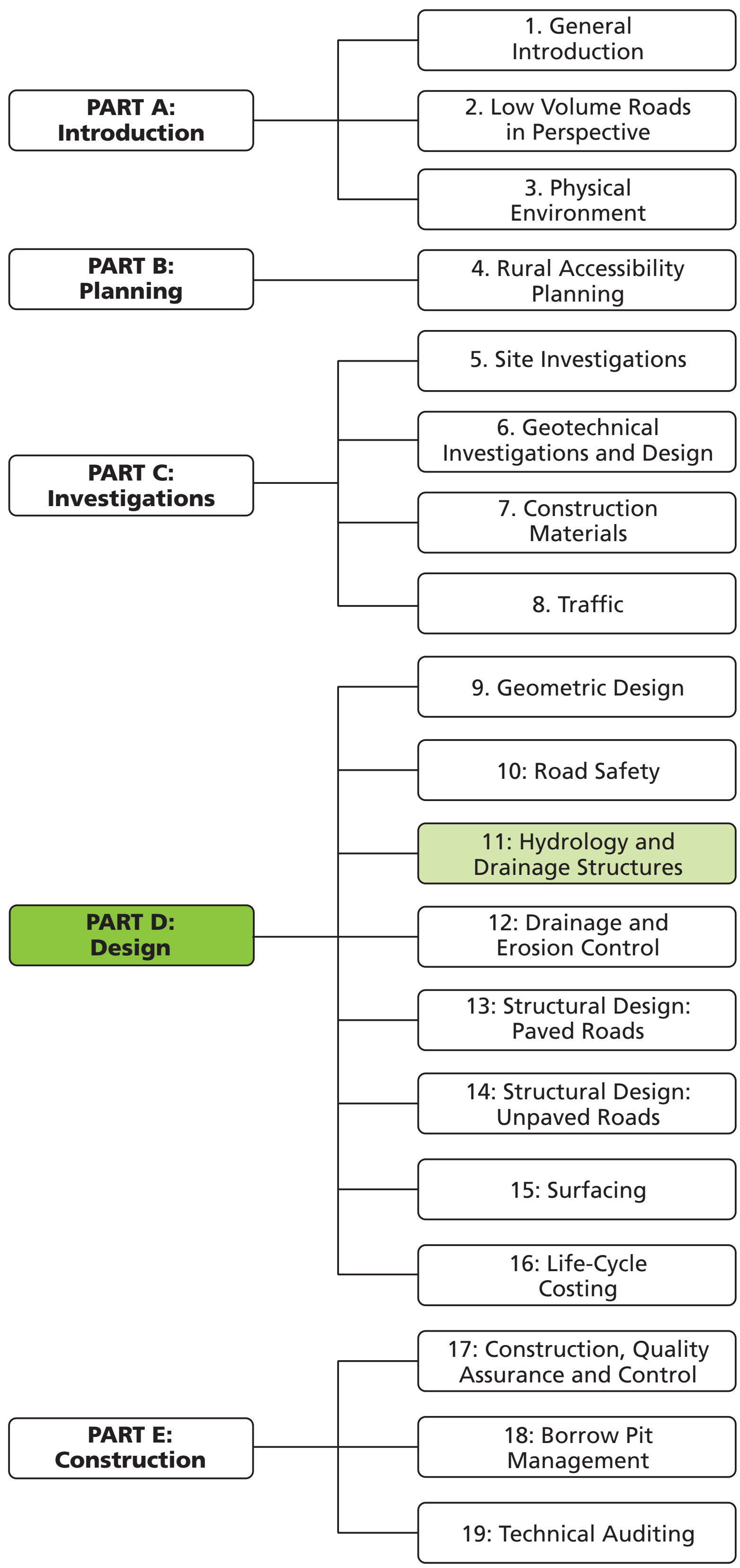

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11.1 INTRODUCTION

11.1.1 Background

Hydrology and hydraulic analysis for road drainage design may be defined as the estimation of flood (rainfall) run-off from a catchment for a specified but rare storm and the design of drainage structures of appropriate capacity. It includes the assessment of risks associated with such rare climatic events. The level of risk that is acceptable depends on the consequences of failure of the drainage structure. For trunk roads and for expensive structures such as bridges, limited risk can be tolerated and so high safety factors and expensive drainage measures are employed. For small drainage structures that can be repaired relatively quickly and easily and for the lower classes of road, higher risks can be tolerated. The challenge for the engineer is to choose a level of protection that is commensurate with the class of road and the structures on it, hence a certain amount of engineering judgement is always required.

11.1.2 Purpose and Scope

This Chapter is concerned with hydrology and the process of determining the quantity of water that the drainage design must cope with. The Chapter is essentially the prelude to Chapter 12, comprising the first step in drainage design. When risk factors have been selected and the volume of water that must be catered for has been determined, then the individual features of the drainage can be designed using hydraulic design principles.

The Chapter deals primarily with new structures such as drifts, culverts and small span bridges. However the design principles are the same for reconstruction, rehabilitation, extension and upgrading of existing structures. The structures are ranked in order of increasing complexity as follows:

- Drifts or simple fords.
- Simple culverts.
- Vented drifts.
- Large diameter culverts.
- Small bridges.

The Chapter does not cover aspects of road drainage such as drainage ditches and sub-surface drainage. These are addressed in *Chapter 12 - Drainage and Erosion Control.*

11.2 DESIGN STORM

11.2.1 General

The first step in the determination of the flow of water that the drainage system needs to cope with depends on the severity of the design storm. The risk of a severe storm occurring is defined by the statistical concept of its likely return period. This is directly related to the probability of such a storm occurring in any one year. Thus a very severe storm may be expected, say, once every 50 years, but a less severe storm may be expected every 10 years. This does not mean that such storms will occur on such a regular basis. A severe storm expected once every 50 years has, on average, a probability of occurring in any year of 1 in 50 based on historic rainfall data.

Rainfall data for the last 50 years or so are usually available for most of Tanzania hence estimates can be made of the rainfall in quite rare storm events. In areas where the data are only available for a relatively short period of less than 10 or 20 years (i.e. insufficient for estimating storms with return periods of, say, 25 years or more) an estimate can be made from the more limited data using the adjustment factors shown in Table 11-1 based on a normalized storm return period of 10 years. Estimating the 100 year storm from, say, less than 5 years data will not be very accurate. A minimum of 10 years is recommended. From the Table, a storm with a return period of 20 years will provide 1.15 times more water than the storm with a return period of 10 years.

Table 11-1: Adjustment factors for different storm return periods

Return period (years)	1	2	5	10	12.5	20	25	50	100
Adjustment factor	0.55	0.7	0.85	1.0	1.05	1.15	1.2	1.35	1.5

Source: Adapted from international review

11.2.2 Design Storm for Different Structures

The indicative design storm return period for different structures is shown in Table 11-2. However there are a number of situations where the design storm could be more severe. Principal routes such as access roads to local markets or emergency routes to a nearby hospital will require higher levels of reliability and shorter periods of closure caused by high water levels hence the design storm design period chosen should be longer (i.e. more severe). The proximity and distance of an alternative route will also affect the choice of design storm (and drainage structure). If there is an alternative secure route with a short acceptable detour, this will allow the road to be closed for longer periods whereas the lack of any alternative route (or one of excessive length) will require a more conservative design. Therefore the choice of storm design period requires careful engineering judgement and local consultations. Table 11-3 should be used when less risk can be tolerated.

Table 11-2: Indicative storm design return period (years) for different structures

Type of drainage structure	Geometric design standard			
	DC5	DC6	DC7	DC8
Gutters and inlets	2	2	2	1
Side ditches	10	5	5	2
Ford (stone drift)	10	5	5	2
Drift or vented drift (1)	10	5	5	2
Culvert diameter <2 m	15	10	10	5
Large culvert diameter >2 m	25	15	10	5
Gabion abutment bridge	25	20	15	-
Short span bridge <10 m	25	25	15	-
Masonry arch bridge	50	25	25	-
Medium span bridge (15-50 m)	50	50	25	-
Long span bridge >50 m	100	100	50	-

Source. Adapted from international review.
Note 1 A drift and a vented drift are designed to be overtopped safely, hence the design is usually based on higher level of risk (shorter storm return period) than for culverts.

Table 11-3: Storm design return period (years) for severe risk situations

Type of drainage structure	Geometric design standard			
	DC 5	DC6	DC7	DC8
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Ford (stone drift)	15	10	10	5
Drift or vented drift	15	10	10	5
Culvert diameter < 2 m	25	20	20	10
Large culvert diameter >2 m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge <10 m	50	50	25	-
Masonry arch bridge	50	50	25	-
Medium span bridge (15-50 m)	100	100	50	-
Long span bridge >50 m	100	100	100	-

Source. Adapted from international review.

If the maximum water cannot be reasonably estimated it may be necessary to provide a structure that can be over-topped during periods of unpredicted water flow.

11.3 METHODS OF DESIGN

11.3.1 General

Water crossing structures must be designed to have a capacity equal to or greater than the maximum water flow that is expected in the water course. This maximum flow depends on the characteristics of the storm itself, namely the intensity, duration and the spatial extent of the rainfall, and the characteristic of the ground, or catchment, on which the rainfall falls.

The area of the drainage catchment (A) should be estimated from topographical maps or through the use of aerial photographs. The following methods are used for estimating maximum flow in a watercourse:

1. Direct observation of the size of watercourse, erosion and debris on the banks, history and local knowledge.
2. Replicating successful practice.
3. The Rational Method – for estimating peak discharges for small drainage areas up to about 100 hectares (larger catchments can be considered using modified Rational Methods, for example, using the areal reduction factor shown in Equation 11-3 below, or the East African Flood Model following MOWTC practice).

11.3.2 The Direct Observation Method

The cross-sectional area of the watercourse and the high water levels at the design storm level are required and the cross-sectional area of the apertures of the structure should then be designed to be equal to that of the storm design flow.

It may be possible to observe previous high water marks from existing structures, trees or other vegetation near the watercourse. Small debris floating down the river will be caught on branches and twigs during floods and indicate the water level during a flood. The highest flood is the most likely to be visible because it will often obliterate evidence of smaller flood tide marks. The problem is that there is often no indication of how old the flood level indicators are and hence what the return periods will be. The evidence of higher floods in the past may have been removed by natural weathering. This method will therefore give an indication of a recent high flood level but it does not guarantee to be the highest expected flood level. The information gathered by observation may be supplemented by interviews with local residents especially the more senior members of the community.

If there are people living near the proposed crossing point it will be possible to ask them how high the water level has risen in previous floods. A number of people should be questioned as memories 'fade' over time. It may be possible to ask people individually how high the biggest flood had been over the previous years and then take an average of the results obtained. Validation may be improved if enquiries are made for each riverbank independently and for different locations along the banks that provide information that can be correlated. Alternatively a group may be asked to collectively agree the maximum height of the floodwater. It will also be necessary to ask how often floods of the maximum size occur in order to determine the return period.

11.3.3 Replicating Successful Practice

If a high proportion of structures along a road or in a region have been in operation for a number of years without overtopping, it is reasonable to assume that the relationship between catchment area, catchment characteristics, rainfall intensity and maximum water flow used in their design is valid. The design of new structures can be based on simply the catchment area using the same relationships.

11.3.4 The Rational Method

The flow of water in a channel, q, is calculated from equation 11-1.

$$Q = 0.278 \times C \times I \times A \quad (m^3/s)$$ ....Equation 11-1

Where:

C = the catchment run-off coefficient

I = the intensity of the rainfall (mm/hour) for the Tc (time of concentration of the catchment area)

A = the area of the catchment (km^2)

The Catchment Run-off Coefficient, C, is obtained from Table 11-4.

Table 11-4: Run-off coefficient

CT (slope-topography		C_S (soils)		C_V (vegetation	
Very flat (<1%)	0.03	Sand and gravel	0.04	Forest	0.04
Undulating (1-5%)	0.08	Sandy clays	0.08	Farmland	0.11
Hilly (5-10%)	0.16	Clay and loam	0.16	Grassland	0.21
Mountainous (>10%)	0.26	Sheet rock	0.26	No vegetation	0.28
Runoff coefficient = $C_T + C_S$ +CV					

Source. Adapted from Robinson and Thagesen (2004)

The intensity of rainfall (I) is obtained from Intensity-Duration-Frequency (IDF) charts usually developed by the National Meteorological Agency. Such charts vary across the country and locally derived charts should ideally be used. In many situations these will not be available (because rain stations often measure only the rainfall in 24 hours) hence the engineer will need to rely on less accurate data.

The time taken for water to flow from the farthest extremity of the catchment to the crossing site is also required. This is called the Time of Concentration (Tc) and the duration of the storm must be set equal to this value because this will give the maximum flow rate, q, required.

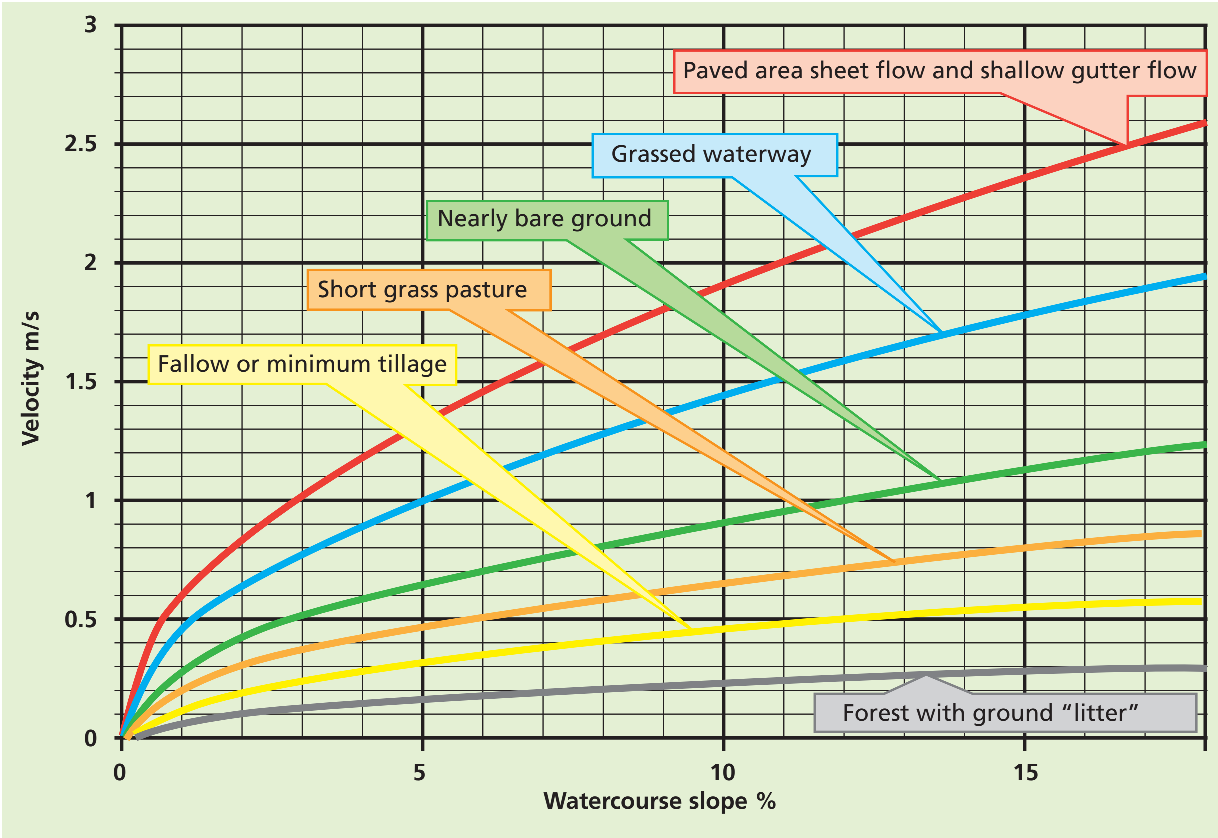
Tc = Distance from farthest extremity (m) / Velocity of flow (m/s)..........Equation 11-2

The velocity of flow depends on the catchment characteristics and slope of the watercourse. It is estimated from Figure 11-1.

The storm design return period is taken from Table 11-2 or Table 11-3.

In the Rational Method it is assumed that the intensity of the rainfall is the same over the entire catchment area. The consequence of applying the method to large catchments greater than 80 hectares is an over-estimate of the flow and therefore a conservative design.

A simple modification can be made to take into account the spatial variation of rainfall intensity across a larger catchment. The effective area of the catchment is reduced by multiplying by the areal reduction factor (ARF) given by the following equation:



Source. FHWA. Hydraulic Engineering Circular No. 19. (1984).

Figure 11-1: Velocity of flow for varying surface cover

$$ARF = 1 - 0.04 \times t^{-1/3} \times A^{1/2} \quad \text{.......Equation 11-3}$$

Where:

t = storm duration in hours

A = catchment area in km^2

This modification allows the catchment size limit to be increased considerably. Secondly, in rural areas where the catchment is often relatively simple (simple in terms of the complexity of the ground cover) the accuracy of the method is increased. However, the run-off coefficient also depends on the existing moisture conditions in the soil and on the storm intensity. When the ground conditions are wet or the storm intensity being used for design is high, the effective value of the run off coefficient will increase considerably, compensating to some extent for the reduction in run off caused by a larger catchment area.

11.4 FLOW VELOCITY

11.4.1 General

It is also important to determine the velocity of the water flow during peak flows because this affects the amount of scour that can be expected around the structure and hence the protective measures that may be required. The velocity can be measured in two ways.

11.4.2 Direct Observation in Flood Conditions

An object which floats, such as a small stick, should be thrown into the flow upstream of the potential structure. The time it takes to float downstream a known distance (e.g. about 100 m) should be measured. The velocity can then be calculated by dividing the distance the floating object has travelled by the time taken. This exercise should be repeated at least 3 times, but preferably 5 times, to get an accurate result. Tests where the floating object is caught on weed or other debris in the water should be discarded. The opportunities for making such observations during flood conditions are obviously very limited.

11.4.3 Manning's Equation

Design volumes of run-off in side drains and other channels can be estimated using the Rational Method. The cross sectional area of the drain must be sufficient to accommodate the expected flow of water, Q,

Where:

$Q = A.V$

The flow velocity is calculated from the Manning equation:

$V = 1/n\ R^{2/3}\ S^{1/2}$ and $Q = A\ V$...Equation 11-4

Where:

V = cross sectional average velocity in m/s

A = cross-sectional area of water (m^2)

R = hydraulic depth (area for the stream flow divided by the wetted perimeter) (Figure 11-2).

S = hydraulic gradient (slope of the drain or watercourse)

n = roughness coefficient (Table 11-5)

Q = discharge volume flow rate (m^3/sec)

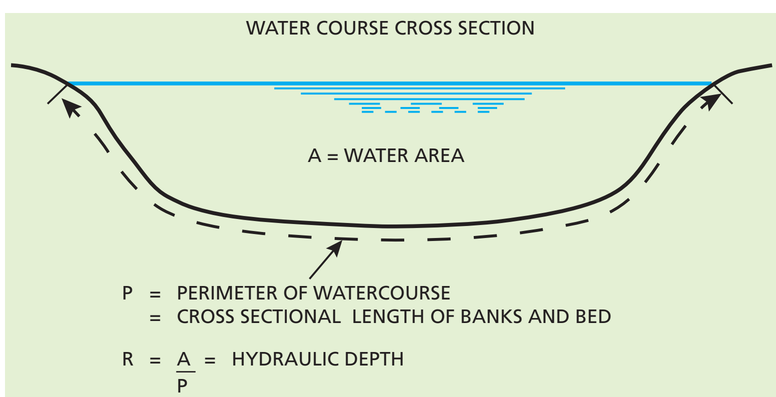


Figure 11-2: Definition of hydraulic depth in Manning's Formula

Table 11-5: Roughness coefficient (n) for drain

Material in the drain	Roughness coefficient
Sand, loam, fine gravel, volcanic ash	0.022
Stiff clay	0.020
Coarse gravel	0.025
Conglomerate, hard shale, soft rock	0.040
Hard rock	0.040
Masonry	0.025
Concrete	0.017

Source. FHWA (1984).

11.5 WATER CROSSINGS AND ASSOCIATED STRUCTURES

11.5.1 General

The policy of the road or local government authorities is invariably to provide as much all-weather access to as many people as possible, hence priorities need to be set. When selecting which roads and which potential structures to address and the type of structure to be constructed, the spot improvement and staged construction approach should be used and a cost benefit analysis carried out. A typical provision rate for culverts in rolling terrain is typically about 2 or 3 per km. In severe terrain or in flat, floodable areas, the frequency will be higher. The cost of providing adequate cross drainage is therefore an important component of road costs.

11.5.2 Type of Structure

The greatest potential cost savings for water crossing options is in the choice of structure type. This Chapter considers different water crossing options, from drifts to small bridges with spans of <10 m, explaining the characteristics of each, the conditions suitable for their use and the advantages and disadvantages associated with each structure.

The design of each structure to cope with the design storm water flow depends on the type of structure and its location and is described in the relevant Sections below. In general, for any structure that might constrict the water flow, e.g. culverts, the size of the aperture through which the water flows should be sufficiently large to prevent water from backing up on the upstream side. This is particularly important in locations where the water flow is relatively fast and therefore able to cause serious erosion and scour if it not adequately controlled.

At the most basic level, a simple ‘drift’ can be created in a stable sandy bed of an occasional watercourse by burying stones of 15 - 30 cm size just below the surface and covering them with sand. This substantially improves the bearing capacity for vehicles.

While structure types are presented individually it should be remembered that, for alluvial plains, a combination of a number of structures, perhaps constructed under a staged construction approach, might be the most cost effective means of spanning the watercourse.

11.5.3 Location of Structure

The following questions should be asked:

- Is the crossing site on a curve in the water course? If so, the water course may move outwards and damage the structure. If the road crosses the water course at an angle of 90°, construction will be cheaper and there will be a reduced risk of erosion and damage.
- Are there signs of erosion or sedimentation? If so, it is probable that these will get worse and damage or block the structure.
- Is the water course in strong bedding material? If so, there is less risk of erosion and undercutting.
- Is the water course straight at the point of crossing? If so, construction of the structure will be cheaper.
- Does the water course flood the surrounding land? If so, an embankment will also be needed across the area.
- Are there strong soils or rocks on both sides of the water course? If so, the structure will be able to support the weight of vehicles passing on the road.
- Is the water course in a deep valley? If so, the approaches can be very steep and at risk of erosion and failure.

After considering these questions, the engineer may decide that it is cheaper to realign the road to a more suitable site.

11.5.4 General Considerations

Traffic and vehicle loading should also be considered when choosing the type of structure. Some types of structure are particularly prone to damage caused by grossly overloaded vehicles at some locations. For example, the durability of corrugated steel or concrete pipe culverts on unpaved roads will be greatly affected when overloaded because loss of fill over the culvert pipe will occur with eventual direct loading for which the culvert pipe was not designed and cannot withstand.

If the loading capacity is limited in any way, signage should be provided to clearly state the loading capacity of the structure. Local road network managers and administrators should also be made aware of any load limitations and the likely consequences of these being exceeded.

If it is not possible, with the resources available, to construct a crossing that will withstand the largest vehicle that could travel down the road, it will be necessary to install a robust non-removable barrier each side of the structure to prevent overloaded vehicles from crossing

When the structure is being designed, the size (dimension) of the vehicle should also be taken into consideration to ensure that it can safely cross the structure without damage to the vehicle or structure. The width of a structure substantially influences the initial construction cost. For bridges the cost is roughly proportional to deck area and for culverts, roughly proportional to the length of the barrel.

Widening most structures in the future is often an expensive process hence, with potentially high traffic growth rates, a vital decision is required concerning whether the structure is to be designed for one or two-way traffic flow. It is probable that two-way traffic for bridges will only be justifiable for the highest category of LVR although local conditions may override this. The secondary decision is with respect to the safe width for the predominant traffic type. These decisions become more important with the increasing size of the proposed structure.

It should be noted that a culvert or other drainage structure is required at all low points of the road alignment. The cost of their provision is usually significant in the overall cost of a low volume road, particularly for unpaved roads. The frequent occurrence of culvert headwalls and width narrowing, and the difficulty for drivers to see them in advance, particularly when travelling at night, raises important safety issues. The provision of minimum two-lane width culverts can therefore be justified in most cases (except for the most severely constrained projects). Culvert headwalls should be set back behind the carriageway and shoulder, and clearly marked or have guide posts at each end of the culvert to prevent vehicles driving into the inlets, outfalls or ditches when passing on-coming traffic. These requirements may be relaxed to provide only clear carriageway width in slow speed mountainous alignments.

For larger structures restricting access to one lane is justified. For single-lane motor vehicle traffic the clear carriageway width (between kerbs, parapets, guardrails or marker posts) should be a minimum of 3.75 m. This width should allow easy single way traffic but clearly restrict two vehicles from passing on the structure at the same time. To accommodate motorcycles as well as a vehicle a minimum of 4.5 m is required. Thus, in view of the rapid increase in motor cycle traffic that is occurring, it is usually prudent to use 4.5 m.

Where justifiable, full two-lane motor traffic provision should allow a minimum of 6.5 m between kerbs provided that vehicles are restricted to slow speed passage.

Where physical restrictions are necessary to prevent passage of heavy good vehicles these will need to limit free passage to about 2.3 m. This requires a clear indication that the roadway narrows (advance warning signs).

11.5.5 Design for Climate Resilience

Climate change will affect roads and highways in many different ways. The accepted characteristics, amongst others, are higher temperatures, higher rainfall, more intense storms and more frequent storms. This will lead to the need to cope with generally more water, more frequent floods, and faster and more destructive water velocities. Thus, much of the historic data on which hydrological analysis and hydraulic design relies, may lead to an under estimation of design floods and high water levels. Until new flood models are developed and verified, one of the simplest and important actions that can be taken is to design drainage structures based on estimates of storm characteristics with currently higher return periods as in Table 11-3 for severe risk situations. This is essentially increasing the safety factor. In addition there are various other strategies that will help to increase climate resilience. In general these comprise:

- Identifying the most vulnerable areas and essentially increasing the 'safety factor' inherent in their design.
- Ensuring that the drainage systems are well maintained and functioning correctly.
- In critical areas or high priority roads where the consequences of failure and closure are more severe, local realignment, if appropriate, may be required, but this will usually only be considered as part of an emergency repair, rehabilitation or upgrading project after storm damage has occurred.

Increasing the safety factor includes using drifts and vented drifts that can be safely overtopped instead of culverts that can become blocked by debris; adding additional protection to culverts that might be blocked by debris; better surface drainage so that water is dispersed off the road more frequently; reducing water concentration by means of additional cross drains and mitre drains to lower the volume of water that each one needs to deal with with *(ref. Chapter 12 – Drainage and Erosion Control).*

Erosion is a serious problem in many areas and adverse climate change and deforestation will make matters worse. Erosion is discussed in *Chapter 12 – Drainage and Erosion Control*. There are also likely to be more severe geotechnical problems (e.g. slope stability) caused by climate change and these are dealt with in *Chapter 5 – Geotechnical Investigations and Design.*

Ensuring that the drainage system is working correctly is essentially a maintenance issue although there will be examples of poorly designed culverts with improper alignment or grade relative to the channels and ditch lines that will need to be repaired or replaced, usually after failures have occurred.

11.6 LOW LEVEL WATER CROSSINGS

11.6.1 General

A low level water crossing is simply one that is designed to be over-topped. The most common is a drift which is constructed from stones or concrete. The simplest is a ford or splash which consists of unbound hand packed stone. Drifts and fords are often dry and cater for relatively low water flows. For higher water flows of longer duration a vented drift is more appropriate. This is essentially a drift constructed on top of a series of culverts thereby allowing considerable water to flow before being over-topped only during severe storms. For such structures that are designed to be overtopped, the culverts should be designed in the normal way using the nomographs shown in Section 11.7.4. The ability to be over-topped with little risk of failure is a relatively inexpensive way to reduce risks, especially for wide river crossings subject to unpredictable flash floods.

11.6.2 Drifts

A drift consists of a flat slab and two inclined approach ramps over which water and vehicles can pass, thus a drift carries water over the road. Drifts are the cheapest form of watercourse crossing. They are also referred to as Irish bridges, fords or splashes. The terms describe essentially the same structure but it is generally accepted that a ford or splash is constructed from the existing riverbed whereas a drift is a ford or splash with an improved running surface constructed from imported materials.

Drifts are suitable for shallow water courses with a gentle gradient and at sites where raising the road over a culvert would require the transport of large quantities of earth.

There are two types of drift:

Relief drifts: These relieve side drains of water where the road is on sloping ground and water cannot be removed from the uphill side drain by mitre drains. It is an alternative to a relief culvert.

Small watercourse (or stream) drifts: Where stream flows are very low with normal water depth of less than 200 mm (or perennial) drifts may be used to allow the stream to cross the road, as illustrated in Figure 11-3.

11.6.3 Key Features of Drifts

The key features of drifts are:

- Stream drifts are structures which provide a firm place to cross a river or stream. Relief drifts transfer water across a road without erosion of the road surface. Water flows permanently or intermittently over a drift, therefore vehicles are required to drive through the water in times of flow.
- Drifts are particularly useful in areas that are normally dry with occasional heavy rain causing short periods of floodwater flow.
- Drifts provide a cost effective method for crossing wide rivers which are dry for the majority of the year or have very slow or low permanent flows.
- Drifts are also easier to maintain than culverts and will also act as traffic calming measures.
- Drifts are particularly suited to areas where material is difficult to excavate, thus making culverts difficult to construct.
- Drifts are also particularly suited in flat areas where culverts cannot be buried because of lack of gradient.
- The drift approaches must extend above the maximum design flood level flow to prevent erosion of the road material.
- If necessary, guide posts must be provided on the downstream side of the drift and be visible above the water when it is safe for vehicles to cross the drift.
- Buried cut-off walls are required upstream and downstream of the drift to prevent under cutting by water flow or seepage.
- The approach road level will normally mean that approach ramps are required. Approach ramps should be provided to the drift in the bottom of the watercourse with a maximum gradient of 10% (7% for roads with large numbers of heavy trucks).
- Drifts should not be located near or at a bend in the river.
- Some form of protection is usually required downstream of a drift to prevent erosion

Posts are fixed to indicate the water level and edges of the drift when water is flowing over it. It is also possible to construct guide blocks along the sides of the drift to help pedestrians to pass when water is flowing. Stones are placed in the water course (apron) to prevent erosion downstream and upstream of the drift.

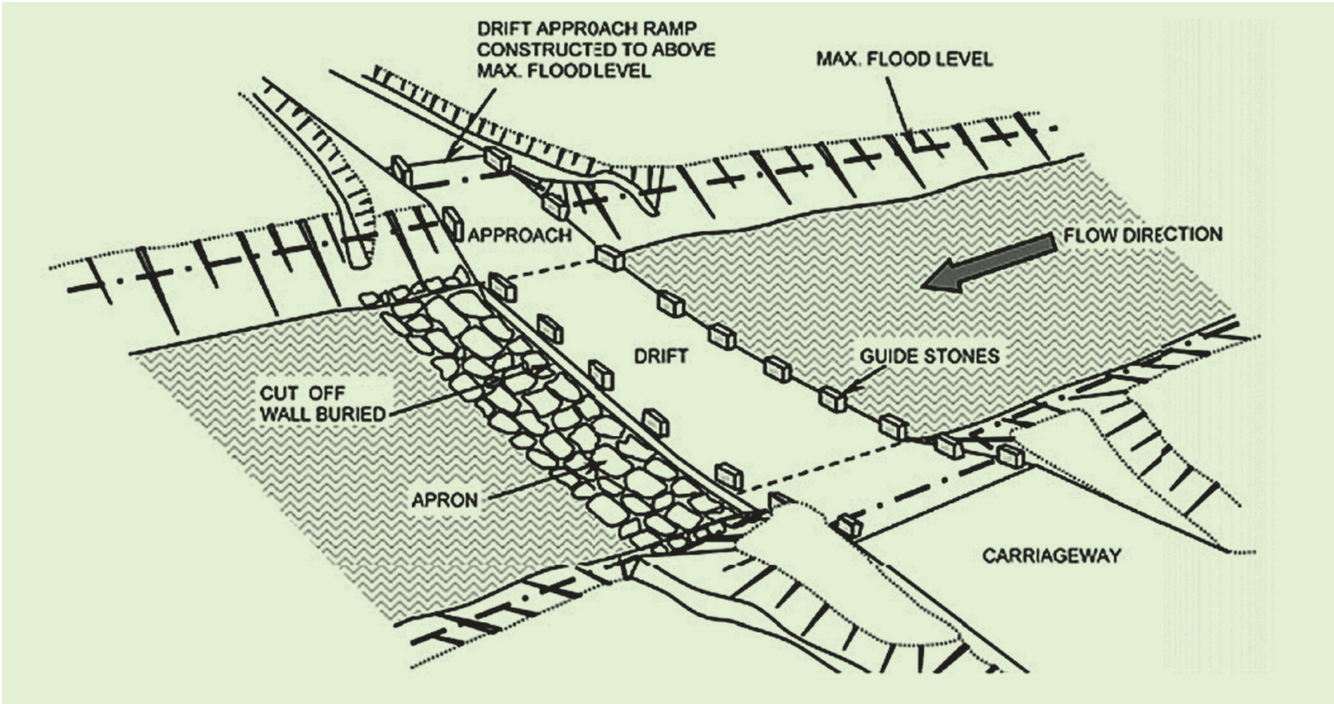


Figure 11-3: Key features of a stream drift

Drifts should be constructed with a shape as close as possible to the shape of the existing water course. The slab should be at the same level as the bed of the water course and road cross fall should not be more than 2%. If the river bed gradient is more than 2%, a stepped structure (cascade) should be used at the outlet. The extent of the drift is normally calculated using the Observational Method. Drifts should cover the entire width of the water course when water is flowing. It is also possible to estimate the width and depth of the drift using the Rational Method. The ramps should extend at least one metre beyond the required high water point. The cost of a drift is normally estimated per metre length.

The slab and the ramps should be durable and non-erodible. They can be made from hand placed stone or concrete. When concrete is used, it should be reinforced with steel mesh (6 mm bars laid in a 150-200 mm grid). Construction joints should be provided so that each slab is no more than 5 m long.

Vehicles that pass over a drift can spread water on the approach roads. This can make the road surface slippery or suffer from erosion, especially if the approach road is steeper than 5% and if the water flows over the drift for more than two days after rain. In this case, the road should be gravelled or provided with an improved surface for 50 m in each direction.

The following criteria should be considered when designing drifts:

- The level of the drift should be as close as possible to the existing river bed level.
- The normal depth of water should be a maximum of 200 mm.
- Approach ramps should have a maximum gradient of 10% (7% for roads with large numbers of heavy trucks).

11.6.4 Advantages and Disadvantages of Drifts

The advantages and disadvantages of drifts are summarized in Table 11-6.

Table 11-6: Advantages and disadvantages of drifts

Advantages	Disadvantages
• Low cost: at the most basic level - can be constructed and easily maintained entirely with local labour and materials. • Volume of excavated material in most cases is small. • Drifts do not block with silt or other debris carried by floodwater. • They can accommodate much larger flows than culverts. • Easier to repair than culverts. • Water flows over a wide area, resulting in less water concentration and erosion downstream than piped culverts.	• The crossing can be impassable to traffic during flood periods. • Foot passage can be inconvenient or hazardous when water is flowing. • Drifts require vehicles to slow down when crossing. This could be considered an advantage because of the traffic 'calming' effect.

11.6.5 Splash

A splash is a type of low cost drift. It consists of a shallow channel which is protected against erosion and which passes across a track. Splashes are recommended for low water volumes. The surface of the channel is protected by a material which is low cost and non-erodible, such as a layer of flat stones. Most low cost surfaces are porous and should not be used if the water flows for more than three hours after rain. A splash is suitable for intermittent water courses across tracks.

When the water course is flowing, all the water should pass between the edges of the splash in order to prevent damage to the surface of the track. Normally the maximum length (in the direction of the road) of a splash is 5 m.

Splashes can also be used to reduce the flow in a side drain, in the same manner as a relief culvert.

11.7 CULVERTS

11.7.1 General

Culverts are usually constructed in narrow well defined water courses but they can also have many apertures in order to cross wide and shallow water courses. Culverts perform two basic functions.

Relief culverts

These are placed at low points in the road alignment (where there is no definable stream) or along long downhill gradients, but the topography of the ground requires a significant amount of cross drainage which cannot be accommodated by side drains, as illustrated in Figure 11-4. A relief culvert should be located at the point where the high volume of water starts to cause erosion or the drain to overtop. Relief culverts should be used only when solutions such as regular drain clearing to maintain and ensure maximum flow and use of a mitre drain are not possible.

Stream culverts

These allow a watercourse to pass under the roadway.
Culverts can be pipe, box, slab or arch type, round, elliptical and square.

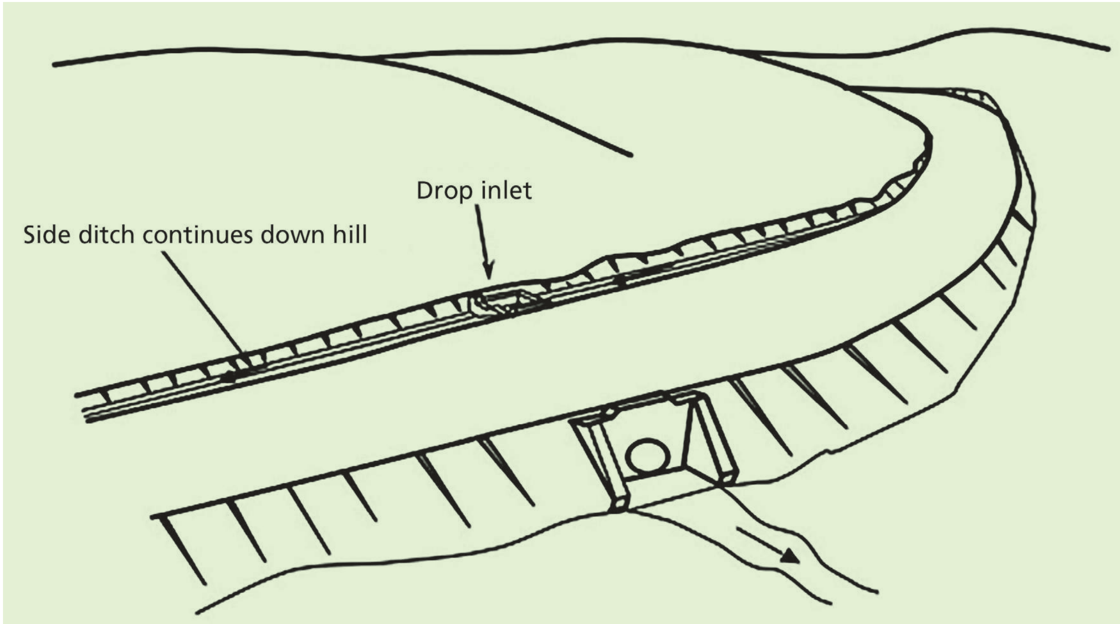


Figure 11-4: Key features of a relief culvert

11.7.2 Key Features of Culverts

The key features of culverts are:

- Culverts are the most commonly used drainage structures on low volume roads. They can vary in number from about two per kilometre in dry and gently rolling terrain up to six or more for hilly or mountainous terrain with high rainfall. In flat areas with high rainfall, the frequency may also be increased to allow water to cross the road alignment in manageable quantities.
- In addition to well-defined water crossing points, culverts should normally be located at low points or dips in the road alignment.
- Relief culverts may be required at intermediate points where a side drain carries water for more than about 200 m without a mitre drain or other outlet.
- Headwalls are required at the inlet and outlet to direct the water in and out of the culvert and prevent the road embankment from eroding (sliding) into the watercourse. Wing walls at the ends of the headwall may also be used to direct the water flow and retain the material of the embankment or inner ditch slope.
- Aprons with buried cut off walls are also required at the inlet and outlet to prevent water seepage, scouring and undercutting.
- Culvert alignment should follow the watercourse both horizontally and vertically where possible.
- The gradient of the culvert invert should be between 2 and 5%. Shallower gradients could results in silting whereas steeper gradients result in scour at the outlet because of high water velocity.
- Culvert invert levels should be approximately in line with the water flow in the streambed, otherwise drop inlet and/or long outfall excavations may be required.
- Cross culverts smaller than 750 mm in diameter should not be installed, as they are very difficult to maintain (clean). A culvert of 900 mm is preferred from a maintenance perspective but extra cover is required which may result in humps on the road alignment.

- Where foundation material is poor, culverts should be placed on a good foundation material or raft foundations to prevent settlement and damage. On very soft ground, it may be necessary to consider concrete, steel or timber piles to provide adequate foundations. This will require specialist design expertise not covered by this Manual. It is necessary to protect the watercourse from erosion downstream from the structure.
- Culverts can exist in pairs or in groups to enable larger stream flows to be accommodated using standard unit designs.
- When silt supply is high, culverts may need to be installed at higher gradient or extra maintenance may be required.

11.7.3 Advantages and Disadvantages of Culverts

The advantages and disadvantages of culverts are summarized in Table 11-7.

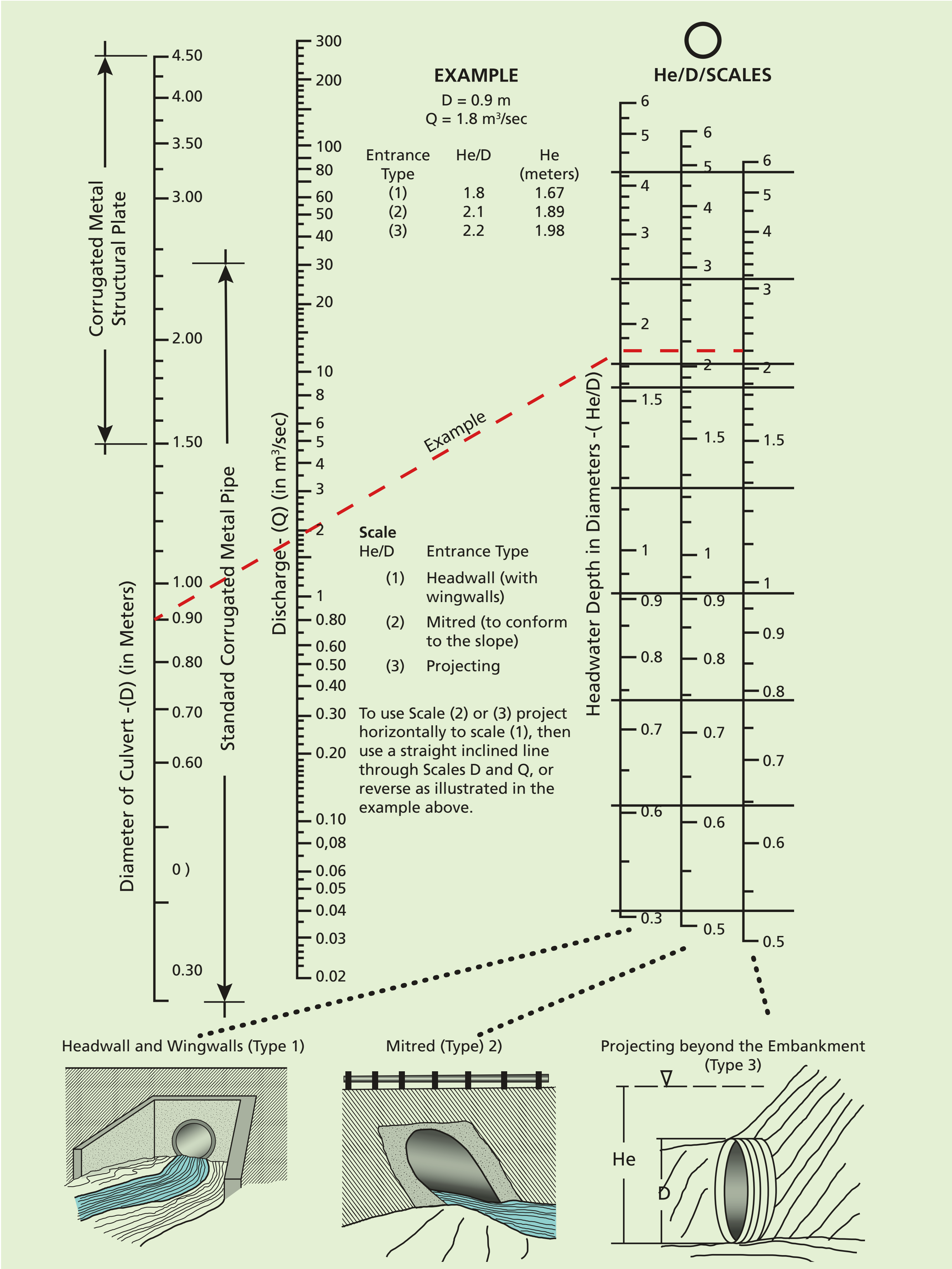
Table 11-7: Advantages and disadvantages of culverts

Advantages	Disadvantages
• Culverts provide a relatively cheap and efficient way of transferring water across a road. • They can be constructed and maintained with local labour and local materials. • Culverts allow vehicle and foot passage at all times. • Culverts do not require traffic to slow down when they are crossed but humps above culverts can also be used for traffic calming if drivers are provided with appropriate warning signs. • Culverts allow water to cross the road at various angles to the road direction for a relatively small increase in costs.	• Regular maintenance is often required to prevent the culvert silting up, or to remove debris blockage. • Culverts act as a channel, forcing water flow to be concentrated, so there is a greater potential for downstream erosion compared with drifts. • Culverts are not suited to occasional high volume flows.

11.7.4 Design of Culverts

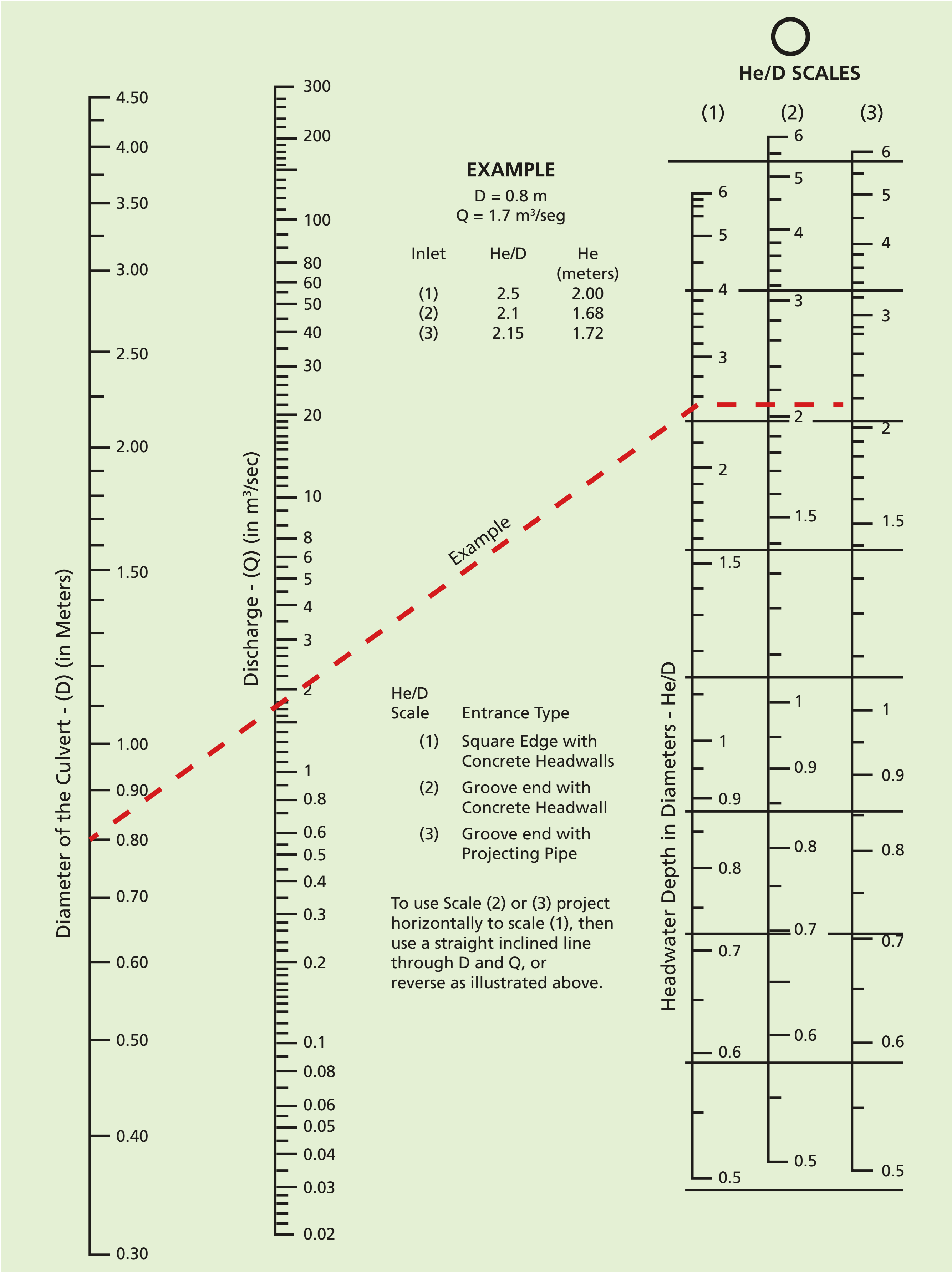
The water flow through the culvert should be estimated by any one of the methods presented in this chapter. The required size of a culvert opening is estimated using the nomograms in Figure 11-5 for corrugated metal pipes, Figure 11-6 for concrete pipes and Figure 11-7 for concrete box culverts. These figures apply to culverts with inlet control where there is no restriction to the downstream flow of the water.

The nomograms are used by identifying the value for the flow of water generated by the design storm on the middle scale and drawing a line from that point across to the left hand scale of the three scales on the right labelled H/D. These scales are for the three types of inlet shown on the nomograms. H/D is the ratio of the maximum head of water to the diameter of the culvert opening that is required to discharge the design flow through the culvert. In general, risks are reduced if the maximum flow does not cause the culvert to run at maximum capacity except for the design storm. Finally the line is extended to the left to intersect the line labelled ‘Diameter of Culvert (D) in m’. The culvert size obtained from the nomogram will probably not be one of the standard sizes that are available hence the next higher available size should be chosen or the nearest available size if the difference is small (<10 % in diameter).



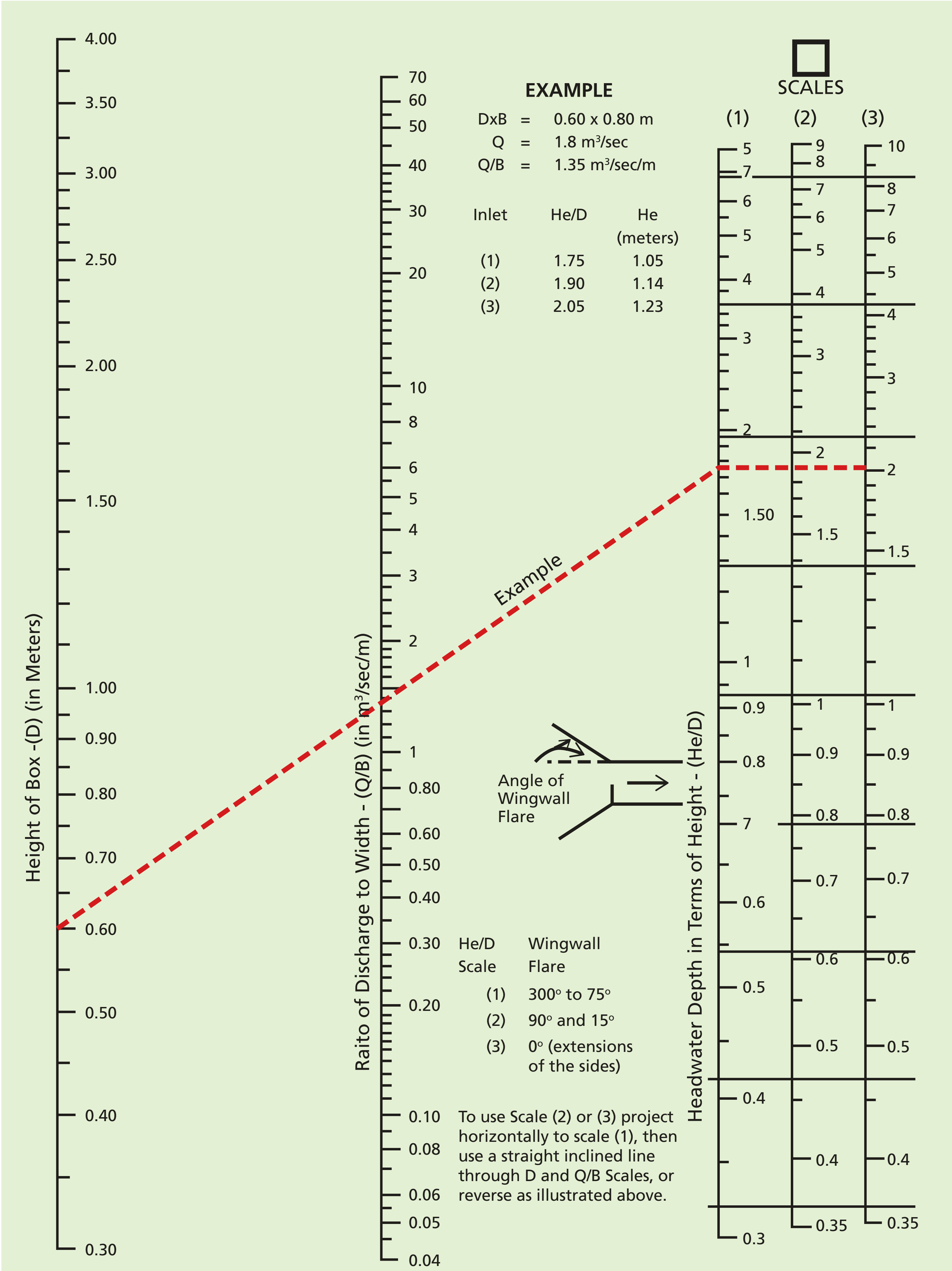
Source: FHWA (2012).

Figure 11-5: Headwater depth and capacity for corrugated metal pipe culverts with inlet control.



Source: FHWA (2012).

Figure 11-6: Headwater depth and capacity for concrete pipe culverts with inlet control



Source: FHWA (2012).

Figure 11-7: Headwater depth and capacity for concrete box culverts with inlet control

In flat terrain, where there is a high risk of silting, as shown in Figure 11-8, a factor of safety of 2 (in terms of water flow capacity) should be allowed in the design of the culvert. To minimise the effect of potential silting and deposition of debris in the culvert, the slope/fall should be 3-5%.

Figure 11-8: Siltation problems

11.7.5 Construction of Culverts

Culverts can be constructed in various ways, depending on the materials and the available skills. Inlet and outlet structures are normally constructed with wet stone masonry. Options for the barrel of the culvert include:

- Walls constructed in wet masonry with a slab of reinforced concrete. This type can have a height up to 1.5 m and a width up to 2 m.
- Arch constructed in wet masonry, using a variety of temporary supports.
- Concrete tubes, constructed alongside the site using a collapsible mould or bought from local suppliers if the quality is acceptable.
- A metal arch or tube.

In areas where it is difficult to obtain stones or rocks, culverts can also be constructed using sacks filled with a mixture of sand and cement.

Culverts should be designed to discharge the peak flow of water for the chosen design period, overtopping only during exceptional rains with a return period in excess of the chosen design period. Therefore the most important aspect of culvert design is the open area through which the water flows. The required open area is calculated using the methods described in Section 11.3 above and the nomographs of Section 11.7.4.

The minimum recommended size of a culvert opening is a tube of diameter 750 mm. This diameter allows a worker to enter the tube during maintenance to remove obstructions. A greater open area can be obtained by using larger tubes, more tubes or different types of aperture. Larger apertures are more efficient in material usage but can require a deeper channel or more fill to carry the road over the culvert. The use of large diameter tubes can require special equipment to transport and lift them into place.

The invert of the culvert at the outlet must be at the same level as the bed of the outlet channel. However, it is possible to lower the invert of the culvert and the bed of the outlet channel by 300 mm if the latter is no more than 20 m long.

If the construction of the culvert requires the road to be raised locally in order to provide sufficient cover above the top of the culvert, humps in the road will be created. To avoid this a drift could be used rather than a culvert, but this will be more expensive. Alternatively, a wider stone masonry culvert could be used to provide the same capacity as a 750 mm culvert. In general, if two solutions are equally valid and acceptable from an engineering and safety point of view, then the least expensive option should be chosen.

When culverts are located on earth roads, it is recommended that a layer of gravel or other improved surface is placed over the culvert and for 20 m on either side of the approaches.

11.8 VENTED DRIFTS AND CAUSEWAYS

11.8.1 General

A vented drift is a combination of a culvert and a drift. They are suitable for carrying roads across water courses which have a perennial (permanent) water flow for most of the year and which have large flows for less than three days after heavy rains.

11.8.2 Key Feature of Vented Drifts and Causeways

A typical example of a vented drift is shown in Figure 11-9 (schematic) and 11-10 (photo). The key features are:

- These structures are designed to pass the normal dry weather flow of the river through pipes below the road. Occasional larger floods pass through the pipes and over the road, which may make the road impassable for short periods of time.
- Vented causeways are the same concept as vented drifts but are longer with more pipes to cross wider watercourse beds.
- The level of the road on the vented drift should be high enough to prevent overtopping except at times of peak flows.
- There should be sufficient pipes to accommodate standard flows. The location of pipes in the drift will depend on the flow characteristics of the river.
- A vented drift should be built across the whole width of the water- course.
- A vented drift requires approach ramps which must be surfaced with a non-erodible material and extend above the maximum flood level.
- Watercourse bank protection will be required to prevent erosion and eventually damage to the entire structure.
- The approach ramps should not have a steeper grade than 10% (7% where there is significant heavy vehicle traffic).
- The upstream and downstream faces of a vented drift require buried cut off walls (preferably down to rock) to prevent water undercutting or seeping under the structure.
- An apron downstream of the pipes and an area of overtopping is required to prevent scour by the water flowing out of the culvert pipes or over the structure.

- There is also a requirement to protect the watercourse from erosion downstream from the structure. There will be considerable turbulence immediately downstream of the structure in flood conditions.
- The road surface longitudinal alignment of a vented drift should be a slight sag curve to ensure that, at the start and end of overtopping, water flows across the centre of the vented drift and not along it.
- There should be guide stones on each side of the structure to mark the edge of the carriageway and indicate when the water is too deep for vehicles to cross safely.

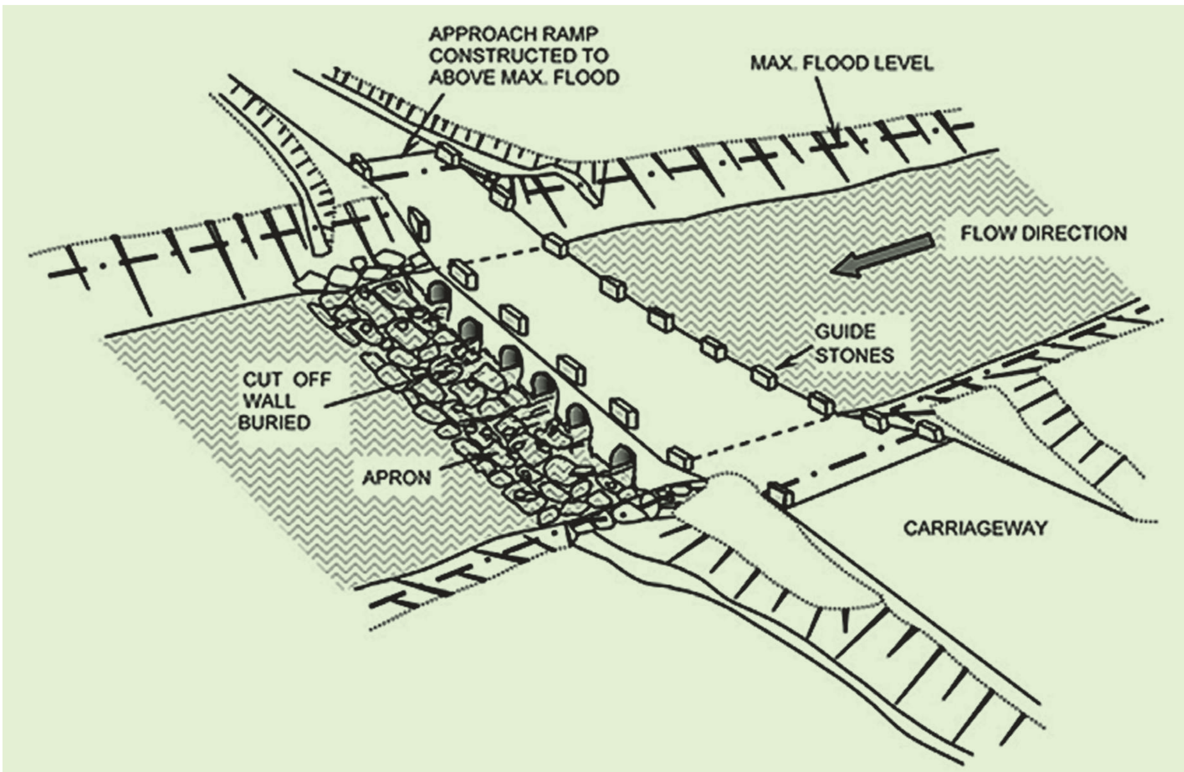


Figure 11-9: A typical vented drift / causeway (schematic)

Figure 11-10: A typical vented drift / causeway

11.8.3 Advantages and Disadvantages of Vented Drifts and Causeways

The advantages and disadvantages of vented fords and causeways are summarized in Table 11-8.

Table 11-8: Advantages and disadvantages of vented drifts/causeways

Advantages	Disadvantages
• Vented drifts can allow a large amount of water to pass without overtopping. • They are cheaper to construct and maintain than bridges. • Construction of vented fords is fairly straightforward compared with bridges. • Vented fords are well suited to cope with short high volume flows. • Can be constructed and maintained primarily with local labour and local materials.	• Vented drifts can be closed for short periods during periods of flooding and high flow. • Floating debris can lodge against the upstream side of the structure and block pipes. • Foot passage can be inconvenient or hazardous when water is flowing.

11.8.4 Design of Vented Drifts/Causeways

The apertures must allow normal flows hence the size of the apertures should be obtained by observing the flow under normal conditions (but not during the dry season). The apertures plus the area above the apertures to a depth of 200 mm must allow the water to flow during heavy rains. They can be designed using the Observational Method or the Rational Method. Vented drifts generally have higher capacity and construction costs than drifts or culverts.

The invert of the apertures should be at the same level as the outlet channel. Each ramp should extend, at a gradient of no more than 10%, to at least one metre above the highest water level observed during heavy rain. Gravel or an improved surface should also be placed on the road for 50 m in each direction.

So that vehicles can pass safely when water is flowing over the slab and the slab itself is not fully visible, posts are fixed to indicate the water level and the edges of the structure. It is also possible to construct blocks along the sides of the vented drift to help pedestrians to pass when water is flowing.

11.9 SUBMERSIBLE BRIDGE

11.9.1 General

A submersible bridge is a form of vented drift with large apertures for the normal water flow. During heavy rains the water can also pass over the deck and the two approach ramps that are constructed over the apertures.

Submersible bridges allow roads to cross water courses which have large flows for most of the year and very large flows lasting for up to a week during and after heavy rains.

Posts are fixed to indicate the width of the structure.

11.9.2 Features of Submersible Bridges

Submersible bridges are often designed using the Observational Method. The apertures and the area above the apertures up to a depth of 200 mm must be sufficient to allow the water to flow during heavy rains.

Each approach ramp should extend at least 2 m beyond the highest water level observed during heavy rains. The level of the deck should be sufficiently high to allow the normal flow of water to pass under the bridge.

It is preferable that all the pillars are seated on a rock foundation. If the bridge is fixed on a rock foundation with steel dowels, it can be up to 2.5 m high. If the bridge is not fixed with steel dowels, it can be up to 1.5 m high.

If it is not possible to provide sufficient open area with a submersible bridge of 1.5 or 2.5 m height, it is necessary to construct a high level bridge.

The structure should be inspected every year to check that maintenance and repairs are being carried out as required.

11.10 MASONRY ARCH CULVERTS

11.10.1 General

For high water flows a masonry arch culvert provides a greater capacity than a 'pipe' culvert and is sometimes a more appropriate option than a small bridge. A masonry arch culvert is illustrated in Figure 11-11. Their capacity is designed in a similar way to small bridges (for guidance on such structures refer to the Bridge Management System for Tanzania: Handbook for Bridge Inventory, Ministry of Infrastructure Development, (2007), and publications such as TRL Overseas Road Note 9.

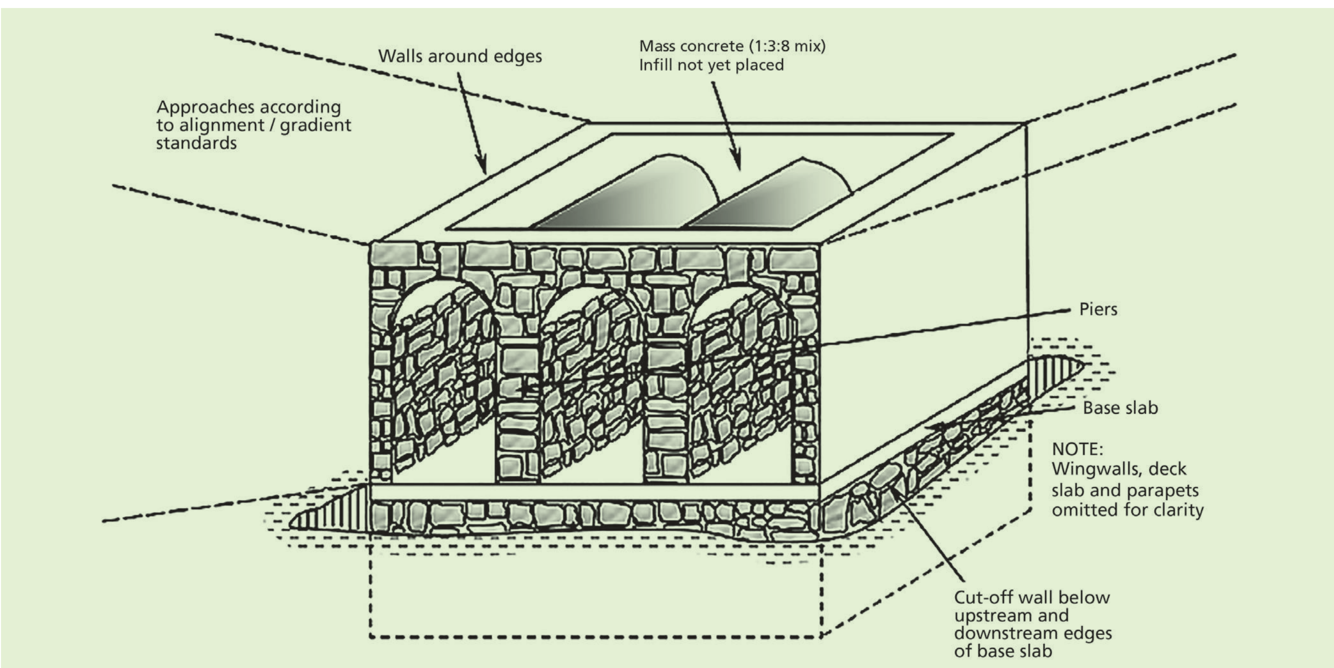


Figure 11-11: Key features of large arch culvert

11.10.2 Key Features of Masonry Arch Culverts

The key features of large arch culverts are:

- If a culvert is required to have a large flow capacity it would typically need a single barrel greater than 2 m in diameter or a number of medium sized barrels. Alternatively a masonry arch culvert could be used.
- Formwork is required to construct the openings. This formwork can be made from wood, stones or metal sheeting and either incorporated into the structure or removed once construction is complete.
- Although these structures are not generally designed to be overtopped, they can be designed and constructed to cope with an occasional overtopping flood flow.

- The road alignment needs to be a minimum of 2 m above the bottom of the watercourse.
- Approach embankments are required at each end of the structure.
- Large arch culverts require solid foundations with a buried cut-off wall on both upstream and downstream sides to prevent water seepage erosion and scouring.
- These structures require large amounts of internal fill material during construction.
- Guide stones or kerbs should be placed at the edge of the carriageway to increase vehicle safety.
- If the crossing is to be used by pedestrians, consideration should be given to installing guard rails and central refuges for long crossings where pedestrians can move off the roadway to allow traffic to pass.
- Water from the roadside drains should be carefully channelled into the watercourse away from the structure to prevent erosion of the bank or scour of the culvert structure.

11.10.3 Advantages and Disadvantages of Masonry Arch Culverts

The advantages and disadvantages of large arch culverts are summarized in Table 11-9.

Table 11-9: Advantages and disadvantages of large arch culverts

Advantages	Disadvantages
• Large arch culverts are usually easier and cheaper to construct than bridges. • They can accommodate flows significantly higher than smaller culverts and vented fords. • Can be constructed and maintained with local labour and materials, without the need for craneage. • They may easily be designed and constructed for occasional overtopping. • They generally require less maintenance than conventional bridges.	• The water opening in large arch culverts is smaller than for a bridge of the same size, which reduces the potential flow rate past the structure at peak flows. • Large arch culverts can require a significant amount of internal fill material.

An alternative to a large or multi-barrel culvert is a reinforced concrete box culvert. This Manual does not cover this type of structure. For guidance on such structures refer to the Bridge Management System for Tanzania: Handbook for Bridge Inventory, Ministry of Infrastructure Development, (2007), and publications such as TRL Overseas Road Note 9.

11.11 EMBANKED CROSSINGS

11.11.1 General

In completely flat terrain where a water course floods during the rains or with generally poor drainage the road will usually be on an embankment. Such an embanked crossing must have one or more cross drainage structures hence the design information for the cross drainage structures is as described in previous sections. Under these circumstances the flow can be relatively slow provided that enough culverts are available, but insufficient culverts can lead to rapid flow along the side of the embankment and consequent scouring. The simplest method of estimating this is by asking the local people how long the water usually takes to dissipate from peak flood condition after the rain. Calculating the likely volume and required number and size of culverts necessary to prevent the flow velocity exceeding the velocities shown in Table 11-15 is then relatively straight forward.

11.11.2 Construction

Embankments can normally be constructed with soils that are found near to the road. The soil that is used should form a strong and stable layer when compacted. Top soil and loose sand should be avoided.

The road surface on the embankment must remain dry all year round. Therefore the height of the embankment depends on the water level in the area. The top of the embankment should be at least 500 mm above the highest water level. In flat areas with poor drainage, it can be sufficient to construct an embankment only 500 mm high. The soil in the embankment should be spread in layers 100-150 mm thick layers and well compacted. The surface should be formed to give a camber. An improved surface should be provided, e.g. good gravel with a surface seal, to better support the weight of the vehicles and to protect the surface of the road against erosion. Grass should be planted or allowed to grow on the sides of the embankment in order to protect the sides against erosion. Rip rap may also be used for this purpose.

An embanked crossing normally needs structures to allow the water to flow. The required size of the apertures can be calculated using the Observational or Rational method. Structures should be constructed along the embankment at intervals of no more than 50 m. The deepest part of the crossing should have the largest structure.

Some embanked crossings, for example, across an area where a water course floods over the land, are constructed with a short length of the embankment at a lower level. The short lengths of embankment act as a drift and can prevent large pieces of debris from blocking the openings in the embankment when the water course is flooding. The part of the lower embankment should be protected against erosion when the water flows over them.

Over time, erosion and other factors can make the embankment narrow, requiring vehicles to pass dangerously close to the edge (i.e. within 500 mm). The embankment must be widened in the following manner:

- Decide if the embankment needs to be widened on one or both sides.
- Wait until the site is as dry as possible.
- Remove vegetation and loose soil from the side of the embankment to 1 m beyond the base.
- Cut steps in the embankment slope. Each step should have a horizontal surface at least 500 mm wide and should be cut into stable, well compacted soil.
- Place and compact suitable material in layers 150 mm thick to the required width of the embankment.
- Shape the slope and protect it with suitable vegetation.
- Reconstruct the road to its original width, with an improved surface such as gravel or Geo-Cells.

11.12 BRIDGES

11.12.1 General

These are generally the most costly structures. This Section covers arch and simply supported bridge types. The Manual does not cover large or multiple span bridges, which may be simply supported or continuous over piers. For such structures and bridges with spans more than 10 m, refer to the *Bridge Management System for Tanzania: Handbook for Bridge Inventory, Ministry of Infrastructure Development (2007),* and publications such as *Overseas Road Note 9 (TRL, 1992)*. The key features of a simply supported bridge deck is illustrated in Figure 11-12.

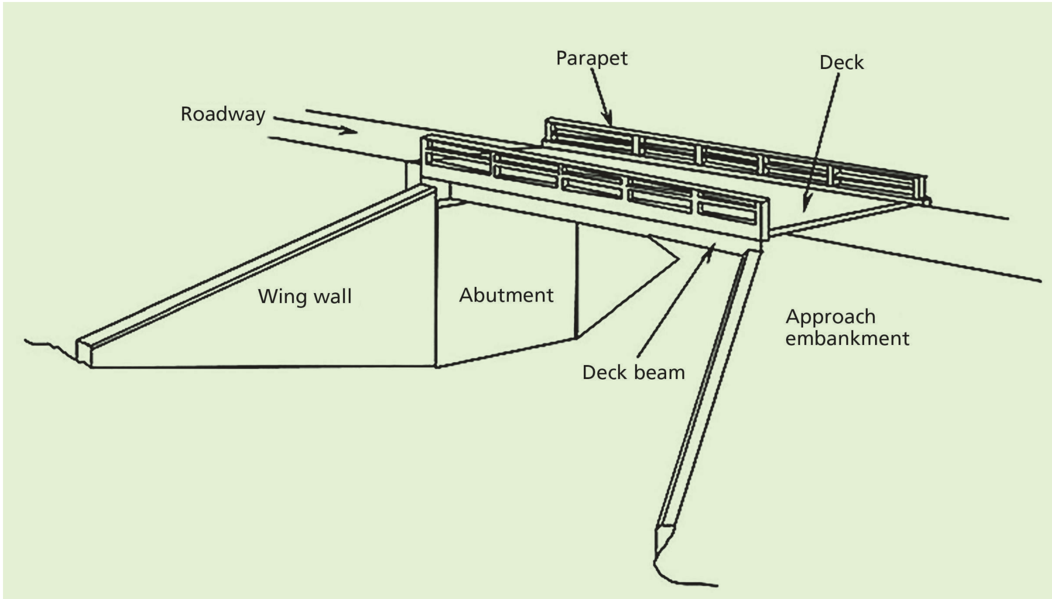


Figure 11-12: Key features of a simply supported bridge deck

11.12.2 Key Features of Small Bridges (< 10 m span)

Key features are:

- The arch is the simplest form of bridge.
- There are a number of different elements to a simply supported deck bridge. These are a superstructure (comprising deck, parapets, guide stones and other road furniture) and substructure (comprising abutments, wing walls, foundations, piers and cut off walls).
- Bridges are generally the most expensive type of road structure, requiring specialist engineering advice and technically approved designs.
- Bridges can be single span or multi span, with a number of openings for water flow and intermediate piers to support the superstructure.
- The main structure is always above flood level, so the road will always be passable.
- Abutments support the superstructure and retain the soil of the approach embankments.
- Wing walls are needed to provide support and protect the road embankment from erosion.
- Embankments must be carefully compacted behind the abutment to prevent soil settlement which would result in a drop off between the bridge deck and the road surface at the end of the bridge.
- Weep holes are needed in the abutment to allow water to drain out from the embankment, and avoid a build-up of ground water pressure behind the abutment.
- Bridges should not significantly affect the flow of water (i.e. the openings must be large enough to prevent water backing up and flooding or over topping the bridge).
- The shape of the abutments and piers will affect the volume of flow through the structure and also the amount of scouring.
- Bridges require carefully designed foundations to ensure that the supports do not settle or become eroded by the water flow. On softer ground this may require piled foundations which are not covered in this Manual.

- Water from the roadside drains should be channelled into the watercourse to prevent erosion of the bank or scour of the abutment structure.
- Guide stones or kerbs should be placed at the edge of the carriageway to increase vehicle safety.
- If the crossing is to be used by pedestrians, proper protected footways should be designed on both sides of the carriageway.
- Reinforced concrete parapets are preferred rather than steel guard rails. They should be flared away at the ends and ramped for safety reasons. Warning or guard posts should be provided on the bridge approaches because vehicles need to slow down for safety.

11.12.3 Advantages and Disadvantages of Small Bridges (Spans <10 m).

The advantages and disadvantages of small bridges are summarised in Table 11-10:

Table 11-10: Advantages and disadvantages of small bridges

Advantages	Disadvantages
• The road is always passable because the structure should not be overtopped. • Simple arch bridges can be constructed primarily with local labour skills and local materials without the need for craneage. However, simply supported spans are more complex.	• Bridges are normally significantly more expensive than other road structures. • They are more complex than other structures and will require specialist engineering support for design and construction. • Additional height and earthworks in approach embankments. • Bridges may require heavy duty lifting cranes for the deck components. • Although all structures should be inspected for defects, bridges require regular detailed checks. • Bridges are likely to fail if flood flow predictions are incorrect and they are over topped. • A small amount of scour and erosion can often result in major damage to structures.

11.12.4 High Level Bridge

A high level bridge allows a road to cross a wide or deep water course without the water passing over the top of the deck, even during heavy rain. The design of high level bridges is beyond the scope of this Manual.

11.13 STRUCTURE SELECTION

11.13.1 General

The objective in selecting a structure for a water crossing is to choose the most appropriate design for each location. This selection should be based on the factors outlined in Figure 11-13.

For small watercourses and relief structures the choice of structure will, in general, be between a culvert and drift and, for larger watercourses, between a vented ford and a large diameter culvert, or possibly a bridge. The choice of structure will be determined by all the factors discussed above, but particularly by the predicted maximum water flow, its seasonal variations and the duration of road closures that can be tolerated. It should also be noted that the Figure only highlights the key issues and should only be used as a guide when determining the most appropriate structure.

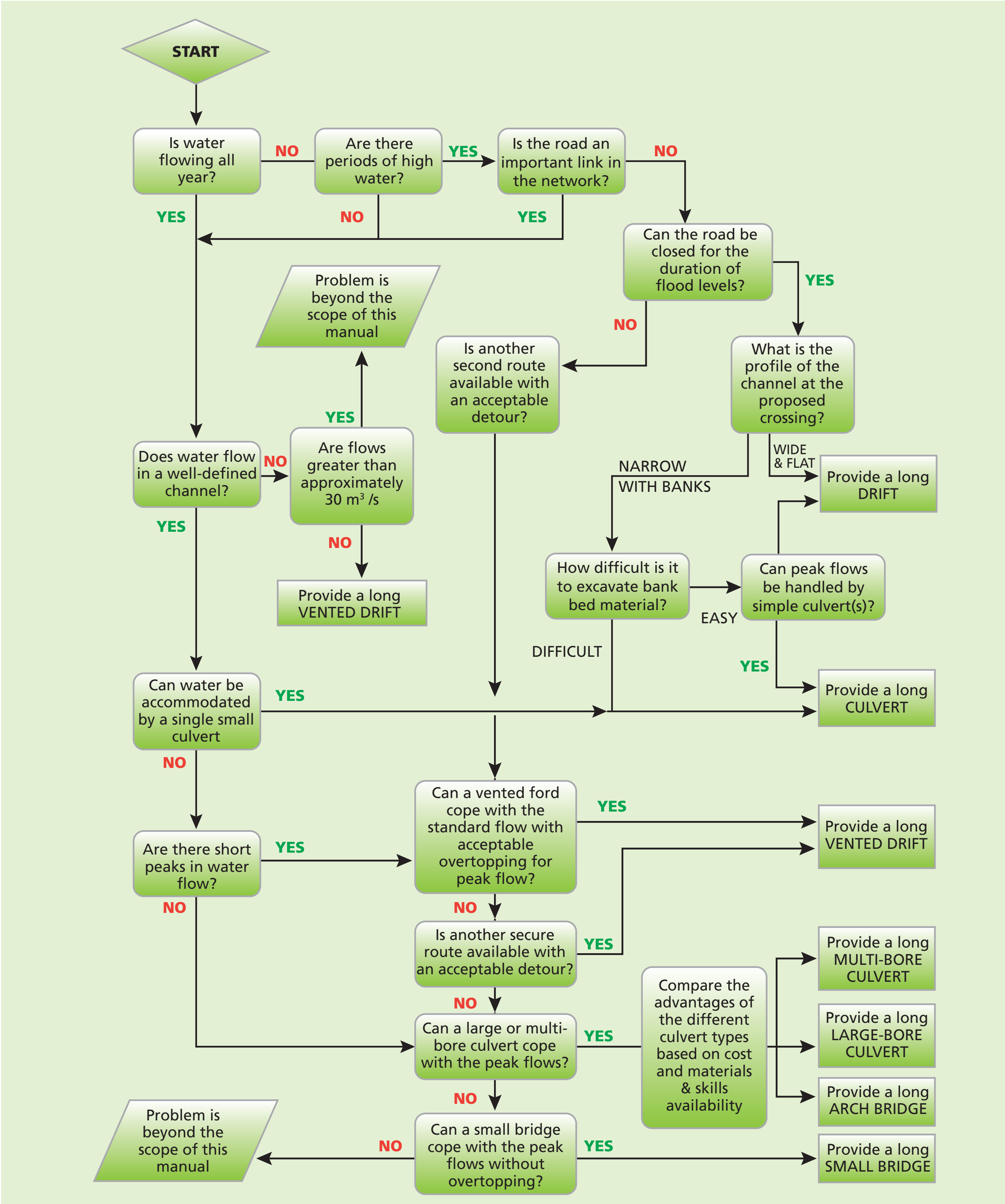


Figure 11-13: Recommended cross drainage structures

The flow rate is dependent on the rainfall in the catchment area and the run off conditions as explained in Section 11.2. The design of structures is primarily based on the peak flow rate but it is often necessary to know the normal flow rate for two reasons.

- For the design of drifts and vented drifts it is necessary to ensure that vehicles can cross the drift during the normal flow or that, in the case of a vented drift, the water passes through the pipes and the vented ford is not overtopped.
- To check that there will be no long-term damage to the structure due to erosion. The short period of peak flows may not damage erodible parts of the structure but it is necessary to ensure that parts of the structure permanently in contact with the water flow are not damaged.

11.13.2 Closure Periods for Seasonal Flow

An investigation into the variation in seasonal water flows is required if the proposed structure will be overtopped. It is necessary to determine the proportion of the year that higher flows will be experienced to estimate the number of days the structure may not be passable. It may be necessary to raise the running surface of the structure, such as a vented ford, to ensure that the structure is only overtopped during particularly rainy months. Unless detailed rainfall data is available for the area it is likely that the only suitable methods for collecting seasonal water levels and flows will be from the knowledge of the local population.

In the absence of any local information and data, suggested upper and lower bounds for closure times are shown in Table 11-11.

Table 11-11: Suggested closure times

Criteria	Drift most favourable	Drift least favourable
ADT	< 5 vpd	>200 vpd
Average annual flooding	< twice per year	More than 10 times per year
Average duration of traffic interruption per occurrence	< 24 hours	>3 days
Extra travel time for detour	< 1 hour	>2 hours / no detour

A combination of structures may often be the most cost effective solution. Wide perennial flood plains may be best crossed by vented fords with long approach embankments with relief culverts along their length. Similarly bridge lengths could be shortened in combination with relief culverts if erosion potential at the crossing point is found to be minimal due to flat terrain and stable material.

This Manual provides an overview of each individual structure type but consideration should always be given at initial design and cost estimation stage as to whether a combination of structures will be more cost effective for watercourse crossings.

When the problem is beyond the scope of this Manual, specialist bridge engineering skills are required.

11.14 SCOUR CONTROL

11.14.1 General

Erosion is a frequent problem that must be addressed during drainage design. The majority of structural failures of drainage structures occur during flood periods and over 50% of these failures can be attributed to scour.

Erosion/scour is also closely linked with the geotechnical problems of slope stability that are discussed in Chapters 5 and 6, and bioengineering methods of stabilising slopes and preventing erosion described in Chapter 12.

Thus erosion and scour is a large subject that often requires specialist advice. This Chapter deals with the most common issues likely to be encountered on LVRs.

11.14.2 Characteristics of Scour

There are basically three types of scour or erosion. The first two are caused by the existence of the drainage structure itself in concentrating the flow of water and/or increasing its velocity. There are two aspects:

- Erosion/scour around the structure itself that threatens its integrity and its continued existence.
- Erosion/scour that occurs because of the structure but upstream and especially downstream away from it.

The third type is essentially natural scour or erosion that occurs within all natural water channels irrespective of the existence of man-made drainage structures. This will alter the hydraulic environment over time and needs to be considered in the design of the road.

The amount of scour is dependent on the speed of the water flow and the erodibility of the material that the water comes into contact with. If the flow is not parallel to the constriction more scour will occur on one side than the other. Water is accelerated around abutments, piers and other obstructions, creating vortices with high velocities at abrupt edges on the obstruction, increasing the scour depth, often dramatically.

Trapped debris can also restrict the flow of water and cause an increase in water velocity. It is important that structures are designed to minimise the chances of debris being trapped and to ensure that inspections and maintenance are carried out after flood periods to remove any lodged debris.

Finally, if the water is already carrying a large amount of material eroded from further upstream, a greater amount of scour will occur at the structure.

It is difficult to predict the level of scour that may be experienced for a particular design. Existing methods require detailed knowledge that is not readily available and they are not very accurate. Thus engineering judgement is required. *A design manual for small bridges, Overseas Road Note 9 (TRL, 1992) is* also a useful reference.

11.14.3 Designing to Resist Scour

This Manual proposes a number of ‘rules’ for designing to resist scour. It must be stressed that these rules are not infallible and local knowledge should also be taken into account when designing a structure.

Constrictions

The amount of scour experienced at a structure is proportional to the restriction in the normal water flow. Hence, as a general principle, wherever possible, any constrictions to water flow should be minimised.

Cut-off-walls

Cut-off walls, also called curtain walls, should be provided at the edge of a structure to prevent water eroding the material adjacent to the structure. The location of cut-off walls for the various structures is shown in Table 11-12.

Table 11-12: Cut-off wall locations

Structure	Locations
Drift	Upstream and downstream sides of drift slab
Culvert	Edges of inlet and outlet apron
Vented drift/ford	Upstream and downstream sides of main structure and approach ramps
Large diameter culvert	Upstream and downstream sides of approach ramps. The foundations of the main structure should be built at a greater depth than standard cut-off walls below the possible scour depth

The absence of cut-off walls at the inlet of the structure could easily allow water to seep under the apron causing settlement and eventually collapse of the structure. At the downstream end of the structure the flowing water could erode the material next to the apron, eventually eroding under the apron and causing it to collapse.

The minimum depth of the cut-off walls depends on the ground conditions. Where a rock layer is close to the ground surface the cut-off walls should be built down to this level and firmly keyed into the rock using dowels. In other situations the depth of the cut-off wall should be greater than the expected depth of scour. This is best estimated from local experience under similar conditions. The depth is measured from the lowest point in the bed of the watercourse at the crossing point. If no information is available Table 11-13 provides guidance. For larger structures advice should be sort from an experienced engineer.

Table 11-13: Foundation and cut off wall depths

Structure	Cut off wall depth (m)	Comments
Drift	1.5	
Relief culvert	1.0	
Water course culvert	1.5	Headwalls and wingwalls
Vented drift	2.0	

Use of piers

If piers are absolutely necessary they should be aligned exactly in the direction of water flow.

Culvert headwalls and wingwalls

Headwalls and wingwalls are required at each end of a culvert and serve a number of different purposes:

- They direct the water in or out of the culvert.
- They retain the soil around the culvert openings.
- They prevent erosion near the culvert and seepage around the pipe, which causes settlement.

The headwall can be positioned at different places in the road verge or embankment as shown in Figure 11-14.

The closer the headwall is placed to the road on an embankment the larger and more expensive it will be. The most economical solution for headwall design will be to make it as small as possible. Although a small headwall will require a longer culvert, the overall structure cost will normally be smaller. If, due to special circumstances at a proposed culvert site, a large headwall with wingwalls is required it should be designed as a bridge wingwall (with a soil retaining function).

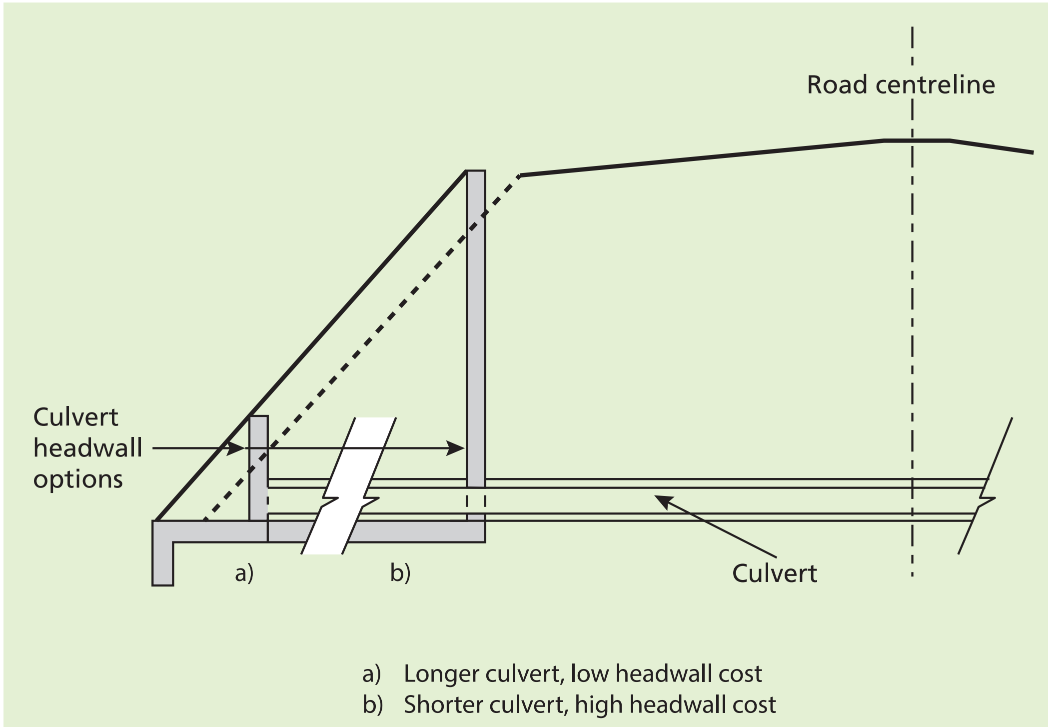


Figure 11-14: Possible culvert headwall positions

Where a road is not on an embankment the size of the headwall will be small regardless of position. In this case the position of the headwalls will be determined by the road width and any requirements of national standards. The headwalls should be positioned at least 1 m beyond the edge of the carriageway to prevent a restriction in the road and reduce the possibility of vehicle collisions.

Headwalls should project just above the road surface (+/- 150 mm) and be painted white so that they are visible to drivers. Marker posts may also be used to warn drivers of the existence of a headwall which may be a potential road safety hazard, There are a number of different layout options for culvert headwalls which are shown in Figure 11-15.

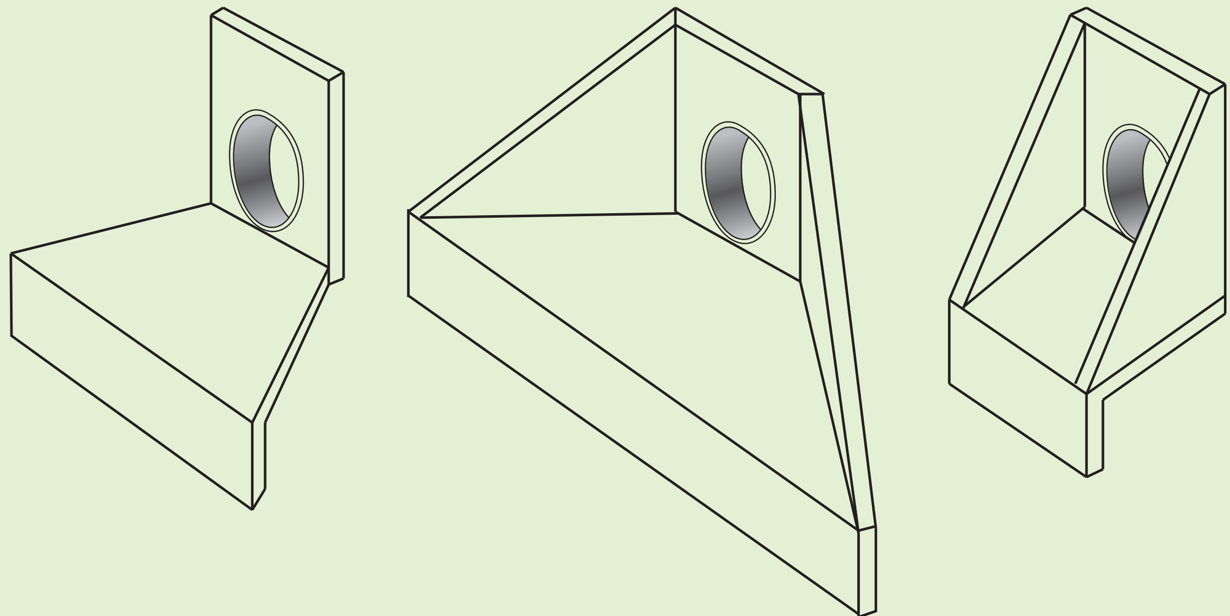
Headwall with drop inlet

This arrangement should be used when the road is on a steep side slope to reduce the invert slope of the culvert.

Headwall with L inlet

This arrangement should be used where the road is on a gradient and water is to be transferred from the carriageway side drain on the high side of the road.

Headwall and adjacent works must be designed so that the culverts can be de-silted manually under maintenance arrangements. This can be difficult with a drop inlet and silt trap arrangement.



Simple headwall and apron	Headwall with splay wingwalls	Parallel wingwalls
This arrangement can be used for simple low flow relief culverts. It is the cheapest option but prone to erosion for larger flows.	This arrangement should be used for larger flows instead of the simple arrangement. The wingwalls increase the protection from erosion.	Where the road is on an embankment it is essential that wingwalls be provided to prevent the risk of water seepage and subsidence of the embankment. This arrangement is likely to be the most economical.

Figure 11-15: Headwall and wingwall arrangements

Vertical positioning of culverts

The vertical positioning of culverts requires particular attention. The consideration of the natural vertical alignment of the watercourse must take precedence over the vertical alignment of the road. Neglect of this factor has led to many culverts being installed incorrectly, leading to excessive silting, erosion and in many cases failure. The most appropriate culvert type will depend on the outfall gradient.

Flat outfall (less than 5%). This culvert type should be used in flat areas and for watercourses with shallow gradients. In these cases the road should be built up over the culvert with ramps 20-50 m long or to comply with national road vertical alignment standards. A culvert will silt up if it is positioned too low .

Intermediate outfall (approx. 5 to10 %). This arrangement requires the culvert to be excavated slightly into the existing ground, although the invert of the culvert at the inlet should be at the same level as the bed of the watercourse. The outlet of the culvert will be below the existing ground level and will require an outfall ditch to be dug with a gradient of 2 - 4%. The road will still have to be built up with ramps or alignment adjustment over the culvert to provide the minimum required cover.

Steep outfall (more than 10%). The culvert can be installed without building up the road level. The culvert should be buried to provide adequate cover over the pipe. A drop inlet will be required at the entrance and a short outfall ditch at the exit. On steeply sloping ground careful attention should be given to preventing erosion downstream of the culvert.

Pipes transferring large water volumes

One of the most important design rules when constructing a road water structure is to disrupt the flow as little as possible. Unfortunately this is not possible for a culvert that is transferring water from a side drain.

The water must make an abrupt right angle change in direction to enter the culvert. For large flows there will therefore be a large amount of turbulence in the water and the potential for scour. The following key features should be used in the inlet design for large flows:

- Rounded wingwalls to 'guide' water into pipe.
- Sloping wingwall on inside radius.
- Lined channel sides and base which extend 5 m up the channel.
- Cut-off wall provided at the edge of the inlet.
- The box culvert option should be considered because this will cause less restriction and turbulence.

Aprons

An apron is required at the inlet and outlet of culverts and downstream of drifts and vented fords to prevent erosion. As the water flows out of or off a structure it will tend to erode the watercourse downstream, causing undercutting of the structure. Aprons should be constructed from a material, which is less susceptible to erosion than the natural material in the streambed.

Drift aprons. Where the discharge velocity across the drift is less than 1.2 m/s, which may be experienced for relief drifts, a coarse gravel layer (10 mm) will provide sufficient protection down stream of the drift. For discharge velocities greater than 1.2 m/s more substantial protection will be required utilizing larger stones. The width of the apron should be at least half the width of the drift and extend across the watercourse for the whole length of the drift.

Culvert aprons. Aprons should be provided at both the inlet and outlet of culverts. They should extend the full width between the headwall and any wingwalls. If the culvert does not have wingwalls the apron should be twice the width of the culvert pipe diameter. The apron should also extend a minimum of 1.5 times the culvert diameter beyond the end of the pipe. Cut-off walls should also be provided at the edge of all apron slabs. The choice of apron construction is likely to depend on the type of material used for construction of the culvert. It may be constructed from gabion baskets, cemented masonry or concrete.

Vented ford aprons. The apron for vented fords should extend the whole length of the structure including downstream of the approach ramps to the maximum design level flood. The other design requirements for vented ford aprons are the same as culvert aprons.

11.15 DOWNSTREAM PROTECTION

11.15.1 General

Whenever watercourses are channelled through pipes, such as in culverts and vented drifts, or through narrow openings in bridges, severe erosion can be caused to land and property downstream of the structure. If agricultural land or buildings are close to the proposed structure, careful consideration must be given to erosion protection. Undersized structures can also cause water to back up causing flooding upstream and possible property damage.

11.15.2 Remedial Measures

The use of aprons downstream of a structure should prevent erosion and undercutting of the structure itself. However, in small, constrained channels severe erosion may still occur after the apron, particularly where the watercourse is on a gradient. It is therefore often necessary to provide additional protection to the watercourse, to reduce the velocity of the water and prevent erosion.

For slow flowing water it is unlikely that any protection will be needed, but for faster flowing water the maximum allowable velocity will depend on the bed material and the amount of silt or other material already being carried in the water.

Erosion can occur in any channel regardless of the presence of any structure. It is therefore not possible to state how far downstream of a structure channel protection should extend. However, the following issues should be taken into account:

- The general erodibility of the bed, which will be based on the type of channel material and the gradient.
- The likelihood of damage to the structure if erosion occurs downstream.
- The potential effects of erosion on downstream areas (e.g. damage to buildings or farming land).

The maximum water flow velocities that can be tolerated without channel protection related to the type of bed material are shown in Table 11-14.

Table 11-14: Maximum water velocities (m/second)

Soil type	Clear water	Water carrying fine silt	Water carrying sand and fine gravel
Fine sand/coarse silt	0.40	0.70	0.40
Fine sand	0.45	0.75	0.45
Sandy soil	0.50	0.70	0.60
Silty soil/sandy clay	0.60	0.90	0.60
Alluvial silts (non colloidal)	0.60	1.05	0.60
Alluvial silts (colloidal)	1.15	1.50	0.90
Stiff clay	1.15	1.50	0.90
Firm soil/coarse sand	0.75	1.05	0.70
Volcanic ash	0.75	1.05	0.60
Firm soil, silt and gravel	1.00	1.50	1.50
Gravel (5 mm)	1.10	1.20	1.20
Gravel (1 mm)	1.20	1.50	1.50
Coarse gravel (25 mm)	1.50	1.90	2.00
Graded silt to gravel	1.10	1.60	1.60
Graded sand and gravel	1.20	1.50	
Cobbles (50 mm)	2.00	2.40	2.40
Cobbles (100 mm)	3.00	3.50	3.50
Lined with established grass on good soil	1.70	2.40	1.70
Lined with bunched grasses (exposed soil between plants)	1.10	1.10	1.10
Grass with exposed soil	1.00	1.80	
Shales	1.85	1.85	1.50
Rock	Negligible scour at all velocities		

Source: Robinson and Thagesen (2004).

11.15.3 Bank Elevation and Bed Material of the Watercourse

The resistance of the watercourse banks and bed to erosion will dictate the type of foundation bank protection and hence structure that can be built. For material which is easily erodible it will be necessary to have deep foundations and possibly extensive bed and bank protection or structures which are not susceptible to damage. The steepness of the banks and difficulty in excavating soil material will also determine the most convenient approach roads.

A major factor affecting the cost of building a structure is the amount of material which needs to be imported to or exported from the site. Where the road alignment is at a similar level to the riverbed it may be difficult to construct a structure that will not be overtopped without large approach ramps/ embankments as illustrated in Figure 11-16.

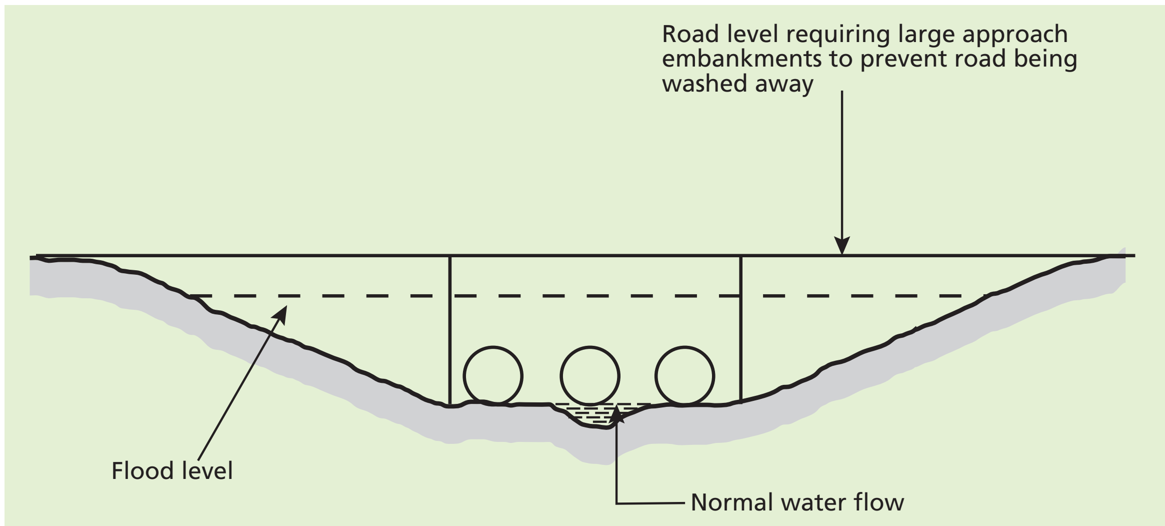


Figure 11-16: Large embankments required to prevent road flooding

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12.1 INTRODUCTION

12.1.1 Background

Moisture is the single most important factor affecting pavement performance and long-term maintenance costs. Thus, one of the significant challenges faced by the designer is to provide a pavement structure in which the weakening and erosive effects of moisture are contained to acceptable limits and degree of acceptable risk. Most LVRs will be constructed from natural, often unprocessed materials, which tend to be moisture sensitive. This places extra emphasis on drainage and moisture control for achieving satisfactory pavement life.

12.1.2 Purpose and Scope

The chapter deals with the elements of both internal and external drainage as well as measures for dealing with erosion caused by run-off from the road prism. The scope of the chapter is limited to the point at which water has entered some suitable collection system, e.g. pipes or side drains.

The chapter does not consider the hydrological aspects of drainage, such as the sizing of catchment areas, determination of run-off volumes and sizing of water crossings and drains to accommodate the flow. These aspects are addressed in *Chapter 11 – Hydrology and Drainage Structures.*

12.2 SOURCES OF MOISTURE IN A PAVEMENT

12.2.1 General

The various causes of water movement into and out of a pavement are listed in Table 12-1 and illustrated in Figure 12-1. The Table highlights those aspects that should be addressed when designing an effective drainage system.

Table 12-1: Typical causes of water movement into and out of a road pavement

Means of Water Ingress	Causes
Through the pavement surface	Through cracks caused by thermal or traffic loads.
	Through cracks and potholes caused by pavement failure.
	Penetration through intact layers.
From the subgrade	Artesian head in the subgrade.
	Pumping action at formation level.
	Capillary action in the subbase.
From the road margins	Seepage from higher ground, particularly in cuttings.
	Reverse falls at formation level.
	Lateral/median drain surcharging.
	Capillary action in the subbase.
	Through an unsealed shoulder collecting pavement and ground run-off.
Through hydrogenesis (aerial well effect)	Condensation and collection of water from vapour phase onto the underside of an impermeable surface.
Means of Water Egress	**Causes**
Through the pavement surface	Through cracks under pumping action through the intact surfacing.
Into the subgrade	Soakaway action.
	Subgrade suction.
To the road margins	Into lateral/median drains under gravitational flow in the subbase.
	Into positive drains through cross-drains acting as collectors.

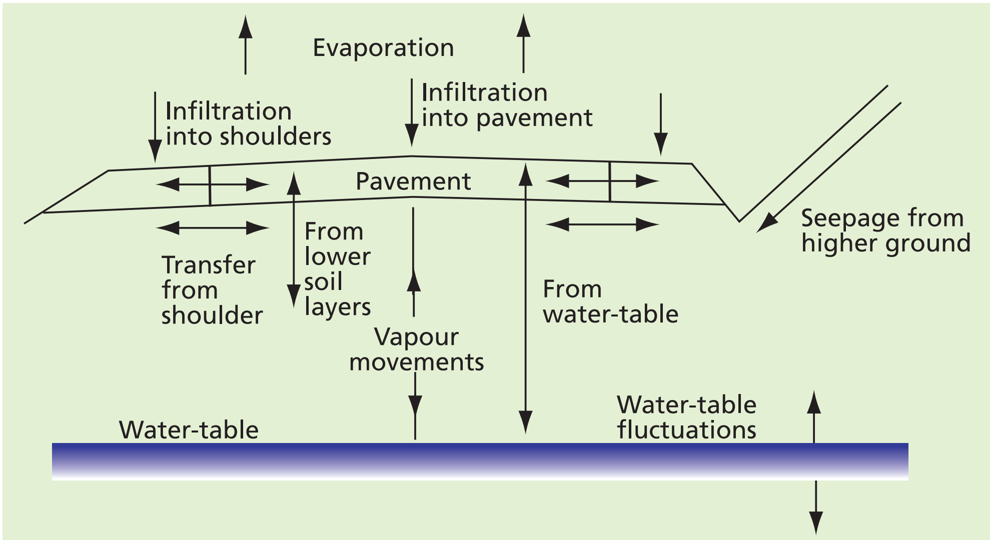

Figure 12-1: Moisture movements in road pavements
Source: ARRB (2000).

12.2.2 Components of Drainage

Drainage is divided into external and internal drainage. External Drainage is concerned with the control of water that is outside the road structure. However, since it is impossible to guarantee that water will not enter road structures during their service lives, it is important to ensure that water is able to drain out from within the pavement itself. Thus internal drainage is concerned with the control of water that enters the road structure, either directly from above the road pavement or from below, and the measures than can be adopted to avoid trapping water within the pavement.

12.3 EXTERNAL DRAINAGE

12.3.1 General

There are three important components of external drainage:

- Preventing water from entering the road structure; for example, aspects of geometric design (e.g. camber) and waterproofing (e.g. surfacings).
- Collecting the water and channelling it safely away from the road by means of drainage channels.
- Allowing water to cross the road effectively from one side to the other.

12.3.2 Road Surfacing

The pavement surfacing of either a sealed or unsealed road constitutes an essential part of the drainage of a road. This surfacing, together with the cross slope on the carriageway ensures that rainwater does not enter the foundation of the road but is led to the side of the road.

Unpaved roads: The use of a natural gravel wearing course requires that the material is well protected from surface water. This can be achieved by reducing the permeability of the surfacing by ensuring that:

- The material is reasonably well graded with an appropriate plasticity for binding the material together (refer to *Chapter 14 – Structural Design: Unpaved Roads)*.
- The soil is well compacted to at least 95% of BS heavy compaction.
- An appropriate camber is used to shed rain water effectively.

Paved roads: The most effective means of preventing water from entering the road pavement from above is by the use of a durable, waterproof surfacing that is adequately maintained over the design life of the road. There are many types of bituminous surfacings that can used for this purpose, some being more impermeable than others. These are addressed in *Chapter 15 – Surfacing* of this Manual.

12.3.3 Crossfall

Effective surface drainage is facilitated by ensuring that the road is designed with an appropriate crossfall to drain water from the road surface. Three types of crossfall may be used for this purpose as illustrated in Figure 12-2.

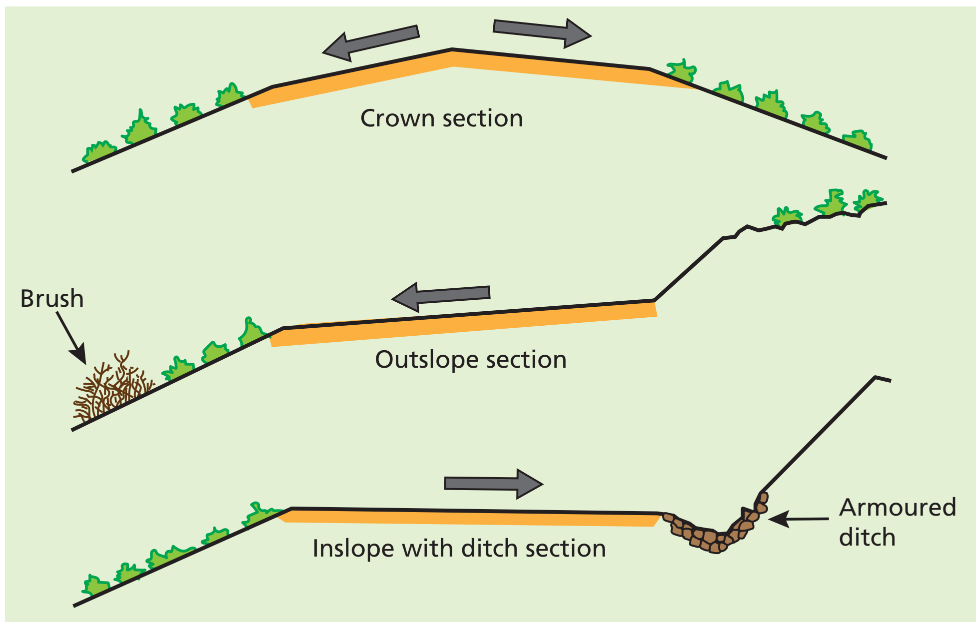

Figure 12-2: Three types of crossfall

Carriageway crossfall: The design of the crossfall is often a compromise between the need for reasonably steep crossfalls for drainage and relatively flat cross-falls that are good for driver comfort and safety. The ideal crossfall depends on the pavement surfacing. The carriageway crossfall on straight road sections for various types of pavement surfaces is shown in Table 12-2**.**

Table 12-2: Typical carriageway crossfall values on straight road sections

Type of Pavement Surface	Crossfall (%)
Portland cement concrete	2.0 – 3.0
Asphalt concrete	2.5 – 3.5
Bituminous surface treatment	3.0 – 4.0
Gravel/waterbound macadam	4.0 – 6.0
Earth[1]	5.0 – 7.0

Note 1: For sandy roads (e.g. fine coastal sands with low plasticity), a 2 – 3% camber would be more suitable to avoid wash out of the finer fraction.

On paved roads a camber of 3.5% is recommended. Although steeper than many traditional specifications, it does not cause problems for drivers in a low speed environment. It also accommodates reasonable construction tolerance of +/- 0.5% thereby taking into account the skills and experience of small scale contractors and LBM of construction, and provides an additional factor of safety against water ingress into the pavement should slight rutting occur after trafficking. Failure to achieve the minimum values of crossfall/

camber will in combination with rutting or other minor depressions result in possible ponding of water on the road surface, leading to potholing and eventual ingress of water into the road pavement.

Failure to achieve the minimum values of crossfall/camber will, in combination with rutting or other minor depressions, result in possible ponding of water on the road surface, leading to potholing and eventual ingress of water into the road pavement.

Shoulder crossfall: When permeable roadbase materials are used, particular attention must be given to the drainage of this layer. Ideally, the roadbase and subbase should extend right across the shoulders to the drainage ditches. In addition, proper crossfall is needed to assist the shedding of water into the side drains. A slope of about 4-6% is suitable for the shoulders. However, it is not usually possible to increase the crossfall from the value used for the running surface to a greater value for the shoulders. Hence every effort should be made during construction to ensure that the crossfall of the road running surface is correct, preferably at the upper limit of the specification range is given in Table 12-2.

Increased shoulder crossfalls, typically by about 2 - 3% more, are required for unpaved roads.

Lateral drainage can also be encouraged by constructing the lower pavement layers with an exaggerated crossfall, especially where a permeability inversion (decreasing permeability as you move down the pavement layers) occurs (see Section 12.4). This can be achieved by constructing the top of the subbase with a crossfall of 3-4% and the top of the subgrade with a crossfall of 4-5% (TRL, 1993). Although this is not an efficient way to drain the pavement, it is inexpensive and therefore worthwhile, particularly as full under-pavement drainage is rarely likely to be economically justified for LVRs. Figure 12-3 illustrates the recommended drainage arrangements.

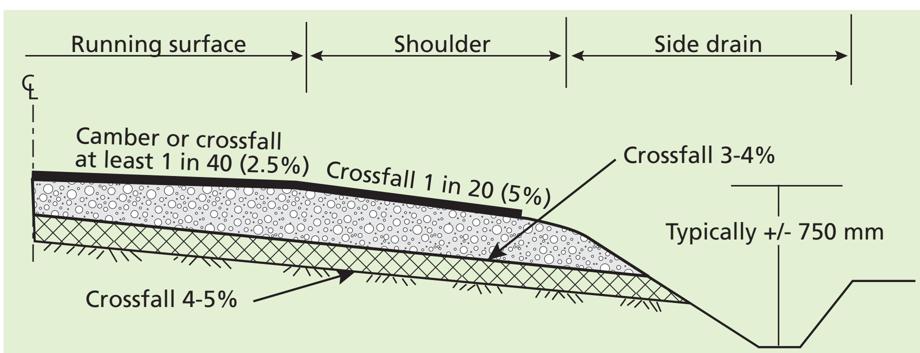

Figure 12-3: Recommended drainage arrangements

Figure 12-4: Infiltration of water through a permeable surfacing

Under no circumstances should the trench (or boxed in) type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders. As illustrated in Figure 12-4, this type of construction has the undesirable feature of trapping water at the pavement/shoulder interface and inhibiting flow into drainage ditches which, in turn, facilitates damage to the shoulders and eventual failure under even light trafficking.

If it is too costly to extend the roadbase and subbase material across the shoulder, drainage channels at 3 m to 5 m intervals should be cut through the shoulder to a depth of 50 mm below subbase level. These channels should be back-filled with material of roadbase quality but which is more permeable than the roadbase itself, and should be given a fall of 1 in 10 to the side ditch. Alternatively, a preferable option would be to provide a continuous layer of pervious material of 75 mm to 100 mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the subbase.

12.3.4 Crown Height

Paved roads

To achieve adequate external drainage, the road must also be raised above the level of existing ground such that the crown height of the road (i.e. the vertical distance from the bottom of the side drain to the finished road level at the centre line) is maintained at a minimum height, hmin. This height must be sufficient to prevent moisture ingress into the potentially vulnerable outer wheel track of the carriageway (Figure 12-5). The recommended minimum crown height of 0.75 m applies to unlined drains in relatively flat ground (longitudinal gradient, g, less than 1%). The recommended values for sloping ground (g > 1%) or where lined drains are used, for example, in urban or peri-urban areas, are shown in Table 12-3. The capacity of the drain should meet the requirements for the design storm return period *(Chapter 11 – Hydrology and Drainage Structures).*

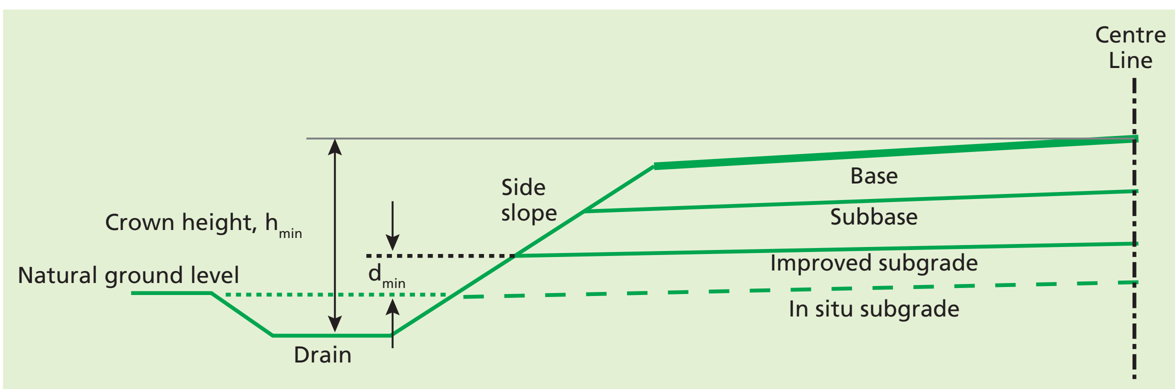

Figure 12-5: Crown height for paved road in relation to depth of drainage ditch

Table 12-3: Recommended crown height, h_{min} above drainage ditch invert for paved roads

Unlined drains		Lined drains	
Gradient < 1%	Gradient > 1%	Gradient < 1%	Gradient > 1%
0.75	0.65	0.65	0.50

In addition to observing the crown height requirements, it is also equally important to ensure that, where practicable, the bottom of the subbase is maintained at a height of at least 150 mm above the existing ground level (distance d_{min} as indicated in Figure 12-5). This is to minimise the likelihood of wetting up of this pavement layer from moisture infiltration from the drain.

Irrespective of climatic region, if the site has effective side drains and adequate crown height, then the in situ subgrade strength will probably remain below OMC. If the drainage is poor, the in situ strengths will fall below the strength at OMC.

Unpaved Roads

Engineered earth roads: Should be raised adequately to allow side drains to be constructed and water to be removed from the road structure. The crown of the road should be at least 350 mm above the base of the drain, irrespective of climatic zone, as the soils in engineered roads are generally more moisture sensitive than those in a well-compacted gravel road (see Section 14.2.3).

Gravel roads: It is also necessary to achieve a minimum crown height which is dependent on the climate and road design class as shown in Table 12-4. Unless the existing road is well below existing ground level, this can usually be achieved by proper 'forming', i.e. shaping the road bed to ensure adequate road levels, coupled with cutting table drains to an appropriate depth below existing ground level. Where necessary, additional fill will have to be imported or obtained from shallow cuttings to achieve the required h_{min}.

Table 12-4: Minimum height h_{min} between road crown and drain invert level for unpaved roads

Road Class	Climate	
	Wet	Dry
	h_{min} (m)	h_{min} (m)
DC-8	0.35	0.25
DC-7	0.40	0.30
DC-6	0.45	0.35
DC-5	0.50	0.40

Because of the critical importance of observing the minimum crown height and minimum height of the bottom of the subbase above existing ground level along the entire length of the road, the measurement of this parameter should form an important part of the drainage assessment carried out during site investigations. This is to avoid any existing drainage problems associated with depressed pavement construction, often observed on gravel roads that have evolved over time with no strict adherence to observing minimum crown heights as illustrated in Figure 12-6.

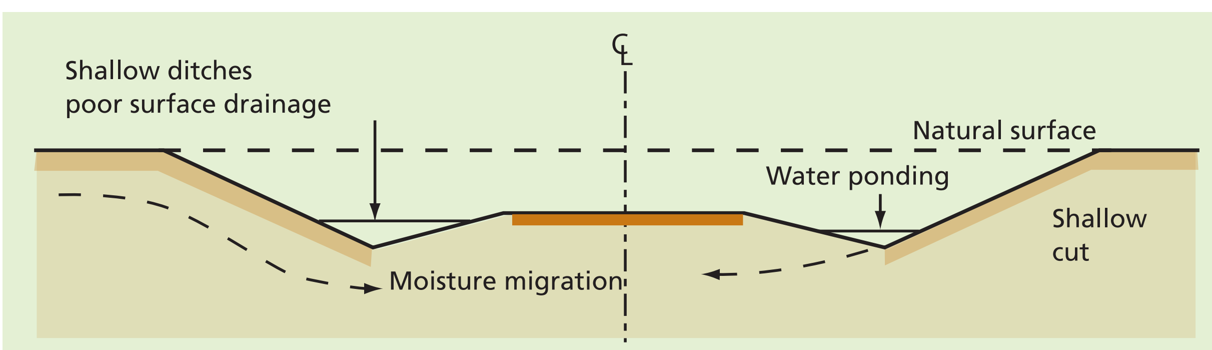

Figure 12-6: Potential drainage problems associated with depressed pavement construction

Roads with sunken profiles

There may be situations where a road has a sunken profile for more than 200 m with no possibility of discharging water, as illustrated in Figure 12-6.

There are three basic options for dealing with a sunken profile:

Lined ditches: If the side ditches are unable to discharge every 200 m, the build-up of water in them becomes significant and erosion is likely to occur, particularly on steeper gradients. Where this is likely, ditch lining with concrete should be considered. Widening the side drain to accommodate the high water flow may also be required.

Parallel drains: In some locations the volume of water in the side drains may be relieved by constructing additional drains (catch water drains) parallel to the road and several m outside of the side drains. These should be 1 m wide and excavated to a level just below the side drains. Water should be channelled from the side drain to the parallel drain by constructing mitre drains between them every 20 m. Note, however, that this option creates considerable additional maintenance requirements because deep parallel drains are difficult to de-silt.

Soak-away ponds: If the soil adjacent to the road is free draining, mitre drains can be constructed to soak-away ponds. These may be constructed of approximate dimensions 5 m x 5 m x 1 m deep every 50 m along both sides of the road. This capacity would be sufficient to hold the water falling on the road from a storm of up to 100 mm of rain. Note, however, that this option may not be applicable in extremely wet regions where water does not soak or evaporate rapidly enough.

12.3.5 Drains

Side drains
Side drains serve two main functions namely to collect and remove surface water from the immediate vicinity of the road and, where needed, to prevent any sub-surface water from adversely affecting the road pavement structure. It is essential to install a system of side drains that discharges water frequently in order to avoid high flow concentrations that will inevitably lead to erosion.

Side drains can be constructed in three forms: V-shaped, rectangular or trapezoidal (Figure 12-7). The choice depends on the type of technology (use of graders, labour based technology), the required hydraulic capacity, arrangements for maintenance, space restrictions, traffic safety and any requirements relating to the height between the crown of the pavement and the drain invert. For safety reasons a wide trapezoidal and shallower drain for a given flow capacity is preferable to a deeper V or rectangular drain.

V-shaped drains
Although relatively easily constructed by a motor or towed grader, they should be discouraged because of their potential to scour easily.

Rectangular shaped drains
These require little space but need to be lined with rock, brick or stone masonry or concrete to maintain their shape.

Trapezoidal drain
These can be constructed and maintained easily by hand and improve traffic safety.

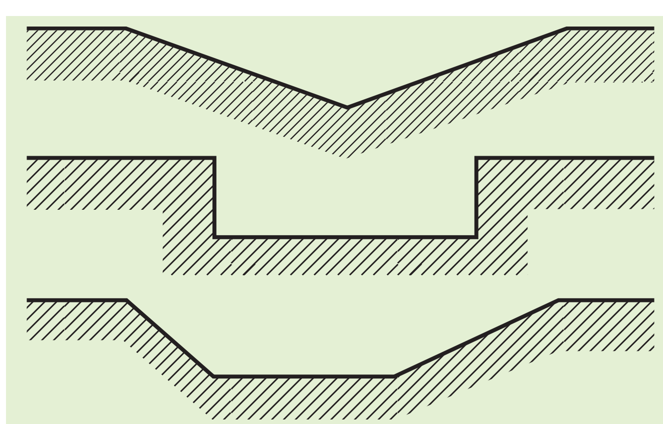

Figure 12-7: Typical types of side drains

The minimum recommended width of the side drain is 500 mm and the minimum recommended longitudinal gradient is 0.5%. Slackening of the side drain gradient in the lower reaches of significant lengths of drain should be avoided in order to prevent siltation.

Side drains are normally located beyond the shoulder breakpoint and parallel to the centre line of the road. While usually employed in cuts, they may also be used to run water along the toe of a fill to a point where the water can conveniently be diverted, either away from the road prism or through it by means of a culvert. When used in conjunction with fills, side drains should be located as close to the edge of the reserve boundary as is practicable to ensure that erosion of the toe of the fill does not occur.

The following recommendations are made regarding desirable slopes for side drains:

- To avoid ponding and siltation, the minimum slope should be in the range 0.4% to 0.5%.
- To avoid erosion, drains steeper than 3% may need scour protection, depending on the erodibility of the soil and the vegetative cover. The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table 12-5 shows recommended values for normal soils. The spacing should be reduced for highly erodible soils.

Table 12-5: Spacing between scour checks

Road gradient (%)	Scour check interval (m)
3	Usually not required
4	17
5	13
6	10
7	8
8	7
9	6
10	5
12	4

Access across side drains for pedestrians, animals and vehicles needs to be considered. Community representatives should be consulted with regard to locations, especially for established routes and in the villages or towns. The methods that could be used are:

- Widening the drain, taking its alignment slightly away from the road and hardening the invert and sides of the drain.
- Beam/slab covers or small culverts.

The arrangement must be maintainable and not risk blockage of the side drain. Failure to accommodate these needs will usually result in ad hoc arrangements that compromise the function of the side drain (resulting in blockage of the water flow).

Mitre drains

These drains are constructed at an angle to the centre line of the road. They are intended to remove water from a side drain and to discharge it beyond the road reserve boundary. The amount of water in the drain should ideally be dispersed and its speed correspondingly reduced before discharge. Speed can be reduced not only by reducing the volume (more frequent spacing of mitre drains), and hence the depth of flow, but also by positioning the mitre drain so that its toe is virtually parallel to the natural contours. The downstream face of a mitre drain is usually protected by stone pitching, since the volume and speed of flow of water which it deflects may cause scour and ultimately lead to breaching of the mitre drain.

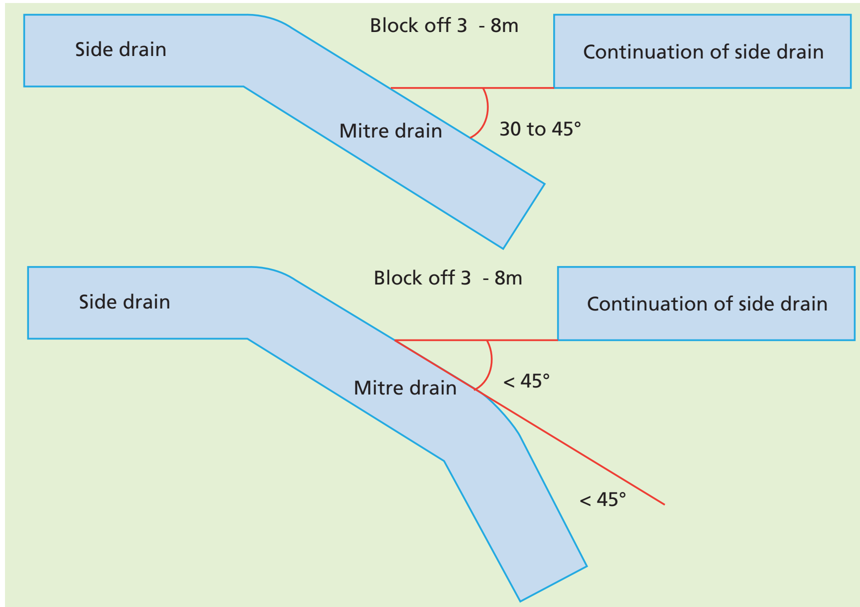

Figure 12-8: Schematic layout of mitre drains

In order to ensure that water flows out of the side drain into the mitre drain, a 'block-off' is required as shown in Figure 12-8.

It is essential that the mitre drain is able to discharge all the water from the side drain. If the slope of the mitre drain is insufficient, the mitre drain needs to be made wide enough to ensure this.

The angle between the mitre drain and the side drain should not be greater than 45 degrees. An angle of 30 degrees is ideal. If it is necessary to take water off at an angle greater than 45 degrees, it should be done in two or more bends so that each bend is not greater than 45 degrees as shown in Figure 12-8.

The desirable slope of mitre drains is 2%. The gradient should not exceed 5% otherwise there may be erosion in the drain or on the land where the water is discharged. The drain should lead gradually across the land, getting increasingly shallower. Stones may need to be laid at the end of the drain to help prevent erosion.

In flat terrain, a small gradient of 1% or even 0.5% may be necessary to discharge water, or to avoid very long drains. These low gradients should only be used when absolutely necessary. The slope should be continuous with no high or low spots. For flat sections of the road, mitre drains are required at frequent intervals to minimise silting. In mountainous terrain, it may be necessary to accept steeper gradients. In such cases, appropriate soil erosion measures should be considered.

As indicated in Table 12-6, the maximum spacing of mitre drains is dependent on the road gradient. However, depending on engineering judgement, mitre drains could be required more frequently than this and values as low as one every 20 m may be required to avoid damage to adjacent land, especially where it is cultivated.

Table 12-6: Maximum spacing of mitre drains

Road Gradient (%)	Maximum mitre drain interval (m)
12	40
10	80
8	120(1)
6	150(1)
4	200(1)
2	80(2)
<2	50

Source: Adapted from Robinson and Thagesen (1996).

Notes: 1. A maximum of 100 m is preferred but not essential.
2. At low gradients silting becomes a problem.

Interceptor, cut-off or catch-water drains

These drains are constructed to prevent water flowing into vulnerable locations (e.g. down cut faces) by 'intercepting', 'cutting off' or 'catching' the water flow and diverting it to a safe point of discharge, usually a natural watercourse, as illustrated in Figure 12-9.

Interceptor drains above cut faces should have a gradient of 2% on their full length and should be at least 3 to 5 m from the cut face. If steeper gradients in the drain are unavoidable then scour checks should be installed or the drain should be lined. The drain should also be lined where seepage will weaken the cut slope. Alternatively, the drain should be replaced by a vegetated earth bund. Interceptor drains should be at least 600 mm wide, 400 mm (minimum) deep with sides back-sloped at 3:1 (vertical: horizontal) or less.

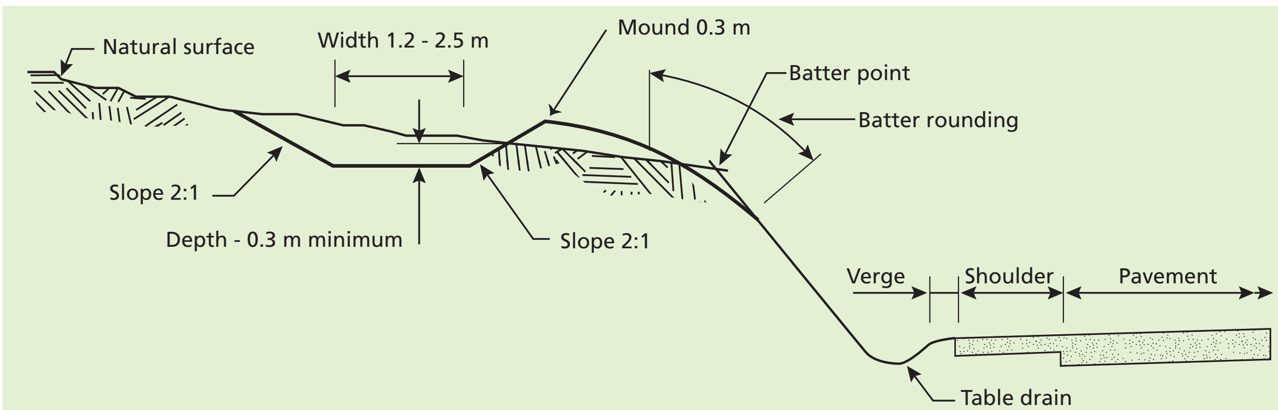

Figure 12-9: Interceptor, cut-off or catch-water drain

Chutes

Chutes are structures intended to convey a concentration of water down a slope that, without such protection, would be subject to scour, as shown in Figure 12-10. Since flow velocities are very high, stilling basins are required to prevent downstream erosion. The entrance of the chute needs to be designed to ensure that water is deflected from the side drain into the chute, particularly where the road is on a steep grade. On embankments it may be necessary to lead water to the top of chutes using kerbing.

Figure 12-10: Open chute

It is important that chutes be adequately spaced to remove excess water from the shoulders of the road. Furthermore, the dimensions of the chutes and stilling basins should be such that these drainage elements do not represent an excessive risk to errant vehicles. Generally, they should be as shallow as is compatible with their function and depths in excess of 150 mm should be viewed with caution.

Because of the suggested shallow depth, particular attention must be paid to the design and construction of chutes to ensure that the highly energised stream is not deflected out of the chute. This is a serious erosion hazard which can be obviated by replacing the chute with a pipe.

12.4 INTERNAL DRAINAGE

12.4.1 General

This is an essential element of road design because the strength of the pavement layers, especially the subgrade, depends critically on the moisture content during the most likely adverse conditions. Such drainage depends primarily on the properties of the materials, including their permeability. Shoulders are also an important aspect of the internal drainage system in that they contribute to the effective drainage of water out of the structure.

12.4.2 Avoiding Permeability Inversion

A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. Under infiltration of rainwater, there is potential for moisture accumulation at the interface of the layers. The creation of such a perched water table often leads to rapid lateral wetting under the seal. This may lead to base or subbase saturation in the outer wheeltrack and result in catastrophic failure of the base layer when trafficked.

A permeability inversion often occurs at the interface between subbase and subgrade since many subgrades are of cohesive and relatively impermeable fine-grained materials. Under these circumstances, a more conservative design approach is required that specifically caters for these conditions, for example, designing for wetter subgrade conditions.

Preventing a permeability inversion can be achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the subbase in a three-layered system. However it is unlikely that for LVRs there will be any choice of materials. For a paved road, if a permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that the lateral wetting front does not extend under the outer wheeltrack of the pavement.

12.4.3 Subsurface Drainage

Seepage may occur where the road is in cut and may result in groundwater entering the subbase or subgrade layers as illustrated in Figure 12-11 and Figure 12-12. Inadequate surface or subsurface drainage can therefore adversely affect the pavement by weakening the soil support, and initiating creep or failure of the downhill fill or slope. Localised seepage can be corrected in various ways but seepage along more impervious layers, such as shale or clay, combined with changes in road elevation grades, may require subsurface drains as well as ditches as shown in Figure 12-13.

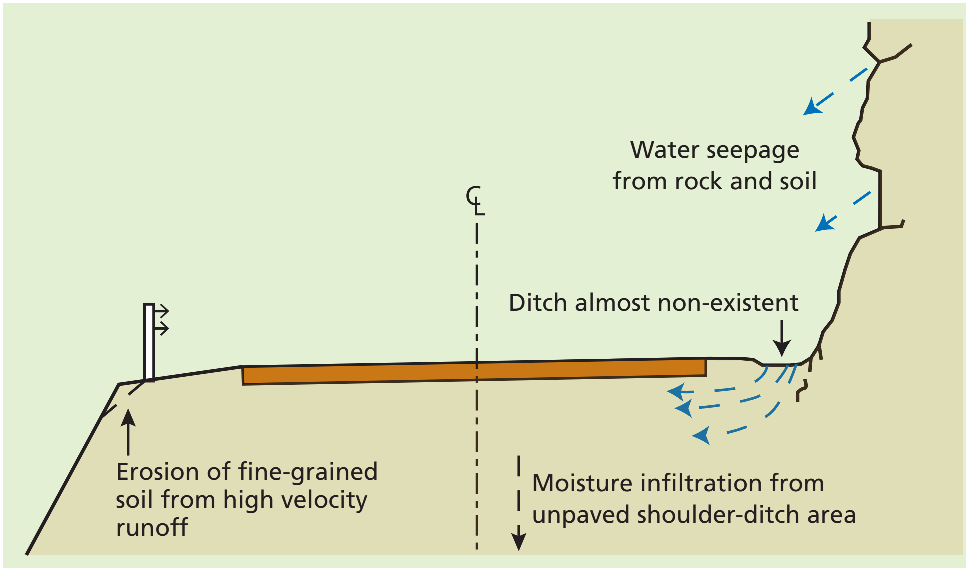

Figure 12-11: Inadequate side drains

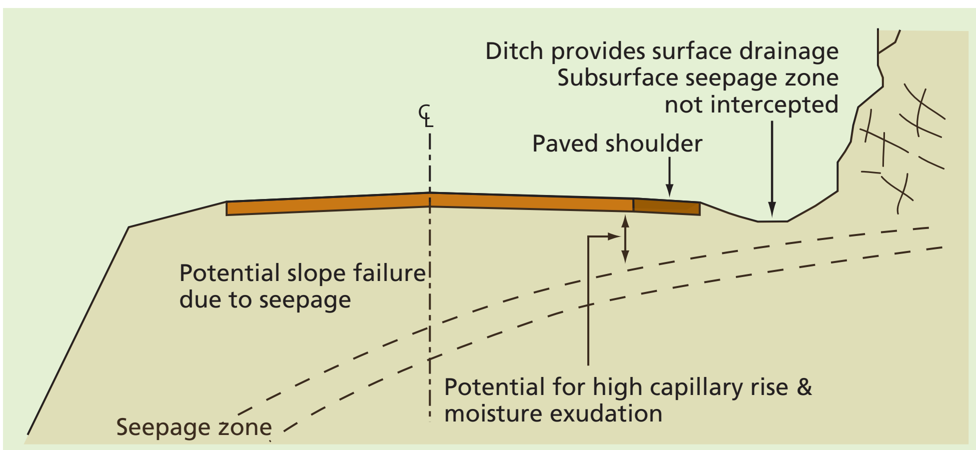

Figure 12-12: Inadequate side drains and subsurface drainage

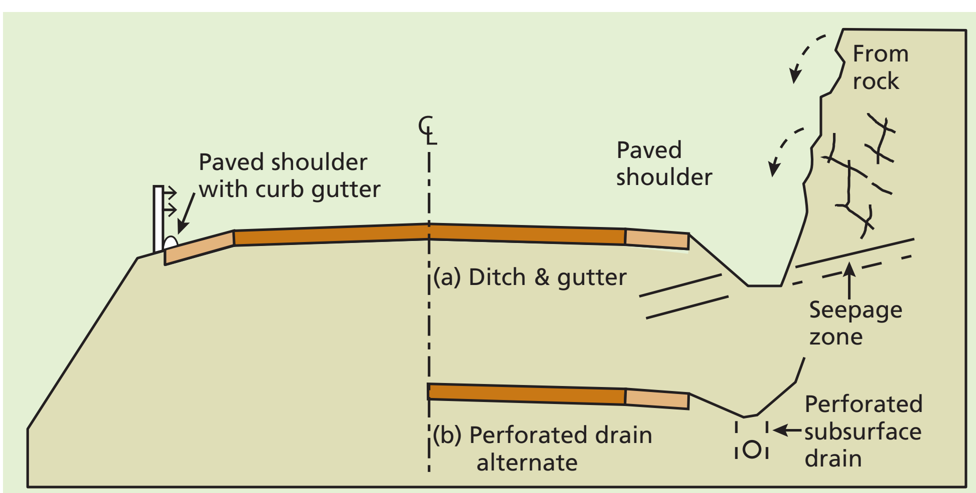

Figure 12-13: Proper interception of surface runoff and subsurface seepage

12.5 TYPES OF EROSION

12.5.1 General

Parts of the drainage system and surrounding terrain are subject to erosion if the quantity and velocity of the flowing water is above critical values. Engineering measures must be taken to prevent serious erosion both close to the road itself but also for some distance away from the road where the drainage system has concentrated the water flow.

Erosion of soil is caused by water travelling on the surface of the soil at a velocity that produces stresses in the soil that exceed the soils cohesion and therefore its resistance to movement. This, in turn, is influenced by a wide range of factors including:

- Physical factors. These include soil type, geology, and climate, particularly rainfall.
- Road location. The location of the road in relation to slope, stream channels, and sensitive soils has a direct effect on erosion and the amount of sediment that needs to be controlled.
- Road standards and construction. Designed road width, steepness of cut banks or road fills, methods of construction, and drainage installations will directly affect the area of disturbance and potential for failure following road construction.

12.5.2 Erosion Problems

Erosion problems may be categorized based on the mitigation measures required in the three distinct areas of the road (Figure 12-14) namely:

- The upper catchment (the area draining towards the road).
- The road reserve.
- The lower catchment area (the area on the down-stream side of the road reserve.

Figure 12-14: Erosion areas–upper catchment, road reserve and lower catchment
Source: Erikson and Kidanu (2010).

Upper catchment area

Washaways or erosion in the road reserve are generally caused by runoff from the upper catchment area, often causing damage to side slopes, drains and drainage structures. The main factors contributing to these problems include:

- High runoff rates caused by naturally low water retention capacity in the catchment areas due to poor vegetation cover and/or impervious soils and/or hilly terrain.
- Excessive water runoff as a result of poor land management practices resulting in low water retention capacity in cultivated fields and grazing land and from built up areas (roofs, foot paths and cattle tracks, etc.).
- Runoff from cultivated land, grazing areas, footpaths, cattle tracks, and so on, adjacent to a road. This carries sediment and transports it towards the road resulting in siltation of the road drainage structures.
- Lack of appropriate erosion control measures.

Road reserve

Typical soil erosion problems in the road reserve include:

- Scouring/gullying in side drains.
- Scouring in culvert inlets and outlets.
- Gullying on culvert outlets.
- Scouring of bridges' wing-walls and abutments.
- Siltation of culverts and drains.
- Slope failures on embankments.

Lower catchment area

The lower catchment refers to the area below the road reserve that receives water from the upper catchment and the road itself. This is where the most serious soil erosion problems usually occur (due to the nature of road alignments acting as barriers to natural surface run off and concentrating water flows) with serious damage to land and other properties. Typical soil erosion problems in the lower catchment areas include:

- Gullying of culvert outfalls.
- Gullying of mitre drain outfalls.
- Flooding and silt deposition causing damage to crops and property.

12.6 EROSION CONTROL MEASURES

12.6.1 General

The first step is to identify erosion prone areas such as high rainfall areas, hilly areas with unstable slopes, deforested areas and areas with easily erodible soil types. This should be followed by an assessment of the road reserve and the upper and lower catchment areas for likely or potential erosion problems due to runoff from these areas.

12.6.2 Gully Erosion

Gully erosion occurs when runoff water accumulates and rapidly flows in narrow channels during or immediately after heavy rains removing soil to a considerable depth. Typical examples of gully activity are illustrated in Figure 12-15.

Figure 12-15: Example of V-shaped gully activity (left) and U-shaped stabilizing gully (right)
Source: Erikson and Kidanu (2010).

A starting point for preventing gully erosion is to anticipate this type of erosion by providing appropriate structures and improving land use in the upper and lower catchment areas. The potential erosion problems can be determined from land use patterns, visual inspection and through community informants. The working principle should be *'arrest it before it is established and control it when it exists.'*

There are two basic principles involved for preventing gully control erosion. These are:

- Reduce the run off volumes of water entering the gulley.
- Reduce the erosive power (speed) of the water flow by installing scour checks and/or check dams (drop structures) which create steps that dissipate the energy of the water flow allowing vegetation to establish itself and stabilize the gully.

Typical measures for the control of gullies are shown in Table 12-7.

Table 12-7: Typical gully control measures

Type of gulley	Suggested control measures
V-Shaped gullies	Install check dams (details in Section 12.6.4) and plant grass in trapped sediments.
U-shaped gullies	No work is needed in most U-shaped gullies since they do not manifest active erosion and usually stabilize on their own. If necessary, however, prevention of gully bank (side wall) erosion on bends through stone lining and/or establishing suitable vegetation along the foot of the bank is required. Experiences in several countries have shown that planting suitable grass like Vetiver in gullies proved to be the fastest and cheapest way of stabilising erosion damages.
Head of gulley	Dam construction downstream of the fall. Sloping and protecting the channel. Construction of drop structures (steps that dissipate energy of the water flow).

12.6.3 Protection of Drains

Unlined drains

Critical length: This is defined as the maximum length of an unlined ditch in which water velocities do not give rise to erosion. The maximum velocity of water can be calculated from the slope, shape and dimensions of the ditch, volume of water and from the roughness coefficient of the material (refer to Section 11.4). The recommended maximum permissible velocities for different types of material to prevent scour in un-lined drains are given in Table 11-15. Knowing the maximum permissible velocity for each type of material, the maximum length of unlined ditch in this material can then be determined.

Where flow velocities exceed those shown in Table 11-15 and are likely to cause scouring, the velocity in the drains must be reduced. Methods include:

- Scour checks.
- Grassed waterways.
- Drain lining.

Scour checks: Act as small dams and, when naturally silted up on the upstream side, effectively reduce the gradient of the drain on that side, and therefore the velocity of the water. Scour checks are usually constructed with natural dry-packed stone, stone masonry, concrete or with wooden stakes (e.g. bamboo) in combination with dry-packed stones.

The level of the scour check must be a minimum of 200 mm below the edge of the carriageway in order to avoid the water being diverted out of the side drains.

Typical designs for scour checks are shown in Figure 12-16.

The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table 12-5 (Section 12.3.5) shows recommended values. In areas that evidently have erodible soils, it is recommended that the drains are lined at gradients above 3%. The preferred shape of lined side drains is trapezoidal.

Lined Drains

Particularly in mountainous terrain, where road gradients reach 8 to 10 percent, it may be necessary to consider lining the side drains in the steep sections to avoid severe erosion, as shown in Figure 12-17. Drain lining can be made from mass concrete, concrete blocks, stone masonry or brick masonry. Rock, if available in the vicinity of the road, is the preferred (lowest cost) option and can be laid as dry or wet masonry, as shown in Figure 12-18. The size of the stone should be a minimum of 200 mm to avoid the rock being washed away by water. The masonry work needs to be well laid to ensure that water does not enter underneath the lining allowing it to become unstable, undercut and eventually wash away.

12.6.4 Protection of Outfalls

In principle, unless culverts and mitre drains discharge directly into a natural water course, onto a non-erodible area, or into water harvesting structures, there is need to construct artificial waterways to conduct the runoff safely to valley bottoms where it can join a stream or river. These waterways are usually aligned straight down a slope (perpendicular to the contours). Where there is a natural depression or small valley that is well-stabilised with vegetation (natural waterway) road drainage water can be directly discharged but if there is no such natural waterway, an artificial waterway must be installed. It may be necessary to grass the waterway as illustrated in Figure 12-19. On relatively steep slopes, it may be necessary to stabilise the waterway with check dams which are similar in principle to scour checks, as shown in Figure 12-20.

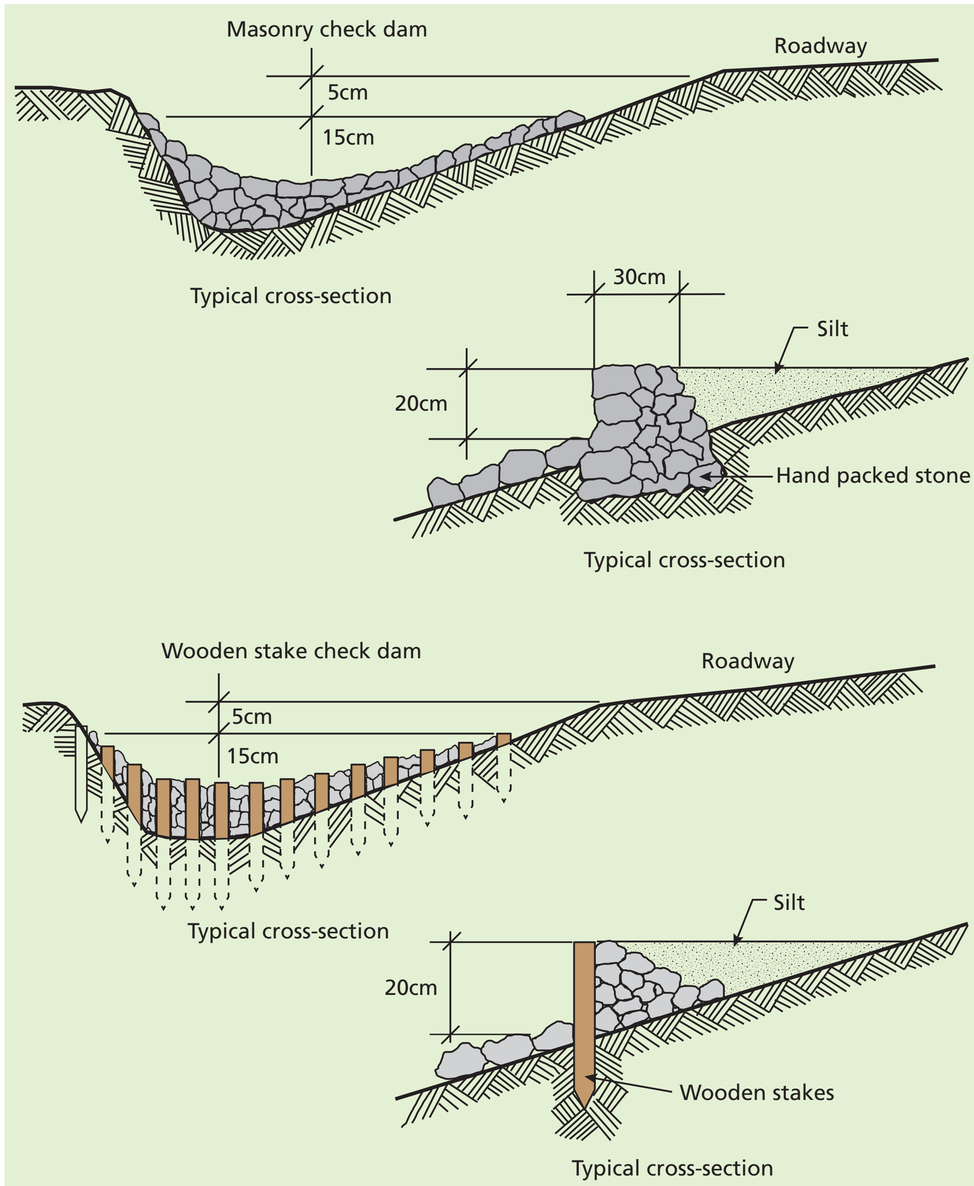

Figure 12-16: Typical design of scour checks

Figure 12-17: Severe erosion

Figure 12-18: Concrete lined drain

Figure 12-19: Grassed waterway

Figure 12-20: Grassed waterway with check dam

Source: Erikson and Kidanu (2010).

12.6.5 Protection of Slopes

It is generally not appropriate to rely on the eventual re-establishment of natural vegetation to protect the side slopes of roads, particularly where they are steep and located in high rainfall areas. In such situations, the use of appropriate bio-engineering solutions is recommended.

Bio-engineering can be broadly defined as the use of vegetation, either alone or in conjunction with engineering structures and non-living plant material, to reduce erosion and shallow-seated instability on slopes. In bio-engineering applications there is an element of slope stabilisation as well as slope protection in which the principal advantages are:

- Vegetation cover protects the soil against rain splash and erosion, and prevents the movement of soil particles down slope under the action of gravity.
- Vegetation increases the soil infiltration capacity, helping to reduce the volume of runoff.
- Plant roots bind the soil and can increase resistance to failure, especially in the case of loose, disturbed soils and fills.
- Plants transpire considerable quantities of water, reducing soil moisture and increasing soil suction.
- The root cylinder of trees holds up the slope above through buttressing and arching.
- Tap roots or near vertical roots penetrate into the firmer stratum below and pin down the overlying materials.
- Surface run-off is slowed by stems and grass leaves.

Examples of bio-engineering solutions typically employed include the following:

- The use of Vetiver grass for stabilising terraces and gullies.
- The use of trees, shrubs and other grasses to stabilise slopes, protect embankments, and provide live check structures in drains.

Figure 12-21 illustrates bio-engineering measures aimed at controlling erosion on moderate slopes.

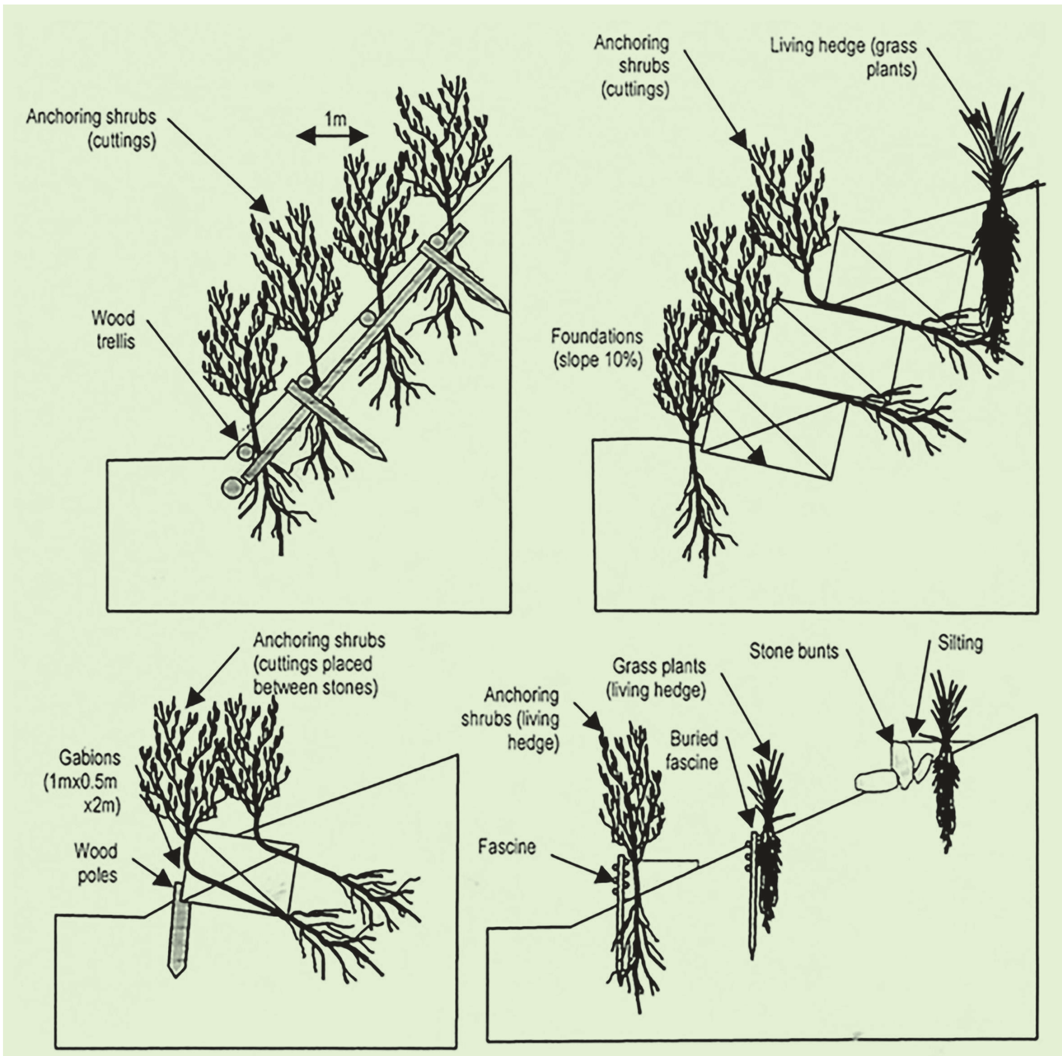

Figure 12-21: Bio-engineered slope protection measures
Source: Lebo and Schelling (2001).

Key factors in plant selection
The main factors are:

- The plant must be of the right type to undertake the bio-engineering technique that is required. The possible categories include:
  - A grass that forms large clumps.
  - A shrub or small tree that can be grown from woody cuttings.
  - A shrub or small tree that can grow from seed in rocky sites.
  - A tree that can be grown from a potted seedling.

- The plant must be capable of growing in the location of the site (i.e. water requirements and slope angle).There is no single species or technique that can resolve all slope protection problems.
- It is always advisable to use local species which do not invade and harm the indigenous environment, and which have been shown to be capable of protecting the slopes from sliding in the past.
- Large trees are suitable on slopes of less than 3H:2V or in the bottom 2 m of slopes steeper than 3H:2V. Maintaining a line of large trees at the base of a slope can help to buttress the slope and reduce undercutting by streams.
- Grasses that form dense clumps generally provide robust slope protection in areas where rainfall is intense. They are usually best for erosion control, although most grasses cannot grow under the shade of a tree canopy.
- Shrubs (i.e. woody plants with multiple stems) can often grow from cuttings taken from their branches. Plants propagated by this method tend to produce a mass of fine, strong roots. These are often better for soil reinforcement than the natural rooting systems developed from a seedling of the same plant.
- In most cases the establishment of full vegetation cover on unconsolidated fill slopes may take one to two rainy seasons. Likewise, the establishment of full vegetation on undisturbed cut slopes in residual soils and colluvial deposits may need 3 to 5 rainy periods. Less stony and more permeable soils have faster plant growth rates, and drier locations have slower rates. Plants do not establish easily on slopes steeper than 1V:1H.
- Plant roots cannot be expected to contribute to soil reinforcement below a depth of 500 mm.
- Plants cannot be expected to reduce soil moisture significantly at critical periods of intense and prolonged rainfall.
- Grazing by domestic animals can destroy plants if it occurs before they are properly grown. Once established, plants are flexible and robust. They can recover from significant levels of damage (e.g. flooding and debris deposition).

Site preparation: Before bio-engineering treatments are applied, the site must be properly prepared. The surface should be clean and firm, with no loose debris. It must be trimmed to a smooth profile, with no vertical or overhanging areas. The object of trimming is to create a semi-stable slope with an even surface to form a suitable foundation for subsequent works.

The soil and debris slopes must be trimmed to the final desired profile, with a slope angle of between 30° and 45°. (In certain cases the angle will be steeper, but this should be carefully reviewed in each case). Excessively steep sections of slope must be trimmed off, whether at the top or bottom. In particular, slopes with an over-steep lower section should be avoided since a small failure at the toe can destabilise the whole slope above.

All small protrusions and unstable large rocks must be removed. Indentations that make the surrounding material unstable must be eradicated by trimming back the whole slope around them. If removing indentations would cause an unacceptably large amount of work, they should be excavated carefully and a buttress wall built. All debris must be removed from the slope surface and toe and taken to an approved tipping site. If there is no toe wall, the entire finished slope must consist of undisturbed material.

Recommended techniques: Table 12-8 indicates the different types of bio-engineering techniques recommended for various kinds of slopes and soil materials for both cut and fill situations.

Table 12-8: Recommended general bio-engineering procedures

Site characteristics	Recommended techniques
Cut-Slopes	
Cut slopes in soil, very highly weathered rock or residual soil, at any grade up to 1H:2V.	Grass planting in lines, using slip cuttings. Only likely to be successful in wet areas where slope is > 1H:1V.
Cut slopes in colluvial debris, at any grade up to 1H:1V (steeper than this would need a retaining structure).	
Trimmed landslide head scarps in soil, at any grade up to 1H:2V.	
Roadside lower edge or shoulder in soil or mixed debris.	
Cut slope in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V.	Direct seeding of shrubs and trees in crevices.
Trimmed landslide head scarps in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V.	
Fill Slopes	
Fill slopes and backfill above walls without a water seepage or drainage problem; these should first be re-graded to be no steeper than 3H:2V.	Brush layers (live cuttings of plants laid into shallow trenches with the tops protruding) using woody cuttings from shrubs or trees.
Debris slopes underlain by rock structure, so that the slope grade remains between 1H:1V and 4H:7V.	Palisades (the placing of woody cuttings in a line across a slope to form a barrier) from shrubs or trees.
Other debris-covered slopes where cleaning is not practical, at grades between 3H:2V and 1H:1V.	Brush layers using woody cuttings from shrubs or trees.
Fill slopes and backfill above walls showing evidence of regular water seepage or poor drainage; these should first be re-graded to be no steeper than about 3H:2V.	Fascines (bundles of branches laid along shallow trenches and buried completely) using woody cuttings from shrubs or trees, configured to contribute to slope drainage.
Large and less stable fill slopes more than 10 m from the road edge (grade not necessarily important, but likely eventually to settle naturally at about 3H:2V).	Truncheon cuttings (big woody cuttings from trees).
The base of fill and debris slopes.	Large bamboo planting; or tree planting using seedlings from a nursery.

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13.1 INTRODUCTION

13.1.1 Background

Considerable research has been carried out in the region showing that LVRs can be built successfully using local materials that do not meet the standard specifications found in most design manuals. Research has been carried out on a number of roads in Tanzania and many such materials have been found to be fit for purpose as described in *Chapter 7 – Construction Materials.* The appropriate structural designs associated with these materials differ from those used for conventional higher class roads and it is these designs that are described in this chapter. The main objective of providing suitable structural layers in a pavement structure is to distribute the loads applied by traffic (axle and wheel loads) so as to avoid overstressing the in situ subgrade conditions as illustrated in Figure 13-1.

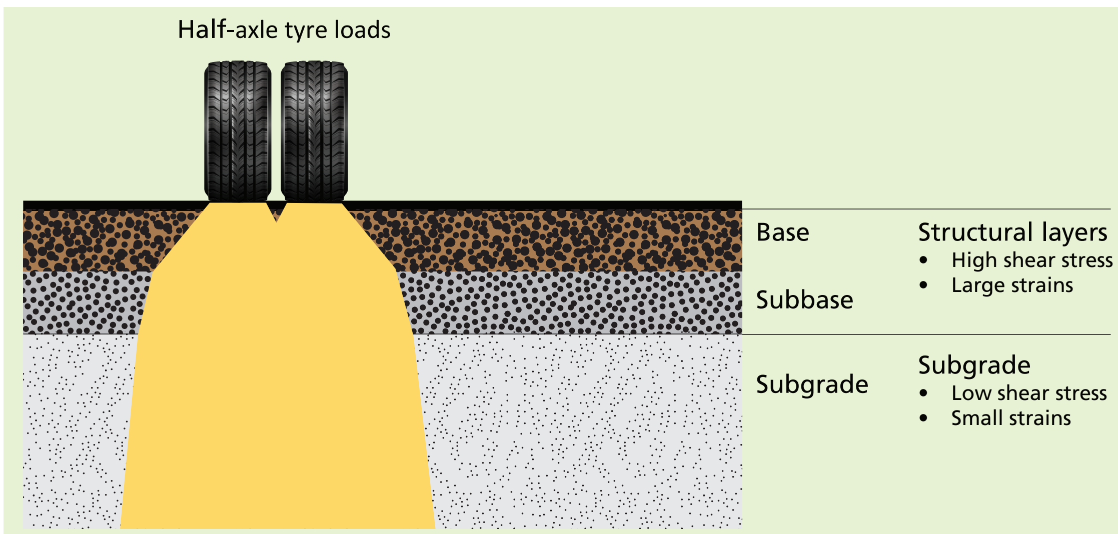


Figure 13-1: Use of stiff upper pavement layers to distribute stress on subgrade

The design methods described are adapted to make maximum use of the local in situ materials and any compaction that has affected these materials from previous trafficking.

The surfacing of many types of low volume sealed roads is a thin flexible bituminous layer designed to produce a durable and waterproof seal. The structural design of such roads is identical because the surfacing adds no significant structural strength.

There are also thicker surfacings that are used for LVRs that provide a structural component and therefore the structural design for such roads is different. The design of such roads is dealt with in Section 13.4.

13.1.2 Purpose and Scope

The purpose of this chapter is to provide details of the structural design of low volume roads that are paved. Gravel and earth roads are dealt with in *Chapter 14 – Structural Design: Unpaved Roads.* Two methods are described and both are essentially 'catalogue' methods. The catalogue method is the most convenient and, indeed, the most common method of design. For each type of structure, designs have been produced based on experimental and empirical evidence for a range of subgrade strengths and a range of traffic loading levels. The chapter describes how the designer must first obtain the key characteristics of subgrade strength and then relate these to the expected traffic loading and the properties of the available materials (some of which may be within an existing road or track) to define the required pavement design.

The scope of this chapter is thus to identify the structural requirements of the pavement in terms of the required layer thicknesses and material quality for different traffic categories. Two methods are described, both accounting for the expected in situ moisture regimes. All of the activities required to carry out the structural designs, many of which have been dealt with in the preceding chapters, are brought together in this chapter. The potential to use a wide range of non-structural and structural surfacings in relation to the pavement structures are also discussed.

13.2 DESIGN OF LOW VOLUME ROADS

13.2.1 General

The general approach to the design of LVRs differs in a number of respects from that of HVRs. For example, conventional pavement designs are generally directed at relatively high levels of service requiring numerous layers of selected materials. However, significant reductions in the cost of the pavement for LVRs can be achieved by reducing the number of pavement layers and/or layer thicknesses by using local materials more extensively as well as lower cost, more appropriate, surfacing options and construction techniques. Both the DCP-DN and DCP-CBR methods of pavement design described below provide the potential for achieving optimum design of LVRs.

For upgrading an existing road, the design engineer must also measure the strength of the existing road structure and determine the strengths and layer thicknesses required for the new structure based on the design catalogue and the associated specifications. A comparison of the existing situation with the required structure then provides the engineer with the information required to design the additions and modifications that are necessary.

Both the DCP-DN and DCP-CBR methods are suitable for entirely new roads and also for upgrading an existing road to a higher standard. Moreover, in both methods, the DCP is normally used for characterizing the strength of the existing in situ materials. However, as indicated in Section 13.2, for assessing the strength of the imported pavement materials, the DCP-DN method requires only the use of the DCP test, whilst the DCP-CBR method requires the use of the CBR. As discussed in *Chapter 7 – Construction Materials*, two design methods are presented. It is good engineering practice to use two design methods and compare the resulting designs to check that they are similar and both reasonable. There should only be minor differences between the designs produced, or else different design assumptions have been included, in which case the most realistic one should be employed.

When there is an existing track or an existing gravel road that is being upgraded, it is often found that the lower layers of such roads have been compacted by traffic over the years and may be stronger than could be obtained with new construction. It is therefore beneficial if these layers can be retained when the upgrading is carried out. This will usually be very cost-effective. Some minor realignment may be required, and therefore some part of the road will be entirely new, but both design methods make full use of existing layers provided they meet design and material criteria.

Figure 13-2 shows the design options discussed in this chapter.

The DCP method of measuring the strength of road materials has a margin of measurement error which is less than that of a CBR measurement and the test is very much quicker and cheaper. Indeed, in situ CBR tests in the road are very difficult and time consuming and are seldom carried out as required. Using the DCP enables many tests to be done and greatly improves the accuracy and reliability of the subsequent designs.

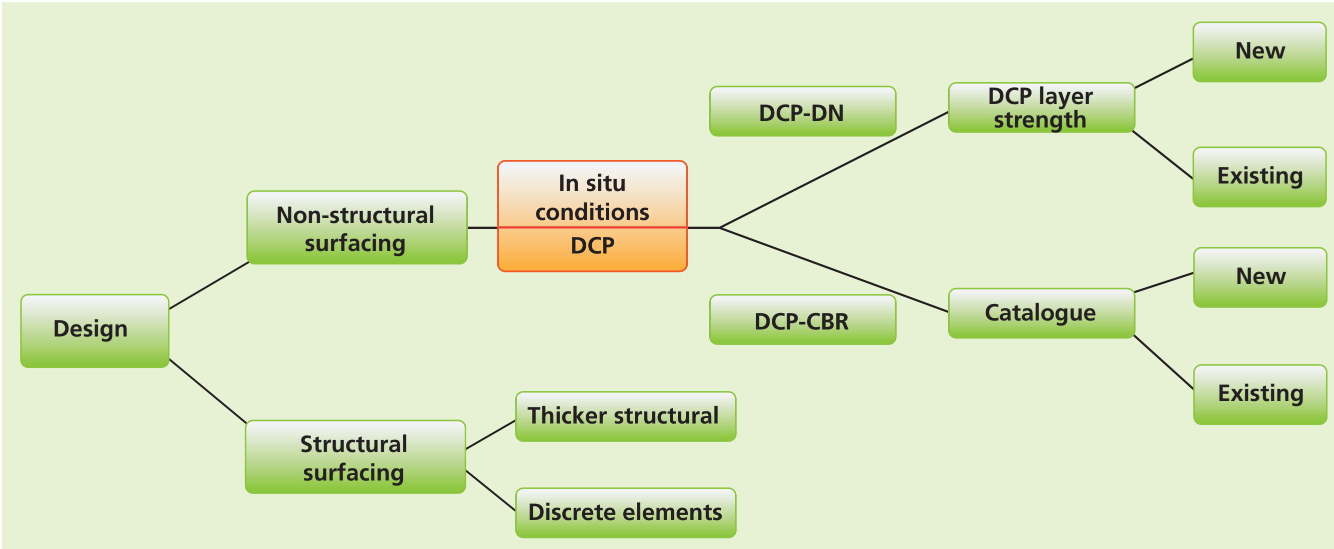


Figure 13-2: Design options available

Usually the interpretation of the DCP results is done automatically in the DCP software programs. However, and very importantly, when upgrading an existing road the DCP results can often reveal pavements that do not follow the 'normal' structure whereby the strength of each layer decreases with depth. For example; the existing road may have been constructed on top of an old road, which can be stronger; drainage problems may give rise to a weak layer within the pavement; the pavement may be fairly weak but also very thick resulting in a high but misleading structural number; the top layer of a gravel road may be weaker than the underlying layer. In other words the DCP results often provide important information about the existing road that requires interpretation that cannot be automated.

Sometimes additional investigations may be required, for example, to identify drainage problems and determine remedial treatments and, quite commonly, to determine whether and how a weak top layer can be improved by processing in some way (maybe merely compaction). The most common problem is that a layer is too weak and outside the specification for its position in the structure. Engineering judgement or a trial is required to determine whether such a layer can be strengthened or whether it is better to add a new roadbase thereby pushing the deficient layer to a lower point in the pavement where it should be acceptable.

The primary differences between the two methods are summarized in Table 13-1, and the various issues are discussed in detail later in the chapter. The pavement balance concept, that is fundamental to the DCP-DN method, ensures that there is a strong base over progressively weaker underlying layers and the ratio of the strengths of two adjacent layers is not too high.

Table 13-1: Comparison of DCP-DN and DCP-CBR methods

Property	DCP-DN method	DCP-CBR method
Samples	Random subgrade samples for moisture content (MC) and compaction testing.	Regular samples for MC and compaction testing. Samples from 3 layers to 450 mm depth.
Strength	Use DCP to assess in situ conditions. Use DCP penetration rate (DN) directly (in situ strength). No modifications required.	Use DCP to assess in situ conditions. Requires conversion of DN to CBR. CBR converted to soaked values. Soaked CBR converted to layer strength coefficients for SN.
Uniform sections	CUSUM based on actual DN and DSN800 values of each point.	CUSUM based on SN deficiency of each individual point or based on any of the parameters obtained from the DCP test.
Layers	150 mm layers with weighted average strength analysed.	Variable layer thicknesses with average strength. Analyses for multiple layers (bases, subbases and subgrade(s).
Design	Uses in situ strength and variable strength for base. Variable percentile used depending on in situ moisture regime and traffic. In-service moisture regime estimated from visual survey.	Requires minimum soaked CBR of 45% for base. For strengthening an existing road the 90th, 75th or 50th percentile of the additional SN required is used, depending on traffic.

13.2.2 DCP-DN Method

This method is based entirely on using the DCP and does not introduce variations related to converting the results to equivalent CBR values. The in situ DN values obtained from a survey of the proposed road are plotted on a chart versus the depth and are compared directly with a related DCP design catalogue. In addition, any laboratory strength testing required is also carried out with a DCP on specimens compacted into moulds in the laboratory. The DCP-DN method of design for LVRs has been developed as a relatively simple, practically oriented and robust alternative to the traditional CBR-based methods (refer to Section 7.4.3).

The flow diagram for the DCP-DN method is shown in Figure 13-3.

For a new road the method is similar but there are no existing pavement layers. In this case only the subgrade properties are determined. The required pavement structures are then obtained directly from the catalogue of structures.

13.2.3. DCP-CBR Method

This is the traditional method based on the CBR test, but because of its many advantages the designer would normally make appropriate use of a dynamic cone penetrometer (DCP) to obtain much of the required design information, particularly a longitudinal profile of in situ strengths of the pavement layers of the existing road in terms of DN values (penetration per blow in mm/blow). The DN number is normally converted to CBR so that a diagram of CBR versus depth is obtained. The equation used to do this is based on the BS method of CBR testing and is:

$$\mathrm{Log}_{10}\mathrm{CBR} = 2.48 - 1.057\ \mathrm{Log}_{10}\ \mathrm{DN}$$

In order to make optimum use of the existing layers the method makes use of the structural number concept (AASHTO, 1993). Using this method the difference between the structural number of the existing road and that required for the upgraded road, obtained from the catalogue of structures, defines the additional requirements for upgrading, rehabilitation or reconstruction. The flow diagram for upgrading an existing road is shown in Figure 13-4.

Uniform (or relatively homogenous) sections can be determined at this stage using the 'CUSUM' method applied to the DCP data, usually the SN or adjusted structural number (SNP) values, and the required designs can be determined based on the appropriate percentiles. However, the designs are best determined for every DCP test point with no averaging or calculations of percentiles at this stage. Only when the final designs are completed is a percentile of the required strengthening requirements selected for implementation.

For a new road the method is somewhat simpler because there are no existing pavement layers. In this case only the subgrade properties are required. The structural designs are then obtained directly from the catalogue of structures.

The flow diagram for new road design is shown in Figure 13-5.

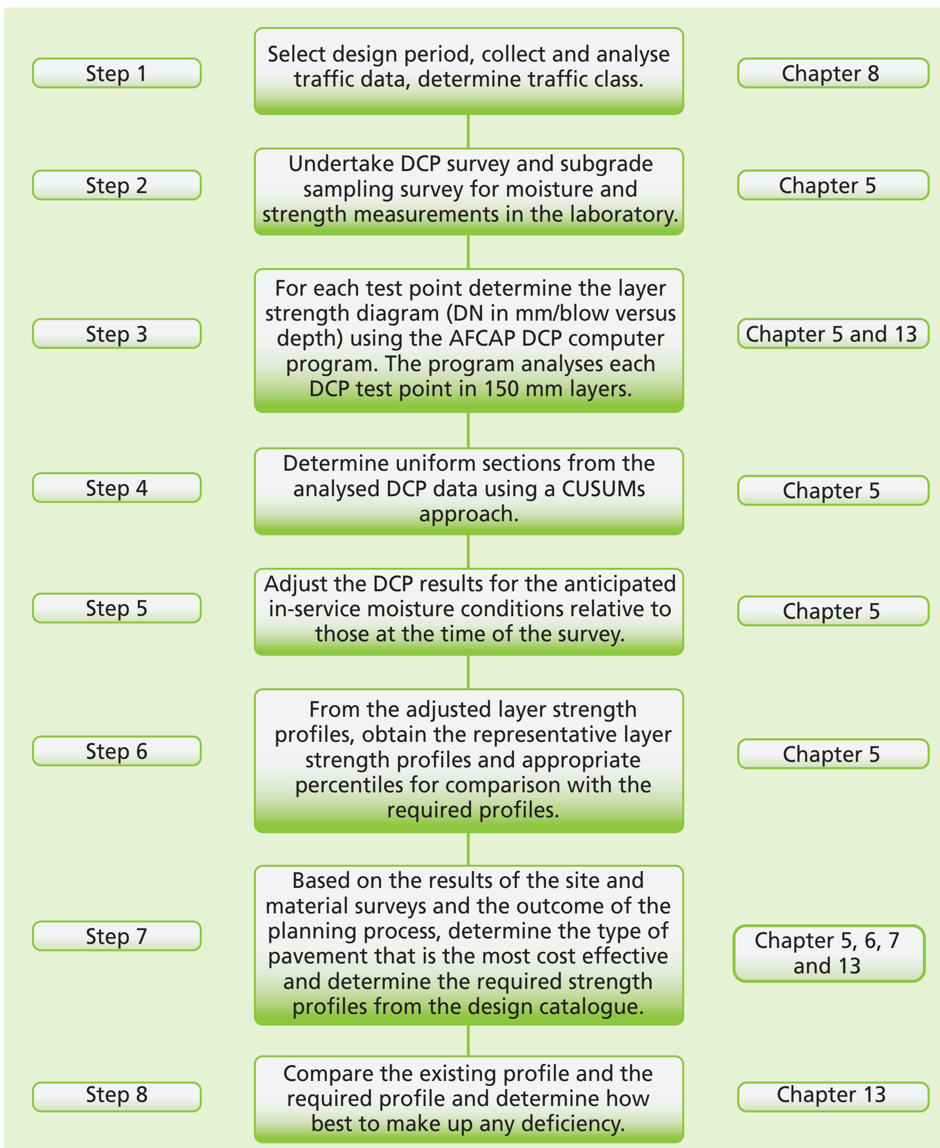


Figure 13-3: Flow diagram for the DCP-DN method

Step 1	Select design period, collect and analyse traffic data, determine traffic class.	Chapter 8
Step 2	Undertake a DCP survey plus a sampling survey for moisture and strength measurements in the laboratory.	Chapter 5
Step 3	For each test point determine the layer strength diagram (DN in mm/blow versus depth and CBR versus depth) and the layer boundaries using the computer program (1).	Chapter 5 and 13
Step 4	Determine the in situ SN values for each layer and the total SN (and SNP) for the pavement at each test point using the DCP program (1, 2).	Chapter 13
Step 5	Convert the CBR values (and hence the SN values) from in situ to soaked conditions for comparison with the design catalogue.	Chapter 13
Step 6	Determine the required SN values at each test point from the catalogue appropriate to the climate, traffic and subgrade strength.	Chapter 13
Step 7	Compare the in situ with the required SN to determine the structural deficiency at each test point.	Chapter 13
Step 8	Identify areas (a) where the structural deficiency is large and (b) areas where layers are very weak and unlikely to meet the specifications for the layer that they will become in the upgraded design (3).	Chapter 13
Step 9	Identify uniform sections based on the structural deficiencies at each test point using a CUSUM method.	Chapter 13
Step 10	Determine the appropriate percentile (4) of the structural deficiencies for each uniform section and design the upgrading requirements in terms of additional layers and/or layer processing.	Chapter 7 and13

Figure 13-4: DCP-CBR method - Flow diagram for upgrading an existing road

Notes:
1. These calculations can be done by hand using a spreadsheet but the UK DCP 3.1 program makes this easy and straightforward.
2. The SNP values are useful when the subbase and/or subgrade comprise a number of different layers of varying strengths.
3. Areas with material that does not meet the specifications for the layers that they will normally become in the upgraded design will require some form of reconstruction (or additional strengthening layers to increase the depth of the inadequate layers in the new pavement).
4. Depends primarily on traffic level.

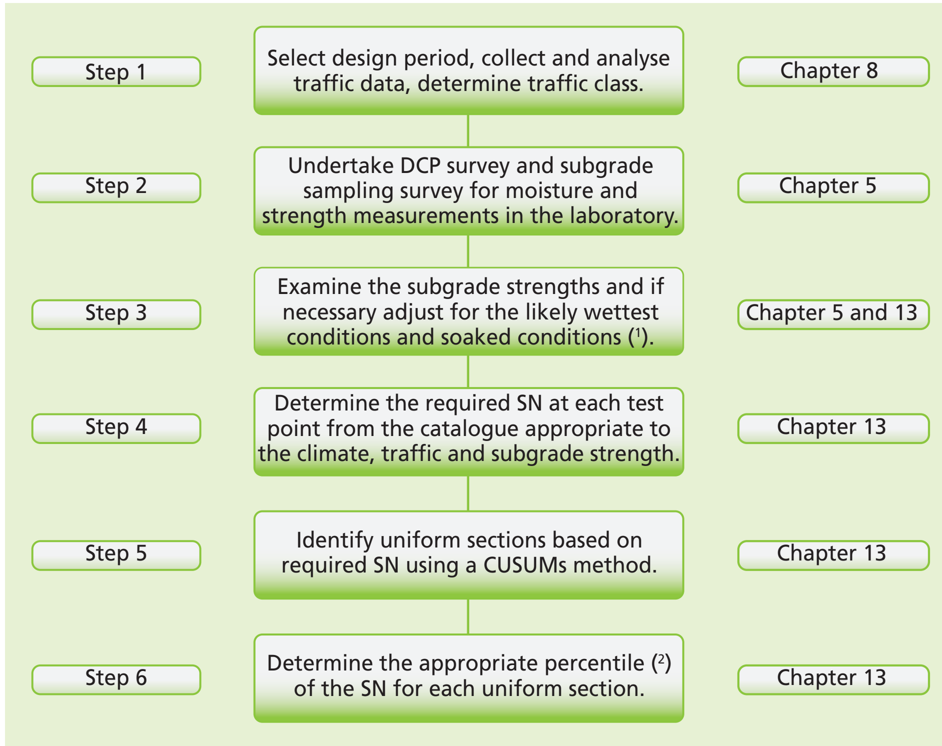


Figure 13-5: DCP-CBR method - Flow diagram for designing a new road

Notes: 1 These calculations can be done using a spreadsheet but the DCP program makes it simpler.
2 Depends primarily on traffic level.

The design should be undertaken in an environmentally optimized manner as described in *Chapter 1 – General Introduction* which ensures that the use of materials and the pavement design are matched to the road environment at a local level.

Worked examples of the two design methods are included at the end of the chapter.

13.3 DESIGN OF ROADS WITH NON-STRUCTURAL SURFACES

13.3.1 General

The majority of paved (or sealed) LVRs will be surfaced with a thin waterproof layer which adds no significant strength to the whole structure and the structural designs are thus the same for all such thin surfacings. These are generally much less costly and easier to design than traditional asphalt or structural surfacings.

13.3.2 DCP-DN Method

Design Approach

The approach behind the DCP-DN design method is similar to that of the DCP-CBR method. It is to achieve a balanced pavement design whilst optimising the use of the in situ material strength as much as possible. This is achieved by:

1. Determining the design strength profile needed for the expected traffic, and
2. Integrating this strength profile with the in situ strength profile.

To use the existing gravel/earth road strength that has been developed over the years, the materials in the pavement structure need to be tested for their actual in situ strength, using a DCP as described in *Chapter 5 – Site Investigations.* The result of each DCP test is a diagram of the strength of the existing pavement measured as DN values as a function of depth as illustrated in Figure 13-6.

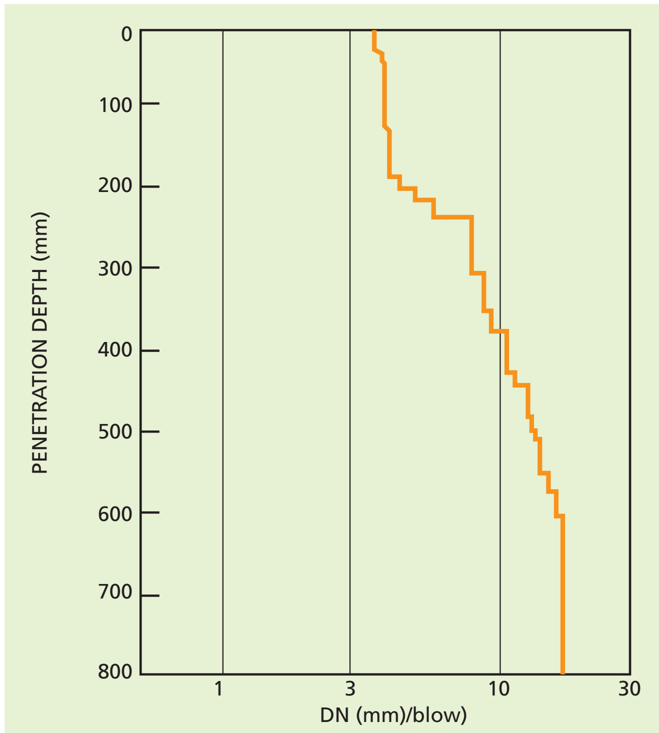


Figure 13-6: Typical DN with depth profile

The rate of penetration is a function of the in situ shear strength of the material at the in situ moisture content and density of the pavement layers at the time of testing, as described in *Chapter 5 – Site Investigations.* However, most methods of pavement design require an estimate of the values of strength that would be obtained under the worst possible conditions: hence it is always recommended that DCP testing is done at the height of the wet season. If this cannot be achieved, the method of adjustment described in Section 13.3.4 can be used.

Useful parameters derived from the DCP analysis are the number of blows DN150 required to penetrate the top 150 mm of the pavement and the number of blows required to penetrate from 150 mm to 300 mm. These are the areas of the pavement that need to be the strongest and hence these parameters provide a quick appreciation of the likely need for strengthening and are also useful for delineating uniform sections. The DN800 is the total number of blows required for the DCP to penetrate to 800 mm and gives a broad measure of overall strength of the pavement somewhat analogous to the AASHTO Structural Number.

The analysis procedure for the DCP-DN method is different to that used in the DCP-CBR method and is described below.

Design Procedure

The main elements are summarised in the flow diagram, Figure 13-3. The details from other chapters are not repeated here, thus Steps 1 to 5 and part of Step 6 are assumed to have been completed.

Step 6 Determining the in situ layer strength profile for each uniform section: This is based on an "average analysis" for each uniform section as undertaken by the computer program (AFCAP DCP), which uses the data from all of the DCP profiles included in that uniform section. The layer strength (DN) profiles for each uniform section are plotted as shown in Figure 13-7 (all data) and Figure 13-8 (average, maxima and minima). Various percentiles of the layer strengths (DN values) to be used in the design process can be selected in the program and are computed automatically. Manual selection is based on the expected moisture conditions as discussed previously and summarized in Table 13-2.

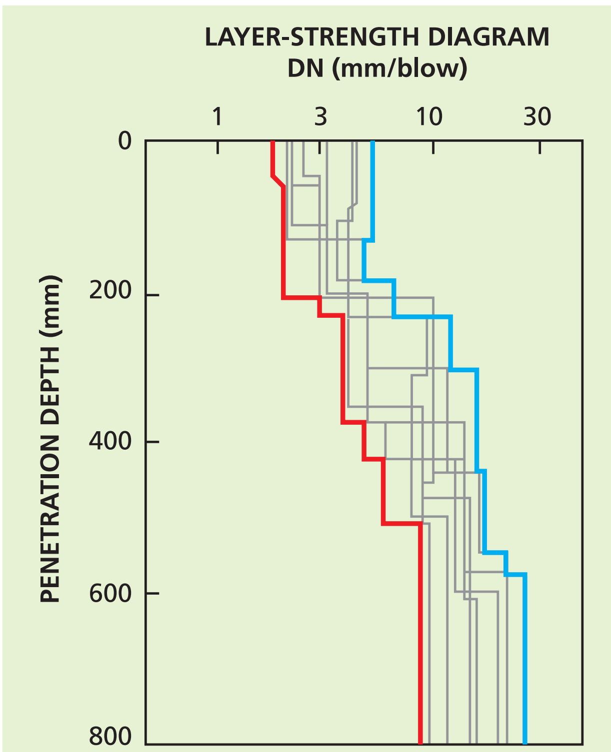


Figure 13-7: Collective strength profile for a uniform section

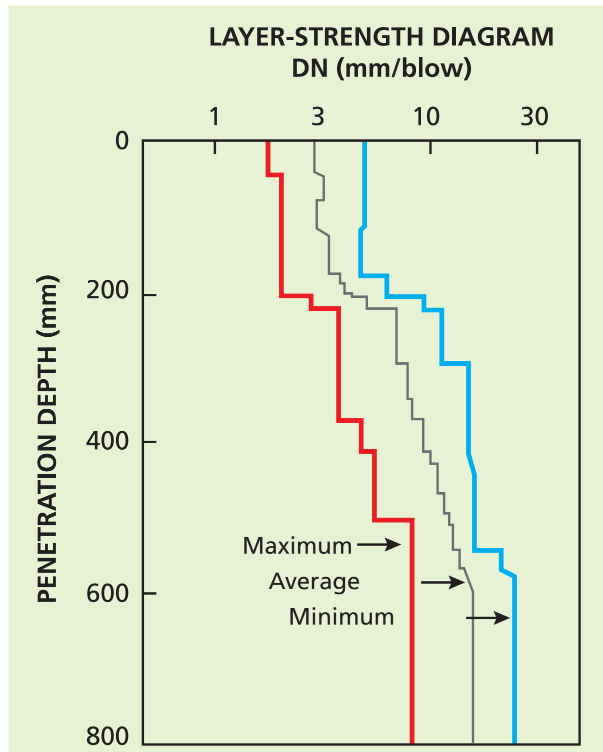


Figure 13-8: Average & extreme DCP strength profiles for a uniform section

Table 13-2: Suggested percentile of in situ DCP penetration rates to be used

Site moisture condition during DCP survey	Percentile of strength profile (maximum penetration rate – DN)	
	Materials with strengths not moisture sensitive*	Materials with strengths that are moisture sensitive*
Wetter than expected in service	20	20 – 50
Expected in service moisture	50	50 – 80
Drier than expected in service	80	80 - 90

* Moisture sensitivity can be estimated by inspecting and feeling a sample of the material – clayey materials (PI > about 12%) can be considered to be moisture sensitive. This can be confirmed by moisture test results.

The required layer strength profile for each uniform section is determined from the DCP design catalogue which is shown in Table 13-3 and illustrated in Figure 13-10 for different traffic categories.

The design catalogue is based on the anticipated, long term, in-service moisture condition. If there is a risk of prolonged moisture ingress into the road pavement, then the pavement design should be based on the soaked or a selected wetter condition. The DN value for any selected in situ moisture condition can be estimated from Table 13-4. This should be used as a guide only and testing of the actual materials involved should preferably be carried out. The values shown in Table 13-4 can be highly material dependent, especially for moisture sensitive materials and certain other materials such as laterites and calcretes.

Table 13-3: DCP design catalogue for different traffic classes

Traffic Class mesa	TLC 0.01 0.003-0.01	TLC 0.03 0.01-0.03	TLC 0.1 0.03-0.10	TLC 0.3 0.1-0.3	TLC 0.7 0.3-0.7	TLC 1.0 0.7-1.0
0- 150 mm Base ≥ 98% Mod. AASHTO	DN ≤ 8	DN ≤ 5.9	DN ≤ 4	DN ≤ 3.2	DN ≤ 2.6	DN ≤ 2.5
150-300 mm Subbase ≥ 95% Mod. AASHTO	DN ≤ 19	DN ≤ 14	DN ≤ 9	DN ≤ 6	DN ≤ 4.6	DN ≤ 4.0
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 12	DN ≤ 8	DN ≤ 6
450-600 mm In situ material	DN ≤ 40	DN ≤ 33	DN ≤ 25	DN ≤ 19	DN ≤ 14	DN ≤ 13
600-800 mm In situ material	DN ≤ 50	DN ≤ 40	DN ≤ 39	DN ≤ 25	DN ≤ 24	DN ≤ 23
DSN_{800}	≥ 39	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

DSN800 is DCP structural number (i.e. number of blows required to reach a depth of 800 mm.

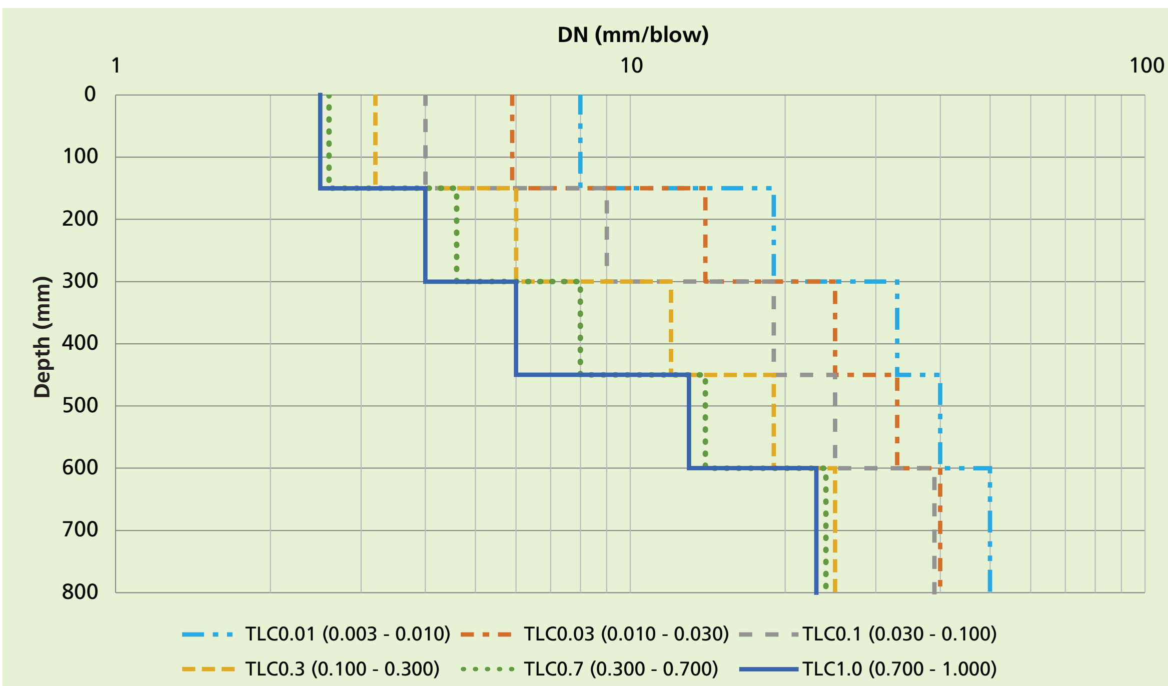


Figure 13-9: Layer strength profile for various traffic classes

Table 13-4: Relationship between standard soaked DN values and in situ DN at various moisture contents for different material strengths (CBR)

Soaked DCP DN value mm/blow	Soaked CBR	Field DCP-DN (mm/blow)					
		Subgrade		Base, subbase and selected layers			
		Wet climate	Dry climate	Very dry state	Dry state	Moderate state	Damp state
3.62	80			1.43	1.73	2.19	3.13
4.54	60			1.65	2	2.5	3.6
5.69	45			1.85	2.23	2.84	4.07
7.84	7			2.15	2.55	3.25	4.7
9.04	25	4.79	4.21	2.25	2.72	3.45	4.93
13.5	15	5.08	4.42	2.58	3.1	4	5.75
18.6	10	6.37	5.59	3.01	3.62	4.6	6.64
24.6	7	6.5	5.79				
48	3	6.94	6.12				

Notes: Moisture contents are expressed as ratios of in situ to Mod AASHTO optimum moisture contents as follows: Very dry = 0.25; Dry = 0.5; Moderate = 0.75; Damp = 1.0. This Table is only a guide and should be used with discretion. Materials that are highly moisture sensitive may produce different results.

Step 7: The representative in situ strength profiles are now compared with the required strength profile. The required strength profile is plotted on the same layer-strength diagram on which the uniform section layer strength profiles were plotted as illustrated in Figure 13-10. The comparison between the in situ strength profile and the required design strength profile allows the adequacy of the various pavement layers with depth to be assessed for carrying the expected future traffic loading.

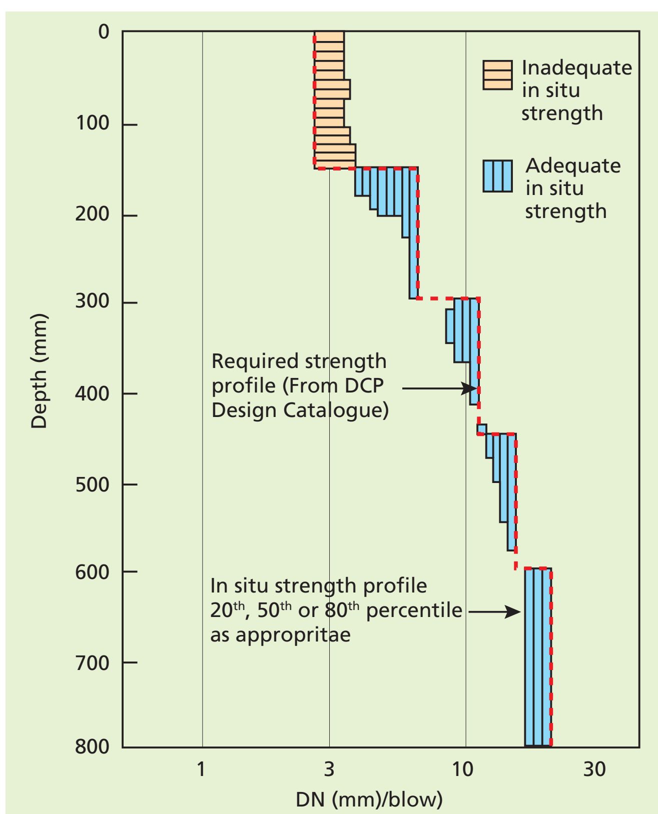


Figure 13-10: Comparison of required and in situ strength profiles

Step 8. Determining the upgrading requirements. Two options may be considered, as follows:
Option 1: If the in situ strength profile of the existing gravel road complies with the required strength profile indicated by the DCP catalogue for the particular traffic class, the road would need to be only re-shaped, compacted and surfaced (assuming that the existing road is adequately above natural ground level to permit the necessary drainage requirements).

Option 2: If the in situ strength profile of the existing gravel road does not comply with the required strength profile indicated by the DCP catalogue for the particular traffic class (as is the case in the upper 150 mm of Figure 13-10), then the upper pavement layer(s) would need to be:

- Reworked- if only the density is inadequate and the required DN value can be obtained at the specified construction density and anticipated in-service moisture content.
- Overlaid – if the material quality (DN value at the specified construction density and anticipated in-service moisture content) is inadequate, then appropriate quality material will need to be imported to serve as the new upper pavement layer(s).
- Mechanically stabilized – as above, but new, better quality material is blended with the existing material to improve the overall quality of the layer
- Augmented – if the material quality (DN value) is adequate but the layer thickness is inadequate, then imported material of appropriate quality will need to be imported to make up the required thickness prior to compaction.

If none of the above options produces the required quality of material, recourse may be made to more expensive options, such as soil stabilisation. However, the design and construction requirements of stabilised layers is outside the scope of this Manual which focuses on the use of natural, untreated, materials. Reference may be made to other texts on the subject of stabilisation, such as the Pavement and Materials Design Manual, (MOW, 1999 - *Chapter 7 – Construction Materials*) which deals with cemented materials. A fully worked example of the design DCP-DN method is included in Section 13.6.

13.3.3 DCP-CBR Method for New Roads

The approach behind the DCP-CBR design method is similar to that of the DCP-DN method, i.e. to achieve a balanced pavement design whilst optimising the use of the in situ material strength as far as possible. The method is based on DCP test results but goes through a process of converting them to CBR values and then defining the pavement structure based on a structural number concept. For roads with non-structural bituminous surfacings the design charts shown in Tables 13-5 and 13-6 are utilised.

The subgrade is classified using the standard soaked CBR test to provide a strength index. It is not expected that the subgrades will become soaked in service except in exceptional circumstances and so the design catalogues show different thickness designs based on climate and drainage conditions for the same indexed subgrade class. A standard soaked CBR test is also used to evaluate the strength of the imported pavement materials.

Two design catalogues (charts) are used and two climatic zones are defined. The use of each chart also depends on the drainage and sealing provisions and the available materials as described below.

Wet climatic zone

In the wet climatic zone, the following situations and solutions apply:

(a) Where the total sealed surface is 8 m or less, Pavement Design Chart 1 (Table 13-5) should be used. No adjustments to the roadbase material requirements are required.

(b) Where the total sealed surface is 8 m or more, Pavement Design Chart 2 (Table 13-6) should be used. The limit on the plasticity modulus of the roadbase may be increased by 20%. (refer to Figure 13-11 and Table 7-9, *Chapter 7 – Construction Materials)*.

(c) Where the total sealed surface is less than 8 m but the pavement is on an embankment in excess of 1.2 m in height, Pavement Design Chart 2 (Table 13-6) should be used. The limit on the plasticity modulus of the road base may be increased by 20%. (refer to Figure 13-11 and Table 7-9, *Chapter 7 – Construction Materials*).

If the design engineer deems that other risk factors (e.g. poor maintenance and/or construction quality) are high, then Pavement Design Chart 1 should be used.

Moderate and dry climatic zone

In a moderate or dry climatic zone Pavement Design Chart 2 (Table 13-6) should be used.

(a) Where the total sealed surface is less than 8 m, the limit on the plasticity modulus of the road base may be increased by 40%. (refer to Figure 13-11 and Table 7-9, *Chapter 7 – Construction Materials*.

(b) Where the total sealed surface is over 8 m and when the pavement is on an embankment in excess of 1.2 m in height, the plasticity modulus of the road base may be increased by up to 40% and the plasticity index by 3 units. (refer to Figure 13-11 and Table 7-9, *Chapter 7 – Construction Materials*).

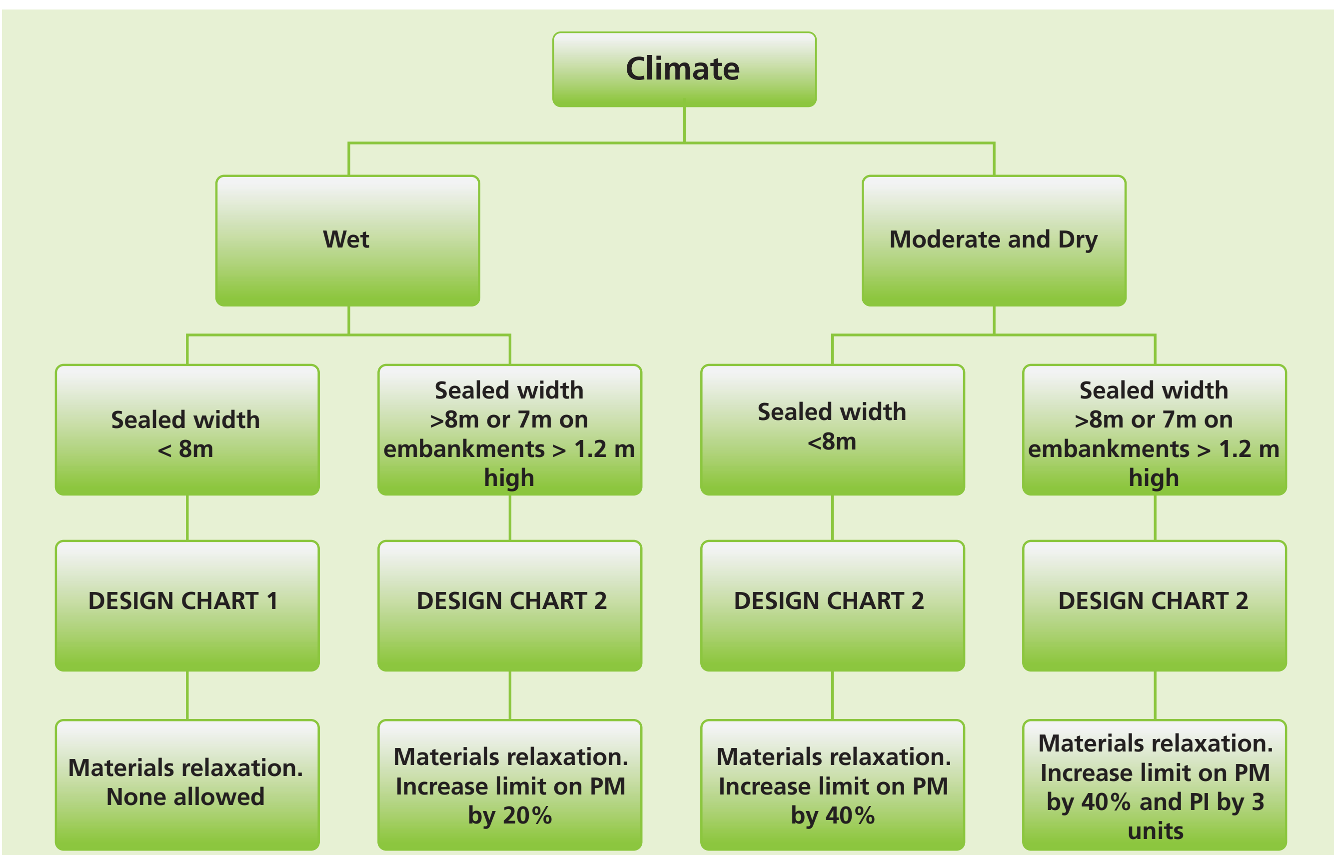


Figure 13-11: DCP-CBR pavement design flow chart

Table 13-5: Bituminous pavement design Chart 1 (wet areas)

Subgrade CBR	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	150 G45 150 G15	150 G65 125 G30 150 G15	150 G80 150 G30 175 G15	175 G80 150 G30 175 G15	200 G80 175 G30 175 G15
S3 (5-7%)	125 G45 150 G15	150 G65 100 G30 100 G15	150 G65 150 G30 125 G15	175 G65 150 G30 125 G15	200 G80 150 G30 150 G15
S4 (8-14%)	200 G45	150 G65 125 G30	150 G65 200 G30	175 G65 200 G30	175 G80 150 G30
S5 (15-29%)	175 G45	125 G65 100 G30	150 G65 125 G30	150 G65 150 G30	175 G80 150 G30
S6 (>30%)	150 G45	150 G65	175 G65	200 G65	200 G80

Table 13-6: Bituminous pavement design Chart 2 (moderate and dry areas)

Subgrade CBR	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	150 G45 150 G15	150 G65 125 G30 150 G15	150 G80 150 G30 175 G15	175 G80 150 G30 175 G15	200 G80 175 G30 175 G15
S3 (5-7%)	125 G45 125 G15	150 G55 175 G30	175 G65 175 G30	175 G80 200 G30	175 G80 250 G30
S4 (8-14%)	200 G45	150 G55 100 G30	150 G55 150 G30	175 G65 150 G30	175 G80 175 G30
S5 (15-29%)	150 G45	200 G55	125 G55 125 G30	125 G65 125 G30	150 G80 125 G30
S6 (>30%)	150 G45	175 G45	175 G55	175 G65	175 G80

Once the quality of the available materials and haul distances are known, the flow chart in Figure 13-11 and the design charts can be used to review the most economical designs.

When the project is located close to the boundary between the two climatic zones, the wetter value should be used to reduce risks. When the design is close to the borderline between two traffic design classes, and in the absence of more reliable data, the next highest design class should be used.

The design charts do not cater for weak subgrades (CBR < 3%) and other problem soils, which will need specialist input and design, typically requiring imported, better quality, selected subgrade materials.

13.3.4 DCP-CBR Method for Upgrading an Existing Road

Design approach

The DCP survey provides the thicknesses and in situ strengths of the layers of the existing road along the entire alignment. The analysis of the DCP data, preferably using the TRL DCP program, provides the overall strength of the pavement at each test point based on the structural number approach. The flow diagram is shown in Figure 13-5.

The structural number is essentially a measure of the total thickness of the road pavement weighted according to the ‘strength’ of each layer and calculated as follows:

$$SN = 0.0394 \sum m_i.a_i.h_i$$

Where:

SN	=	structural number of the pavement
a_i	=	strength coefficient of the ith layer
h_i	=	thickness of the ith layer, in millimetres
m_i	=	‘drainage’ coefficients that modify the layer strength coefficients of unbound materials if drainage is poor and/or climate is favourable or severe

The summation is over the number of pavement layers, n.

The individual layer strength coefficients are determined from the normal tests that are used to define the strength of the material in question e.g. CBR for granular materials, UCS for cemented materials etc. Table 13-7 shows typical values.

The drainage coefficients are effectively calibration factors for the moisture regime experienced by the road and are therefore related to both climate and drainage. Values range from 0.7 for extremely poor conditions up to 1.3 for very good conditions, but the usual working range is 0.9 to 1.1. In wet areas, a value of mi of 0.9 will provide a suitable safety factor. However, for a well-designed road the effects of its moisture regime or climate on the strength of the road are primarily manifest in the strength of the subgrade and so a value of 1.0 should be used for the pavement layers for relatively ‘normal’ conditions and a value of 1.1 for very dry conditions.

Design procedure

To design the upgrading or rehabilitation of a road, it is first necessary to measure the structural number at each test point as indicated above. The calculation of SN for design purposes is the AASHTO method which is based on the value of the soaked CBR of the layers. To convert from the in situ values to the soaked values requires a measurement of the in situ moisture condition, expressed as the ratio of in situ moisture content divided by the optimum moisture content, and the use of Figure 13-12. The in situ moisture condition is obtained from the samples collected for laboratory analysis during the DCP survey. A minimum of three samples per kilometre is recommended. It is often more useful to obtain the samples once the DCP survey has been analysed and the most appropriate sampling points can be identified to ensure that maximum benefit is obtained from the sampling and testing. However, the delay between the in situ testing and sampling must be minimal (less than 14 days).

The relationship between soaked and in situ strength (CBR) depends on the characteristics of the materials. However, for the level of accuracy required, Figure 13-12, which is based on extensive research, is adequate. It should be noted that the comments regarding moisture sensitivity of some materials explained for Table 13-4 are equally applicable to Figure 13-12.

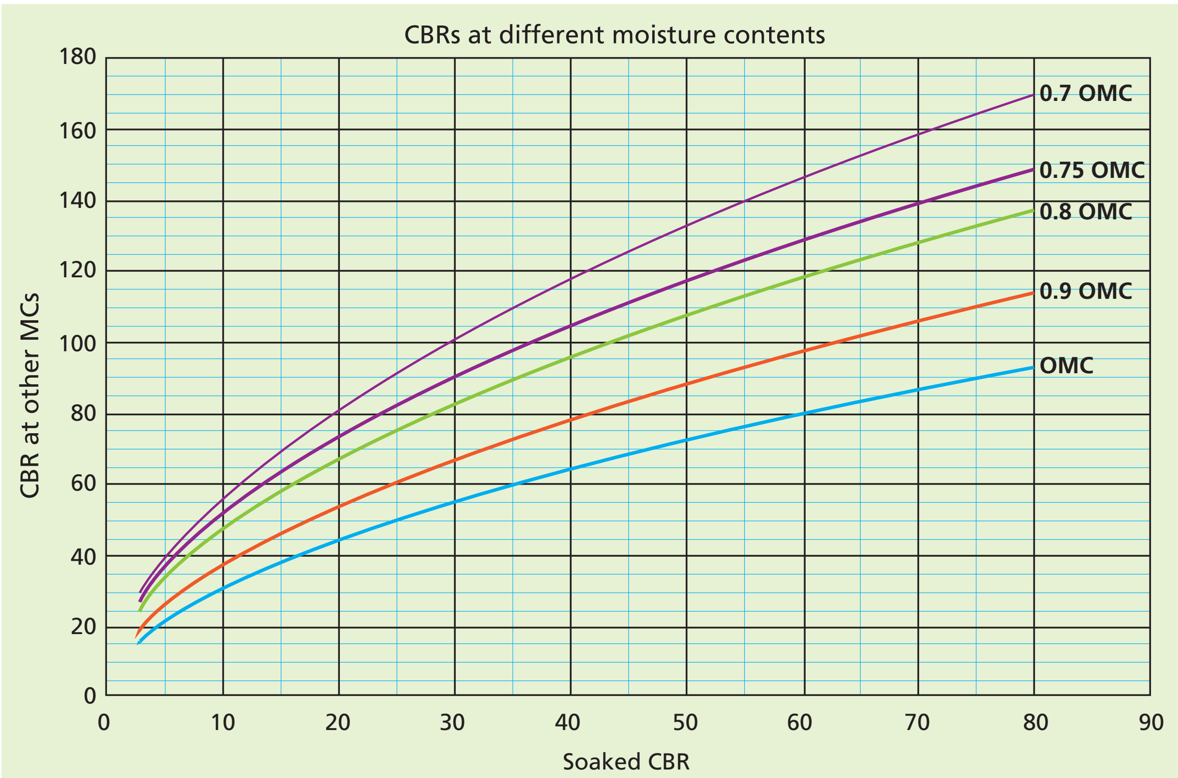


Figure 13-12: CBRs at different moisture contents

Table 13-7: Pavement layer strength coefficients

Layer Type	Condition	Coefficient
Surface treatment		ai = 0.2
Granular unbound roadbase	Default	ai = (29.14 CBR - 0.1977 CBR^2 + 0.00045 CBR^3) 10^{-4}
	CBR > 100%	a_i = 0.145
	CBR = 100%	a_i = 0.14
	CBR = 80% With a stabilised layer underneath With an unbound granular layer underneath	 a_i = 0.135 a_i = 0.13
	CBR = 65%	a_i = 0.12
	CBR = 55%	a_i = 0.107
	CBR = 45%	a_i = 0.1
Bitumen treated gravels and sands	Marshall stability = 2.5 MN	a_i = 0.135
	Marshall stability = 5.0 MN	a_i = 0.185
	Marshall stability = 7.5 MN	a_i = 0.23
Cemented	Equation	a_i = 0.075 + 0.039 (UCS) – 0.00088$(UCS)^2$
	CB 1 (UCS = 3.0 – 6.0 MPa)	a_i = 0.185
	CB 2 (UCS = 1.5 – 3.0 MPa)	a_i = 0.23
Granular unbound subbases	Equation	aj = -0.075 + 0.184 ($\log_{10}$ CBR) – 0.0444 $(\log_{10} CBR)^2$
	CBR = 40%	a_i = 0.11
	CBR = 30%	a_i = 0.1
	CBR = 20%	a_i = 0.09
	CBR = 15%	a_i = 0.08
	CBR = 10%	a_i = 0.065
Cemented	(UCS = 0.7 – 1.5 MPa)	a_i = 0.1

Note: Unconfined Compressive Strength (UCS) is stated in MPa at 14 days.

Modified Structural Number

The effect of different subgrades can also be included in the structural number approach. The subgrade contribution is defined as follows:

$$SNC = SN + 3.51 (\log10\ CBRs) – 0.85 (\log10\ CBRs)^2 – 1.43$$

Where:

SNC = Modified structural number of the pavement

CBRs = In-situ CBR of the subgrade

The modified structural number (SNC) has been used extensively over the past 20 or 30 years and forms the basis for defining pavement strength in many pavement performance models. It should be used to identify the overall strength of each DCP test point in the old road if the subgrade is particularly variable.

Target Structural Numbers: When designing upgrading or rehabilitation it is necessary to determine the existing effective SN as described above at each test point and the required SN to carry the new design traffic. Table 13-8 to Table 13-11 show the target values of SN and SNC for different subgrade conditions and for different traffic levels calculated from the design charts for roads with a thin bituminous surfacing. The difference between the required structural number and the existing structural number is the deficiency that needs to be corrected.

The final step is to determine uniform sections based on the strengthening requirements using a CUSUM method. For each uniform section the following percentiles of the strengthening requirements should be used:

1. Median for TLC 0.01 and TLC 0.1
2. Upper 75 percentile for TLC 0.3
3. Upper 90 percentile for TLC 0.5 and TLC 1.0

However, when the strengthening requirements are large it may be more cost effective to carry out some reconstruction and, conversely if they are small, maintenance may be all that is required. Table 13-12 is a guide to the treatments.

Table 13-8: Structural Numbers (SN) for Bituminous Pavement Design Chart 1 (Table 13-4: Wet areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.05	1.7	1.95	2.05	2.30
S3 (5-7%)	0.95	1.45	1.70	1.85	2.1
S4 (8-14%)	0.8	1.25	1.55	1.65	1.85
S5 (15-29%)	0.7	1.0	1.25	1.35	1.5
S6 (>30%)	0.6	0.7	0.85	0.95	1.0

Note: These values exclude a contribution from the surfacing.

Table 13-9: Structural Numbers (SN) for Bituminous Pavement Design Chart 2 (Table 13-5: Moderate & Dry areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.05	1.55	1.80	2.0	2.15
S3 (5-7%)	0.9	1.35	1.55	1.70	1.95
S4 (8-14%)	0.7	1.05	1.35	1.45	1.6
S5 (15-29%)	0.6	0.85	1.05	1.1	1.3

Note: These values exclude a contribution from the surfacing.

Table 13-10: Required Modified Structural Numbers (SNC) for Chart 1 (Table 13-4: Wet areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2 (3-4%)	1.1	1.75	2.0	2.1	2.35
S3 (5-7%)	1.55	2.05	2.35	2.45	2.7
S4 (8-14%)	1.85	2.25	2.6	2.7	2.9
S5 (15-29%)	2.2	2.55	2.75	2.9	3.05
S6 (>30%)	2.5	2.6	2.75	2.85	2.9

Table 13-11: Required Modified Structural Numbers (SNC) for Chart 2 (Table 13-5: Moderate & Dry areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2 (3-4%)	1.1	1.6	1.85	2.05	2.2
S3 (5-7%)	1.5	1.95	2.15	2.35	2.55
S4 (8-14%)	1.75	2.1	2.4	2.5	2.65
S5 (15-29%)	2.1	2.35	2.55	2.65	2.8
S6 (>30%)	2.5	2.6	2.65	2.75	2.8

Table 13-12: Structural Deficiency Criteria

Structural deficiency based on appropriate percentiles	Action	Notes
0.2 or negative	Maintain with a surface treatment (e.g. a surface dressing).	A thin granular overlay can be used to correct other road defects.
0.2 – 1.2	New granular layer. The existing layers must be checked for quality (subbase or roadbase). The minimum thickness of new roadbase should be 50 mm.	Some localised remedial works can be expected. A surface treatment is required.
1.2 – 1.8	The existing roadbase is likely to be only of subbase quality and should be checked. Additional subbase and a new roadbase are required.	Some localised remedial works will be needed. A surface treatment is required.
> 1.8	The existing layers are likely to be less than subbase quality, hence a new subbase and roadbase are required. Chemically stabilising existing material should be considered.	Localised remedial treatment and a surface treatment are required.

13.4 DESIGN OF ROADS WITH NON DISCRETE SURFACES

13.4.1 General

Structural surfaces may have a place for use on LVRs. Initial cost is usually a constraining factor but the whole life costs may sometimes make these options favourable. The most common use is for semi-urban areas where marketing and trading takes place and where vehicle movements are unpredictable and on sections that are very steep or otherwise difficult from an engineering point of view.

13.4.2 Un-reinforced Concrete (URC)

The un-reinforced cement concrete option for LVRs involves casting slabs 4.0 to 5.0 m in length between formwork with load transfer dowels between them. The thickness of the concrete depends on the traffic and subgrade support as shown in Table 13-13. In some cases, where continuity of traffic demands it, these slabs may be half carriageway width.

Table 13-13: Thickness (mm) – Un-reinforced concrete pavement (URC)

Subgrade Class (CBR	TLC 01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	160 URC	170 URC	175 URC	180 URC	190 URC
	150 G30	150 G30	150 G30	150 G30	150 G30
S3 (5-7%)	150 URC	160 URC	165 URC	170 URC	180 URC
	125 G30	125 G30	125 G30	125 G30	125 G30
S4 (8-14%)	150 URC	150 URC	160 URC	170 URC	180 URC
	100 G30	100 G30	100 G30	100 G30	100 G30
S5 (15-29%)	150 URC	150 URC	160 URC	170 URC	180 URC
	100 G30	100 G30	100 G30	100 G30	100 G30
S6 (>30%)	150 URC	150 URC	160 URC	170 URC	180 URC

Notes:
1. Cube strength = 30 MPa at 28 days.
2. On subgrades > 30%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >30% is in agreement with the recommendations.

13.4.3 Concrete Strips

Concrete strips are currently not commonly used in Tanzania but they are a viable solution where traffic volumes are very low (< about 30 vpd). The pavement thickness under discrete elements given in Table 13-15 is used for the design. It is important to ensure adequate support under the strips to prevent cracking and movement under load especially in conditions of high moisture.

The strips must be constructed of B20 class concrete. If heavy trucks are expected, mesh wire reinforcement shall be used and placed at 1/3 depth from the surface. The concrete strips shall be 0.5 m wide, 1.5 to 3.0 m (max) in length and 0.2 m in thickness. The distance from centre to centre shall be 1.0 m.

13.5 DESIGN OF ROADS WITH DISCRETE ELEMENT SURFACINGS

13.5.1 General

Discrete element surfaces for LVRs do not usually provide much structural strength in terms of load spreading because the interlock between the elements is poor. However such surfacings are very useful for areas of marketing and trading and some have the advantage that they can be uplifted and replaced if damage to the surfaces themselves occurs or if there is a need to repair the underlying layers because of soil movement and deformation.

13.5.2 Hand-packed Stone (HPS)

HPS paving consists of a layer of large broken stone pieces (typically 150 mm to 300 mm thick) tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel. A degree of interlock is achieved and has been assumed in the designs shown in Table 13-14. The structures also require a capping layer when the subgrade is weak and a conventional subbase of G30 material or stronger is required. A capping layer also provides a smooth stable platform to work on.

The HPS is normally bedded on a thin layer of sand (SBL). An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.

Table 13-14: Thicknesses designs for Hand Packed Stone (HPS) pavement (mm)

Subgrade Class (CBR)	TLC 01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	175 G30	125 G30	150 G30	150 G30	
		150 G15	200 G15	200 G15	
S3 (5-7%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	125 G30	200 G30	150 G30	150 G30	
			150 G15	150 G15	
S4 (8-14%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	100 G30	150 G30	200 G30	200 G30	
S5 (15-29%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	Note	*Note*	*Note*	*Note*	
S6 (>30%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	Note	*Note*	*Note*	*Note*	

Notes:
1. The capping layer of G15 material and the subbase layer of G30 material can be reduced in thickness if stronger material is available.
2. The capping layer can be G10 provided it is laid 7% thicker.
3. The subbase layers can be material stronger than G30 and laid to reduced thickness.
4. On subgrades > 15%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >15%.

13.5.3 Pave or Stone Setts
Stone sett surfacing or Pavé consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. The individual stones should have at least one face that is fairly smooth to be the upper or surface face when placed. Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregate is brushed into the spaces between the stones and the layer is then compacted with a roller. Suitable structural designs are shown in Table 13-15.

13.5.4 Clay Bricks
Fired Clay Bricks are the product of firing moulded blocks of silty clay. The road surfacing consists of a layer of edge-on engineering quality bricks within mortar bedded and jointed edge restraints, or kerbs, on each side of the pavement. The thickness designs are as shown in Table 13-15 for TLC 0.01 and TLC 0.1. Fired clay brick surfacings are not suitable for traffic classes above TLC 0.1.

13.5.5 Cobble Stones or Dressed Stone Pavements
Cobble or Dressed Stone surfacings are similar to Pave and consist of a layer of roughly rectangular dressed stones laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. The individual stones should have at least one face that is fairly smooth to be the upper or surface face when placed. Each stone is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregates is brushed into the spaces between the stones and the layer then compacted with a roller. Cobble stones are generally 150 mm thick and dressed stones

generally 150-200 mm thick. These options are suited for homogeneous rock types that have inherent orthogonal stress patterns (such as granite) that allow for easy break of the fresh rock into the required shapes by labour-based means.

The thickness designs are given in Table 13-15 except that the thickness of the cobblestone is generally 150 mm.

Table 13-15: Thicknesses designs for various discrete element surfacings (DES) (mm)

Subgrade Class (CBR	TLC 01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	100 G65	125 G65	150 G80	150 G80	150 G80
	100 G30	150 G30	150 G30	175 G30	200 G30
	100 G15	150 G15	175 G15	200 G15	200 G15
S3 (5-7%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	150 G65	125 G80	150 G80	150 G80
	100 G30	175 G30	125 G30	150 G30	175 G30
			150 G15	150 G15	175 G15
S4 (8-14%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	150 G65	150 G65	150 G80	150 G80	175 G80
		100 G30	150 G30	200 G30	225 G30
S5 (15-29%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	100 G65	125 G80	150 G80	150 G80
		125 G30	125 G30	125 G30	150 G30
S6 (>30%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	150 G65	150 G80	150 G80	150 G80
		Note	*Note*	*Note*	*Note*

Notes:

1. The capping layer of G15 material and the subbase layer of G30 material can be reduced in thickness if stronger material is available.
2. The capping layer can be G10 provided it is laid 7% thicker.

13.5.6 Mortared options

In some circumstances (e.g. on slopes in high rainfall areas and subgrades susceptible to volumetric change) it may be advantageous to use mortared options for the discrete element surfacings. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone (or Dressed Stone), and Fired Clay Brick pavements. The construction procedure is largely the same as for the un-mortared options except that cement mortar is used instead of sand for bedding and joint filling. The behaviour of mortared pavements is different to that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on mortared options, although empirical evidence indicates that inter-block cracking may occur. For this reason the option is currently only recommended for the lightest traffic divisions up to TLC 0.1. Hence, refer to Table 13-15 until further locally relevant evidence is available.

13.5.7 Concrete Blocks

Concrete blocks are usually constructed on a cement-stabilised base. A 25-50 mm sand blanket should be placed on top of the cement-stabilised base to provide a cushion and a drainage layer. The blocks shall be made of B20 concrete. Interlocking blocks are recommended.

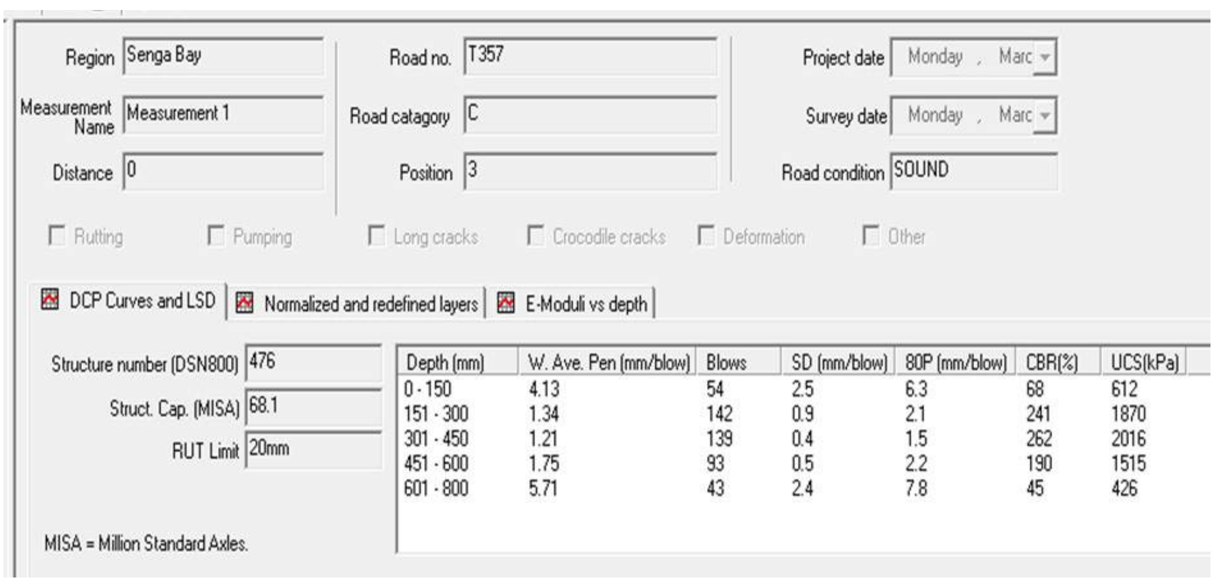
13.6 DESIGN EXAMPLE DCP-DN METHOD

13.6.1 Design Problem

1. A new paved road is to be built on the alignment of an existing gravel road to carry a cumulative design traffic of 0.3 MESA.
2. A DCP survey was carried out in the intermediate season (i.e. expected in-service conditions) and the data were analysed using WinDCP5.1 (the predecessor of AFCAP DCP).
3. In all, 87 DCP tests were carried out, one every 100 m over the total length of the road of 8.6 km.

The following design procedure was followed:

Step 1: Each DCP test was analysed using WinDCP5.1 “single measurement analysis”. From the outputs (Figure 13-13), the DSN800, and weighted average DN values for the upper three 150 mm layers of each DCP test were determined.



Region: Senga Bay
Measurement Name: Measurement 1
Distance: 0
Road no.: T357
Road catagory: C
Position: 3
Project date: Monday , Marc
Survey date: Monday , Marc
Road condition: SOUND

Rutting
Pumping
Long cracks
Crocodile cracks
Deformation
Other

DCP Curves and LSD | Normalized and redefined layers | E-Moduli vs depth

Structure number (DSN800): 476
Struct. Cap. (MISA): 68.1
RUT Limit: 20mm

Depth (mm)	W. Ave. Pen (mm/blow)	Blows	SD (mm/blow)	80P (mm/blow)	CBR(%)	UCS(kPa)
0 - 150	4.13	54	2.5	6.3	68	612
151 - 300	1.34	142	0.9	2.1	241	1870
301 - 450	1.21	139	0.4	1.5	262	2016
451 - 600	1.75	93	0.5	2.2	190	1515
601 - 800	5.71	43	2.4	7.8	45	426

MISA = Million Standard Axles.

Figure 13-13: Typical output of WinDCP5.1

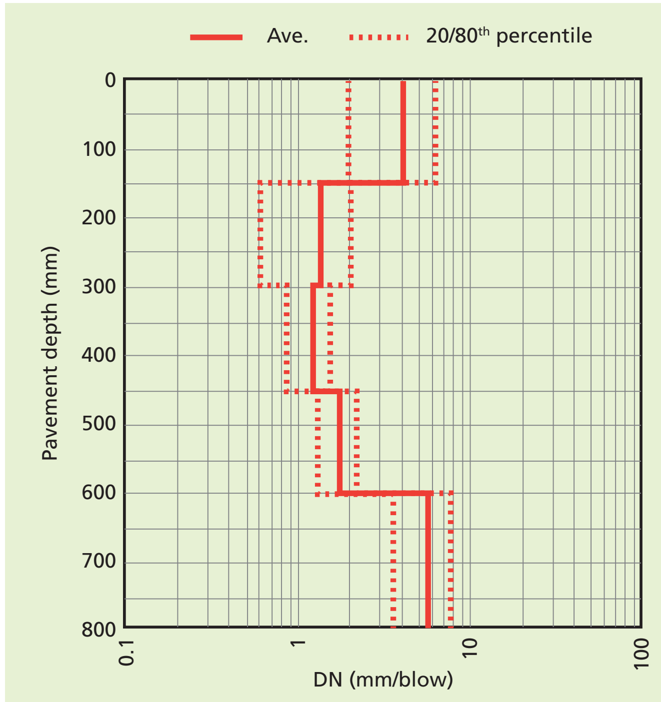


Figure 13-14: WinDCP5.1 plot of penetration with depth for Test 1 (0.1 km)

Step 2: These results were then used to identify uniform sections using a cumulative sum technique. Prior to this all obvious outliers based on DSN800 (very high or very low) were removed from the dataset (14 readings out of 87). The majority of these were particularly high, probably the result of stones within the layer. It is important, however, to check on site the actual reasons for the very high or very low readings as far as possible. Removal of the outliers only results in a smoothing of the curves and does not affect the actual "change points".

Figure 13-15 shows a part of the spreadsheet used to calculate the "cumulative sums" and Figures 13-16 and 13-17 plots of the CUSUM curves for the different parameters. The CUSUM for the DSN800 values is calculated by obtaining the average of all of the DSN800 values (Column D in Figure 13-15) and then subtracting this from each of the DSN800 values (column E). The results are then added together (Column F). These values are then plotted against distance.

It is clear from the plots of DSN800 and DN301-450 that there are significant changes in the support at about km 2.0 and km 7.0. The change at km 7.0 is also reflected in the DN150 and DN151-300 plots. It is thus possible to derive 3 distinct uniform sections from these plots – 0 – 2.0 km, 2.0 – 7.0 km and 7.0 -9.0 km.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	CUSUM Analysis E1641														
2	Test			DSN800			0-150 mm			151-300 mm			301-450 mm		
3	no	Chainage	Position	DSN	DSN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum
4	2	0,100	RHS	198	5,59	5,59	4,09	-1,77	-1,77	2,49	-2,40	-2,40	5,76	0,15	0,15
5	4	0,300	RHS	169	-23,41	-17,82	3,84	-2,02	-3,79	3,44	-1,45	-3,84	4,77	-0,84	-0,70
6	6	0,500	RHS	134	-58,41	-76,23	4,22	-1,64	-5,43	6,80	1,91	-1,93	7,87	2,26	1,56
7	8	0,700	RHS	206	13,59	-62,63	2,49	-3,37	-8,80	3,44	-1,45	-3,37	5,77	0,16	1,72
8	9	0,800	LHS	207	14,59	-48,04	3,45	-2,41	-11,21	2,11	-2,78	-6,15	4,90	-0,71	1,00
9	10	0,900	RHS	164	-28,41	-76,45	3,41	-2,45	-13,66	4,39	-0,50	-6,64	5,41	-0,20	0,80
10	11	1,000	LHS	146	-46,41	-122,86	3,92	-1,94	-15,60	5,14	0,25	-6,39	7,65	2,04	2,83
11	13	1,200	LHS	188	-4,41	-127,27	3,80	-2,06	-17,66	3,30	-1,59	-7,97	4,39	-1,22	1,61
12	15	1,400	LHS	194	1,59	-125,68	3,90	-1,96	-19,62	2,94	-1,95	-9,92	4,49	-1,12	0,49
13	16	1,500	RHS	230	37,59	-88,08	5,51	-0,35	-19,97	2,11	-2,78	-12,69	4,14	-1,47	-0,99
14	17	1,600	LHS	163	-29,41	-117,49	5,11	-0,75	-20,72	3,47	-1,42	-14,11	3,80	-1,81	-2,80
15	18	1,700	RHS	210	17,59	-99,90	3,09	-2,77	-23,49	3,51	-1,38	-15,48	4,08	-1,53	-4,33
16	19	1,800	LHS	113	-79,41	-179,31	5,72	-0,14	-23,63	4,26	-0,63	-16,11	11,80	6,19	1,85
17	20	1,900	RHS	191	-1,41	-180,72	9,01	3,15	-20,48	4,87	-0,02	-16,12	5,69	0,08	1,93
18	21	2,000	LHS	169	-23,41	-204,13	3,59	-2,27	-22,75	6,24	1,35	-14,77	21,20	15,59	17,52
19	22	2,100	RHS	217	24,59	-179,54	2,66	-3,20	-25,95	4,11	-0,78	-15,54	5,68	0,07	17,58
20	23	2,200	LHS	272	79,59	-99,94	2,57	-3,29	-29,24	2,94	-1,95	-17,49	3,83	-1,78	15,80
21	24	2,300	RHS	271	78,59	-21,35	4,20	-1,66	-30,89	2,47	-2,42	-19,90	3,15	-2,46	13,33
22	25	2,400	LHS	258	65,59	44,24	2,90	-2,96	-33,85	3,15	-1,74	-21,64	3,40	-2,21	11,12
23	28	2,700	RHS	253	60,59	104,83	5,68	-0,18	-34,03	4,93	0,04	-21,59	2,54	-3,07	8,05

Figure 13-15: Part of the spreadsheet showing the CUSUM calculation

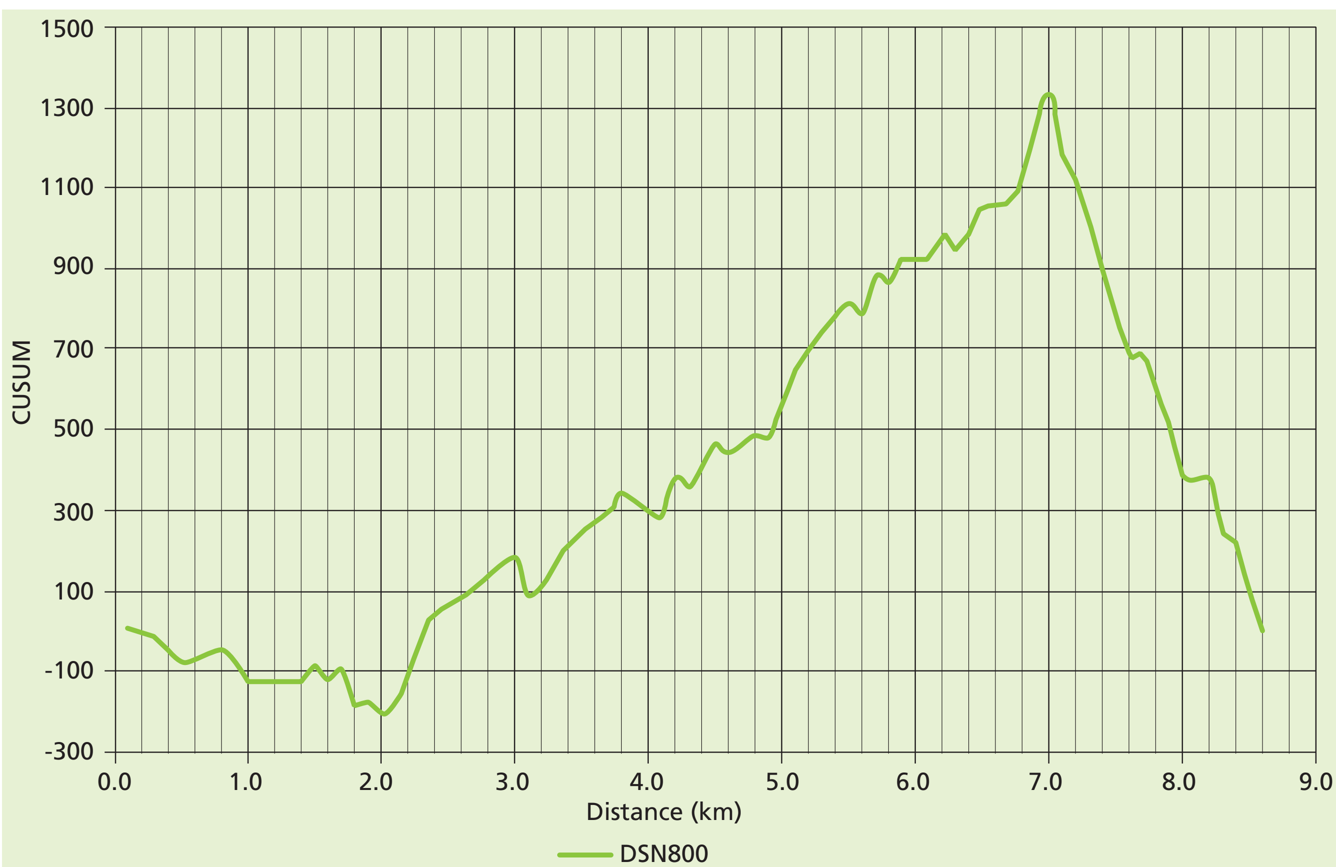


Figure 13-16: Plot of the CUSUM versus distance for the DSN_{800} results

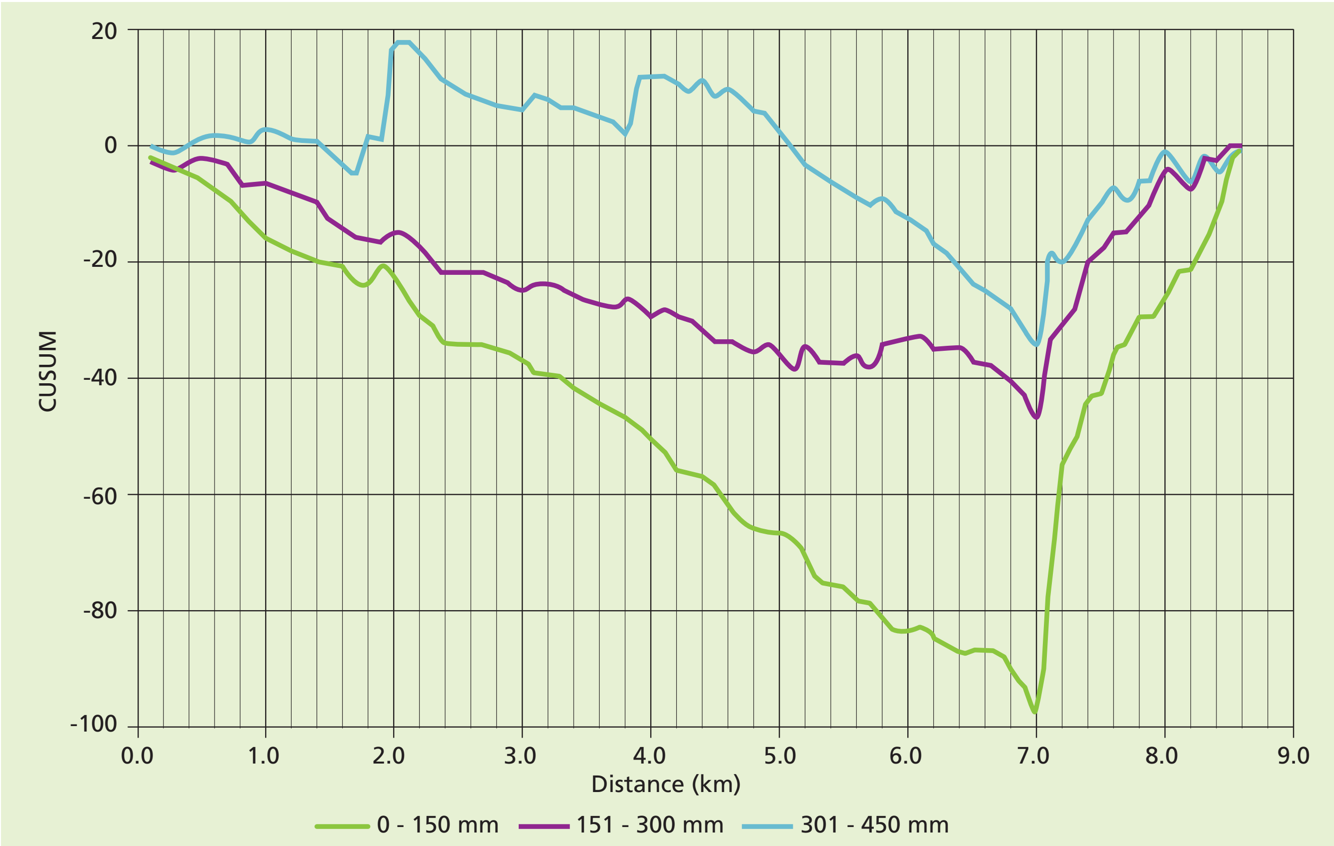
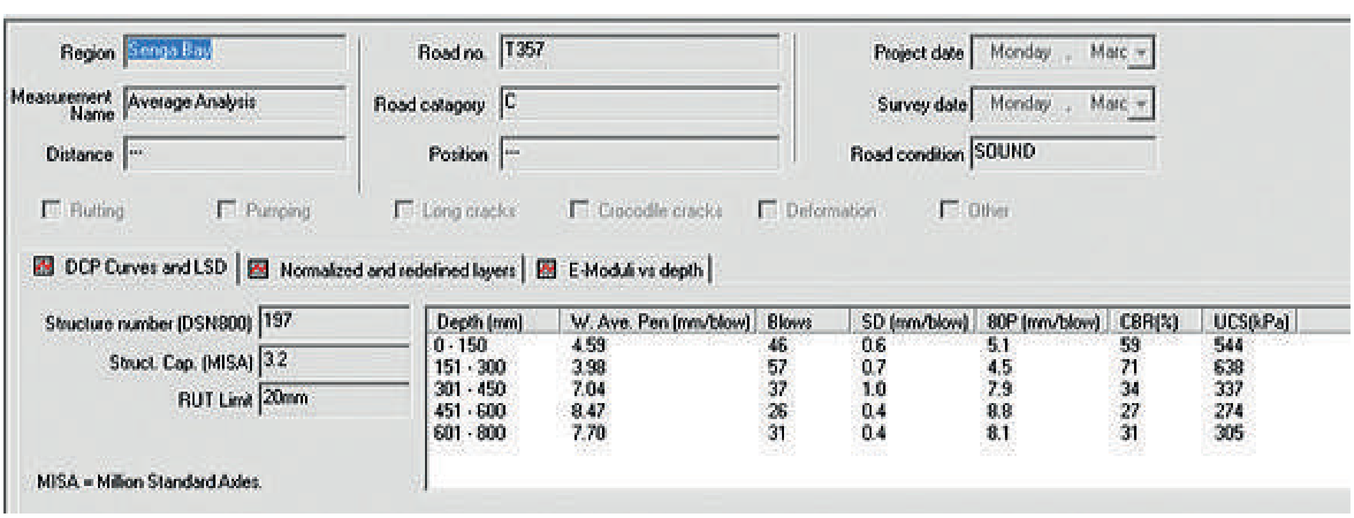


Figure 13-17: Plot of the CUSUM versus distance for the DN150, DN151-300 and DN301-450 results

Step 3: The data for each of these uniform sections are then analysed individually. The outliers can be retained or removed and generally have little impact on the final result. It can be seen that by retaining the outliers, the average DN150 is 4.59 compared with a value of 4.34 obtained when they are excluded, as shown in Figures 13-18 and 13-19 respectively.

The data can be analysed using either the average point analysis function in WinDCP5.1 or using the spreadsheet developed for the CUSUM analysis. The layer strength diagram output of the WinDCP5.1 analysis is shown in Figure 13-18. It can easily be seen that the average (note that this is not the 50th percentile) and the range between the 20th and 80th percentiles is very small.


Region: Senga Bay
Measurement Name: Average Analysis
Distance: ---
Road no.: T357
Road category: C
Position: ---
Project date: Monday , Marc
Survey date: Monday , Marc
Road condition: SOUND

Rutting · Pumping · Long cracks · Crocodile cracks · Deformation · Other

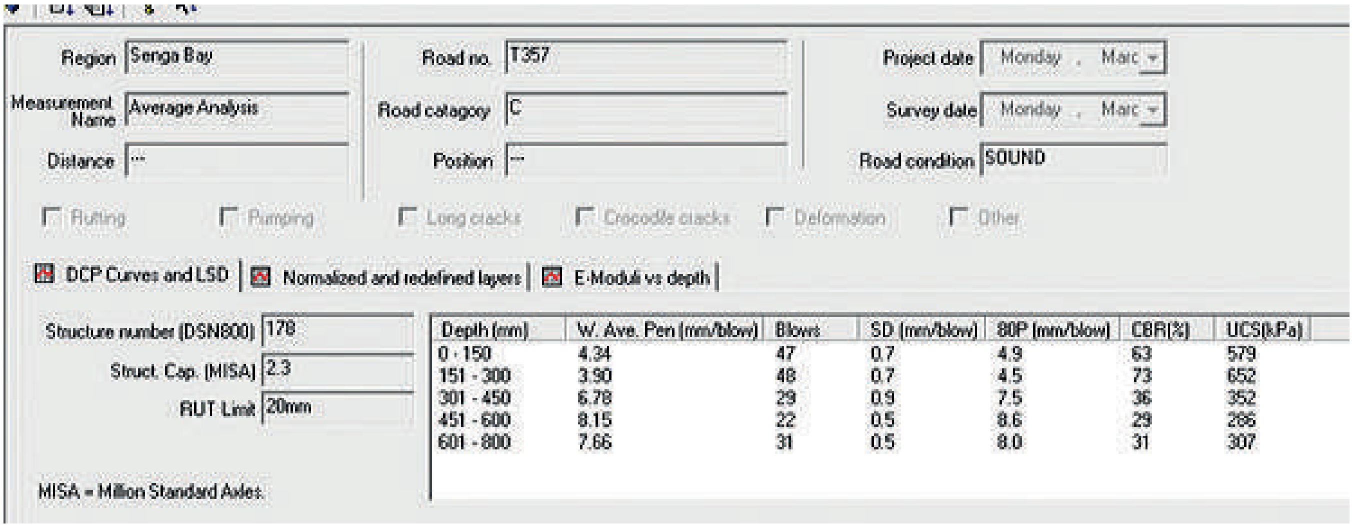
DCP Curves and LSD | Normalized and redefined layers | E-Moduli vs depth

Structure number (DSN800): 197
Struct. Cap. (MISA): 3.2
RUT Limit: 20mm

Depth (mm)	W. Ave. Pen (mm/blow)	Blows	SD (mm/blow)	80P (mm/blow)	CBR(%)	UCS(kPa)
0 - 150	4.59	46	0.6	5.1	59	544
151 - 300	3.98	57	0.7	4.5	71	638
301 - 450	7.04	37	1.0	7.9	34	337
451 - 600	8.47	26	0.4	8.8	27	274
601 - 800	7.70	31	0.4	8.1	31	305

MISA = Million Standard Axles

Figure 13-18: Output of "average points analysis" (WinDCP5.1) for uniform section 1 including all points



Depth (mm)	W. Ave. Pen (mm/blow)	Blows	SD (mm/blow)	80P (mm/blow)	CBR(%)	UCS(kPa)
0 - 150	4.34	47	0.7	4.9	63	579
151 - 300	3.90	48	0.7	4.5	73	652
301 - 450	6.78	29	0.9	7.5	36	352
451 - 600	8.15	22	0.5	8.6	29	286
601 - 800	7.66	31	0.5	8.0	31	307

Figure 13-19: Output of "average points analysis" (WinDCP5.1) for uniform section 1 excluding "outliers"

The percentiles can be calculated equally easily on the initial spreadsheet using the Excel functionalities.

Step 4: The process is repeated for each of the uniform sections. The results for each uniform section are summarised in Table 13-16.

It is clear from the results that the upper layer in all cases is inadequate. A single design solution for Uniform sections 1 and 2 requires that the upper layer be improved. The material should be investigated as it is only marginally inferior to see if processing such as blending, better compaction or stone removal could improve its quality to the required specification. If not the layer should be overlaid with a new 150 mm layer of selected material.

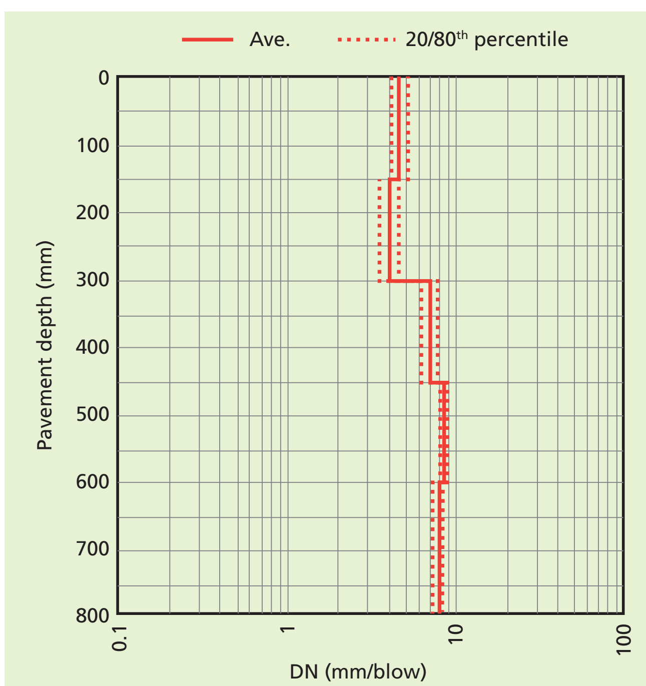


Figure 13-20: WinDCP5.1 plot of average analysis for uniform section 1

Table 13-16: Summarised DN values for each layer and uniform section

Design class LE 0.3	Spec. DN mm/bl	Uniform Section 1 Uniform Section 3 50th %-ile	Uniform Section 2 km 2+000 - 7+000 50th %-ile	Uniform Section 3 km 7+000 - 8+600 50th %-ile
0-150 mm	3.2	4.01	3.68	10.07
151-300 mm	6	3.46	3.66	7.68
301-450 mm	12	5.16	3.83	7.81
451-600 mm	19	6.88	3.89	8.96
601-800 mm	25	7.06	4.57	11.04
DSN800	100	198.00	237.00	109.00

Uniform section 3 on the other hand is particularly poor. Neither of the two upper 150 mm layers are adequate. The addition of a single 150 mm layer would not prove adequate and in this case, the upper 150 mm layer needs to be removed and discarded. The underlying layer (150 – 300 mm) should be assessed to see if it could be improved by blending or some other treatment. If not, this section of the road requires the addition of 300 mm of material after removal of the upper 150 mm.

This is clearly illustrated for Uniform section 1 by Figure 13-21, showing a comparison of the in situ layer strengths with the layer strengths required for the selected traffic category. The areas shaded green are adequate and the yellow area is deficient. It is interesting to note that although the 50th percentile was used in this example, both the 80th and 20th percentiles for all but the upper layer would have proved adequate. In the upper 150 mm layer, none of the percentiles would allow the material to be used in its current condition.

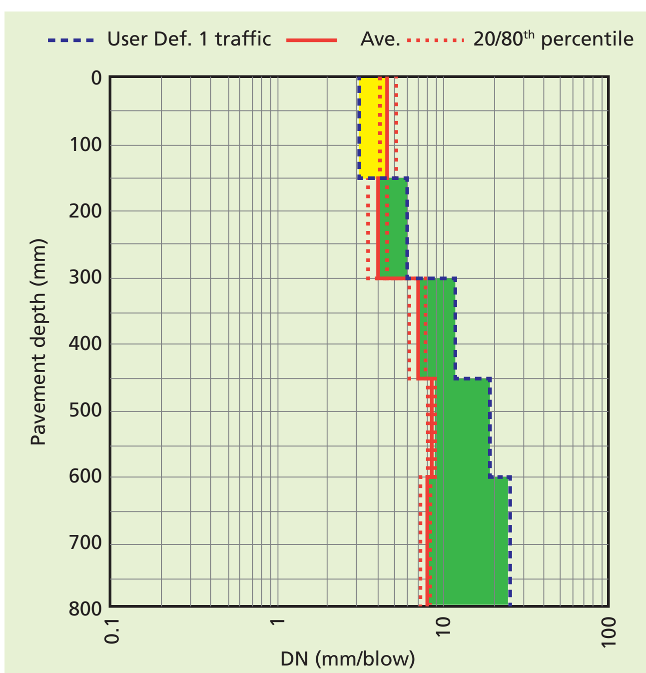


Figure 13-21: Layer strength diagram showing material strengths and traffic requirements

13.7 DESIGN EXAMPLE DCP-CBR METHOD

13.7.1 Design Problem

4. An old road needs to be upgraded to carry a cumulative design traffic loading of 0.3 MESA.
5. A DCP survey is carried out in the intermediate season. The data are analysed using the TRL DCP program.
6. Samples of each layer of the pavement are taken for laboratory testing.
7. The climatic zone is dry hence design Chart 2 in Table 13-5 and Table 13-8 are to be used.

13.7.2 Basic Analysis Procedure

The following analysis is carried out for each DCP test point.

Step 1 Initial analysis of each DCP test

A typical DCP result is shown in Figure 13-22. The program identifies the layer boundaries automatically and outputs, for each layer, the thickness, average mm/blow, and CBR. Usually more than one subbase layer is identified.

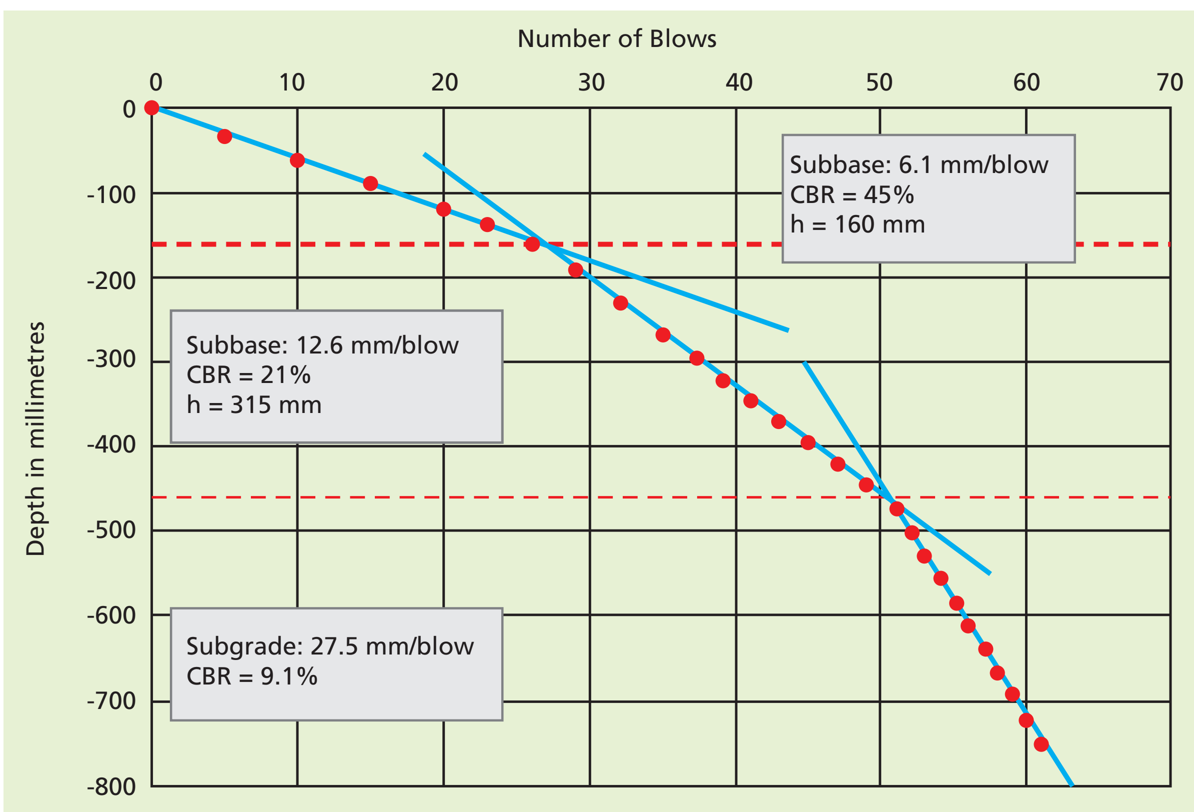


Figure 13-22: Typical DCP test result

Step 2 Defining the pavement layers and computing pavement strength

The user must define each layer as roadbase, subbase, or subgrade. The program then calculates the contribution of each layer to the overall structural number. The strength coefficients are calculated automatically (Table 13-17).

Table 13-17: Example of CBRs, strength coefficients and SNs at a DCP test chainage

Layer No		CBR %	Thickness (mm)	Depth (mm)	Position	Strength Coefficient	SN
1		45	160	160	Roadbase	0.10	0.63
2		21	315	475	Subbase	0.09	1.12
		9.1	-		Subgrade		
Total			475				1.75

Step 3 Adjustment for moisture conditions

These SNs are the values obtained at the in situ conditions. For evaluation and design purposes the SN of each layer in the soaked condition is required. The user must estimate the soaked CBR values from the in situ moisture contents measured in laboratory tests of the samples. This is done using Figure 13-7. This conversion cannot be exact because the relationships shown in the Figure depend on various material properties such as PI, hence a high level of precision is not possible, and nor is it necessary. In this example the in situ conditions are not extreme (in terms of wet or dry). An average in situ moisture content of OMC was obtained and used with Figure 13-12 to convert the CBRs to soaked conditions.

The strength of the subgrade must also be adjusted to give an estimate of the soaked value. However, it is only necessary to identify the subgrade class. For low values of CBR, if the in situ moisture regime is OMC the soaked value is typically half to one third of the in situ value. If the moisture regime is dry (0.75*OMC) then the soaked value is one third to one quarter of the in situ value.

The revised CBRs are shown in Table 13-18. Using the revised CBR values, the revised SN is calculated for each layer and then summed to give the total value as shown in Table 13-18.

Table 13-18: Example of revised CBRs and SNs corrected for moisture at a DCP test chainage

Layer No	CBR (%)	Thickness (mm)	Position	Revised CBR (%)	Revised strength coefficient	Revised SN
1	45	160	Roadbase	20	0.09(1)	0.57
2	21	315	Subbase	5	0.03	0.37
	9.1	-	Subgrade			
Total		475				0.94

Step 4 Estimation of strengthening requirements

Having determined the subgrade strength and the existing SN for each of the DCP test points, design Chart 1 (Table 13-8 or 13-10) is used to determine the SN or SNC required for the new road to carry the design traffic on the design subgrade. The difference between the required SN and the existing SN (ΔSN) is the key parameter on which the upgrading design is based. The calculations are carried out for every test point remembering that the subgrade may not be the same for all the DCP test points.

The choice of using SN or SNC depends on the variability of the subgrade. SNC should be used if the subgrade is variable. Calculations can be done using both SN and SNC and the more conservative results used in the design.

Step 5 Defining the upgrading treatments

The results of the DCP testing and preliminary analysis using the DCP program is a graph showing the CUSUM calculations of structural deficiencies as a function of chainage along the road as shown in Figure 13-23. (Using the UK DCP 3.1 program, similar CUSUM calculations can be automatically generated for any of the parameters derived from the DCP tests if desired).

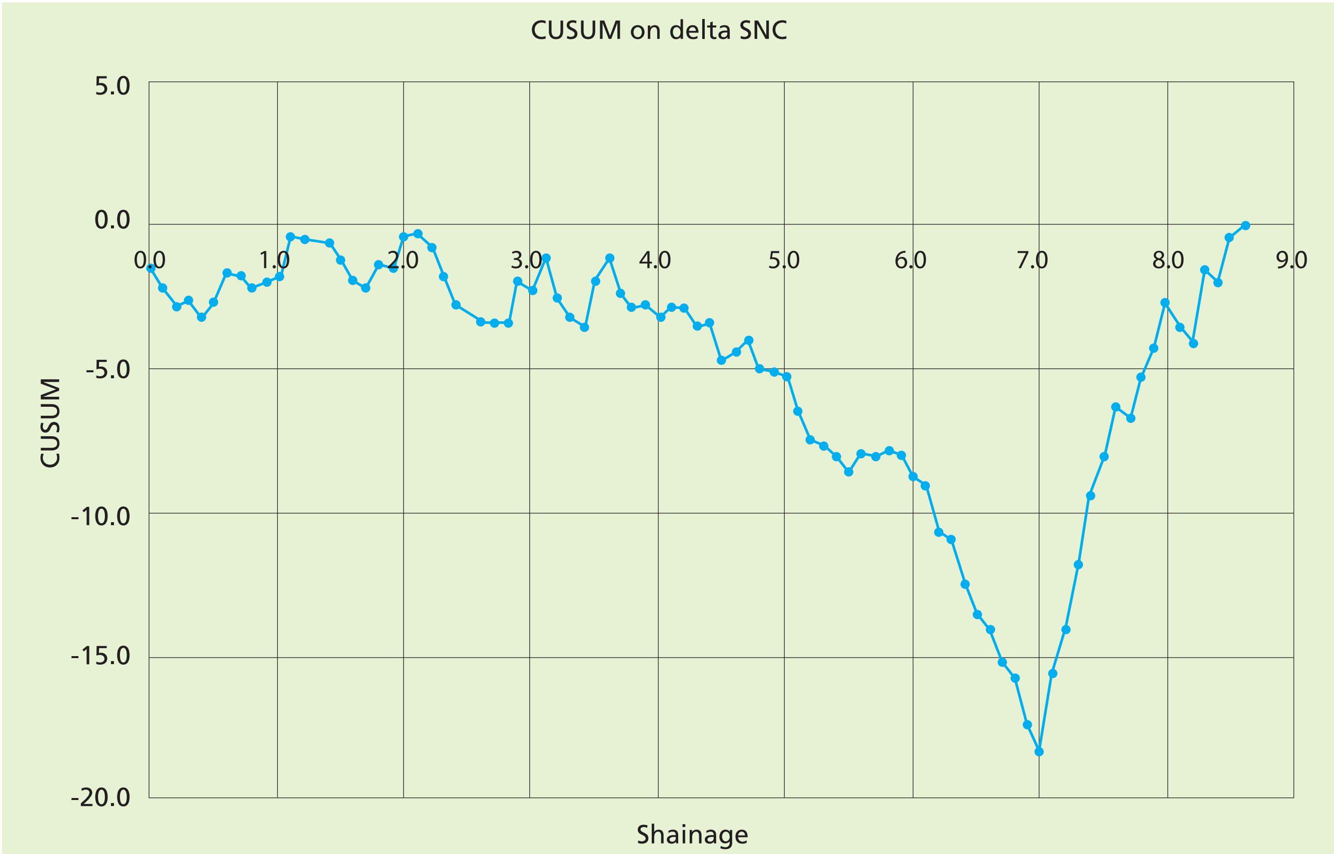


Figure 13-23: CUSUM analysis to identify uniform sections

Changes in the slope of the trend line identify relatively homogenous or uniform sections. There are 3 sections that are distinct namely:

- Section 1 Chainage 0.000 to 3.600 where the road is generally thick but weak and not very uniform (little variation in fluctuating CUSUM).
- Section 2 Chainage 3.700 to 7.000 where the road is stronger and more uniform (steeply decreasing CUSUM).
- Section 3 Chainage 7.100 to 8.600 where the road is much weaker (increasing CUSUM).

Section 1

In general the main problem in Section 1 is that out of 35 test points there are 21 with distinct roadbase problems, identified from a visual inspection of the individual DCP test results and/or the tables summarising the properties of each individual test that are computed automatically in the program. The problems are of one or more of three types.

1. Loose upper layer.
2. Weak middle layer.
3. Overall weakness in the road base.

A total of 14 of the test points require no treatment at all except surface smoothing and compacting but these points are distributed fairly evenly along the section with never more than two being adjacent to each other. It is not feasible to change the treatment at short intervals, even with labour based construction, because it is probably impossible to identify the boundaries. The variations along the road are frequent and probably largely random hence the sections of similar characteristics may be very short.

The treatment for Type 1 is simply to test the top (roadbase) layer to see if it can be made strong enough by some sort of processing, for example, merely compaction or possibly blending.

The treatment of Type 2 is not so straightforward because it could be an internal drainage problem, but the weak layer is normally underneath a strong layer and is therefore quite deep. The true nature of the problem needs to be determined before any major processing is considered but it may simply require an additional surface layer to "push" the weak layer to a lower (subbase) level.

The treatment of Type 3 is the same as for Type 1 but the chances of getting a strong enough layer by blending and/or compaction are probably less.

In Section 1 there are just 6 DCP results that indicate that additional material may be needed simply because the existing SN is too low (in contrast to much of the Section where the existing material is deficient in quality but where the overall SN is high because of thickness). These are at chainages 0.6, 1.1, 2.0, 2.9, 3.1 and 3.5 and correspond to the lowest subgrade strengths. The CUSUM graph shows that these are identified by the large jump in the y-coordinate that occurs between the chainages mentioned and the preceding chainage. They appear to be isolated points so they may be associated with local drainage issues. Five of them are also associated with weak bases so could possibly be corrected by reprocessing the existing material. The 6th (at chainage 2.9) may require an additional layer of base material (100 mm) because the existing base is only 100 mm thick and the SN is marginal. A subsidiary investigation is needed to determine the extent of these six problems.

Figure 13-24 shows the thicknesses required at each test point. The chainages with no vertical bar do not need additional strengthening but, based on practical issues, the entire section, after the six weak sections have been re-examined, will require the same treatment.

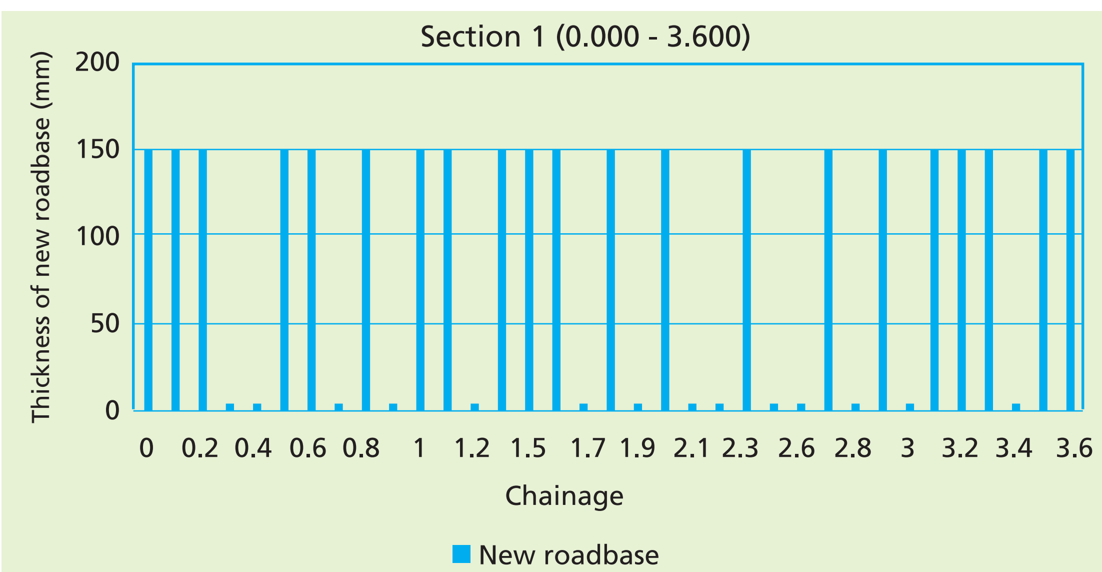


Figure 13-24: Section 1: Potential requirements for additional roadbase for each DCP test.

Section 2

Section 2 is much more uniform than Section 1, but 18 of the 34 test points show the same roadbase problems, as shown in Figure 13-25. There are 16 chainages that require no treatment but some of them (but not all) tend to occur slightly more often adjacent to one another in contrast to those in Section 1. For example chainages 4.0 to 4.2; 4.5 to 4.7; 6.8 to 7.0 but these sub-sections are also probably too short to warrant different treatment to that of the rest of the Section.

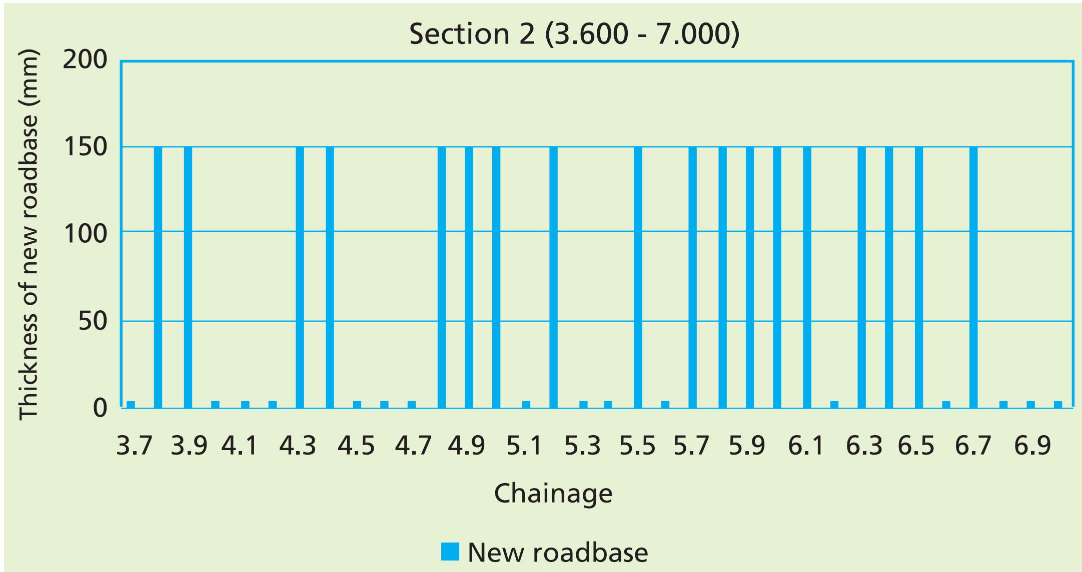


Figure 13-25: Section 2: Potential requirements for additional roadbase for each DCP test

Section 3

The whole of Section 3 (16 DCP test points) also shows roadbase deficiencies including a complete absence of any roadbase or subbase at many of the test points, as illustrated in Figure 13-26. The subgrade is quite strong hence a relatively thin pavement is required. Most of the Section is founded on an S4 subgrade or stronger and, except for three chainages (7.1, 7.4 and 7.8) already has a sufficiently high SN. The reason why additional material is needed is that the uppermost layer (be it roadbase, subbase or simply the top of a strong subgrade) are too weak for their position in the pavement. An additional 150 mm layer of suitable material on all the test points is required for a TLC 0.3 MESA design and, on the 8 test points with no subbase, an addition subbase layer is required. However, it is not usually feasible to change designs frequently hence the whole section requires two additional layers of pavement. The weakest test point is at chainage 7.1 and corresponds to the weakest subgrade and could therefore also be a drainage problem that needs investigating separately before a design for this area can be chosen.

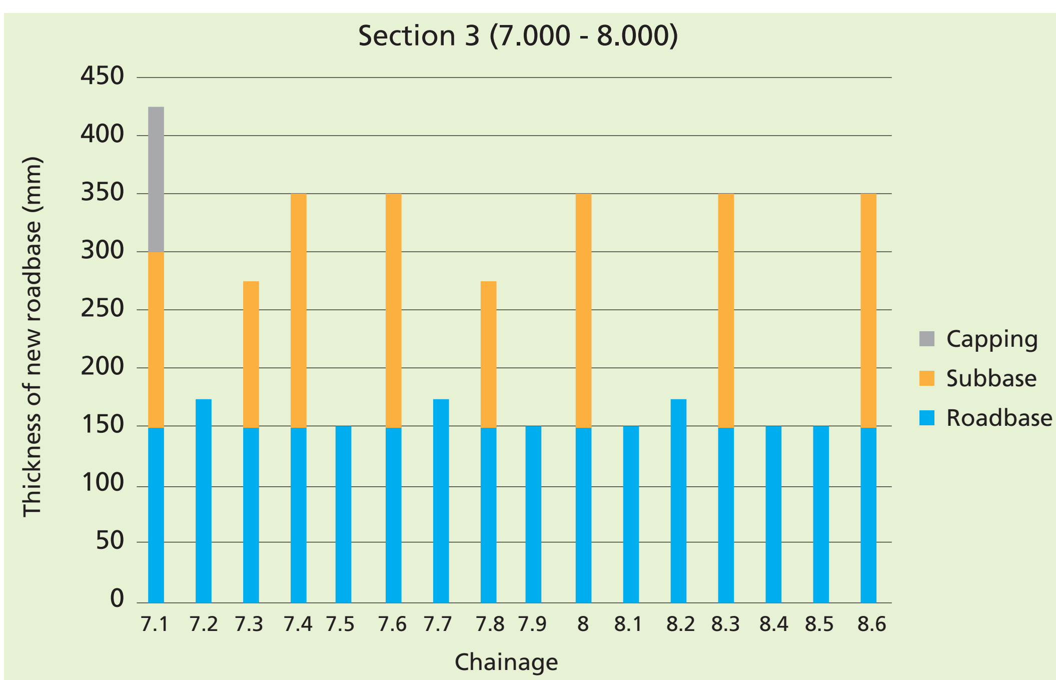


Figure 13-26: Section 3: Potential requirements for additional roadbase and subbase for each DCP test point

All of this information has been obtained directly from the DCP profiles on a point by point basis.

Step 6 Design for each uniform section

For each uniform section there is a range of values of ΔSNs but this is much smaller now that the uniform sections have been defined. The next step is to choose the appropriate percentile of those ranges for design. The percentile depends on the reliability required; the recommended values (Section 13.3.3) are:

- Median for TLC 0.01 and TLC 0.1
- Upper 75th percentile for TLC 0.3
- Upper 90th percentile for TLC 0.5 and TLC 1.0

Thus for TLC 0.3 a 75th percentile is recommended but for this example there are only two different designs for two sections and only three for Section 3 hence the choice of percentile requires no calculation. A summary is shown in the Table 13-19.

Table 13-19: Results of the analysis for road class TLC 0.3

Section	1	2	3
Chainage	0.000 – 3.600	3.600 – 7.000	7.000 – 8.600
Material to be added	150 mm of CBR 65%(1)	150 mm of CBR 65%(1)	150 mm of CBR 65% and 200 mm of CBR 30%

Note 1: Additional roadbase is needed only if the existing roadbase cannot be brought up to specified characteristics by other means, e.g. blending.

Note 2: Chainages with possible drainage problems (see text) require further investigation.

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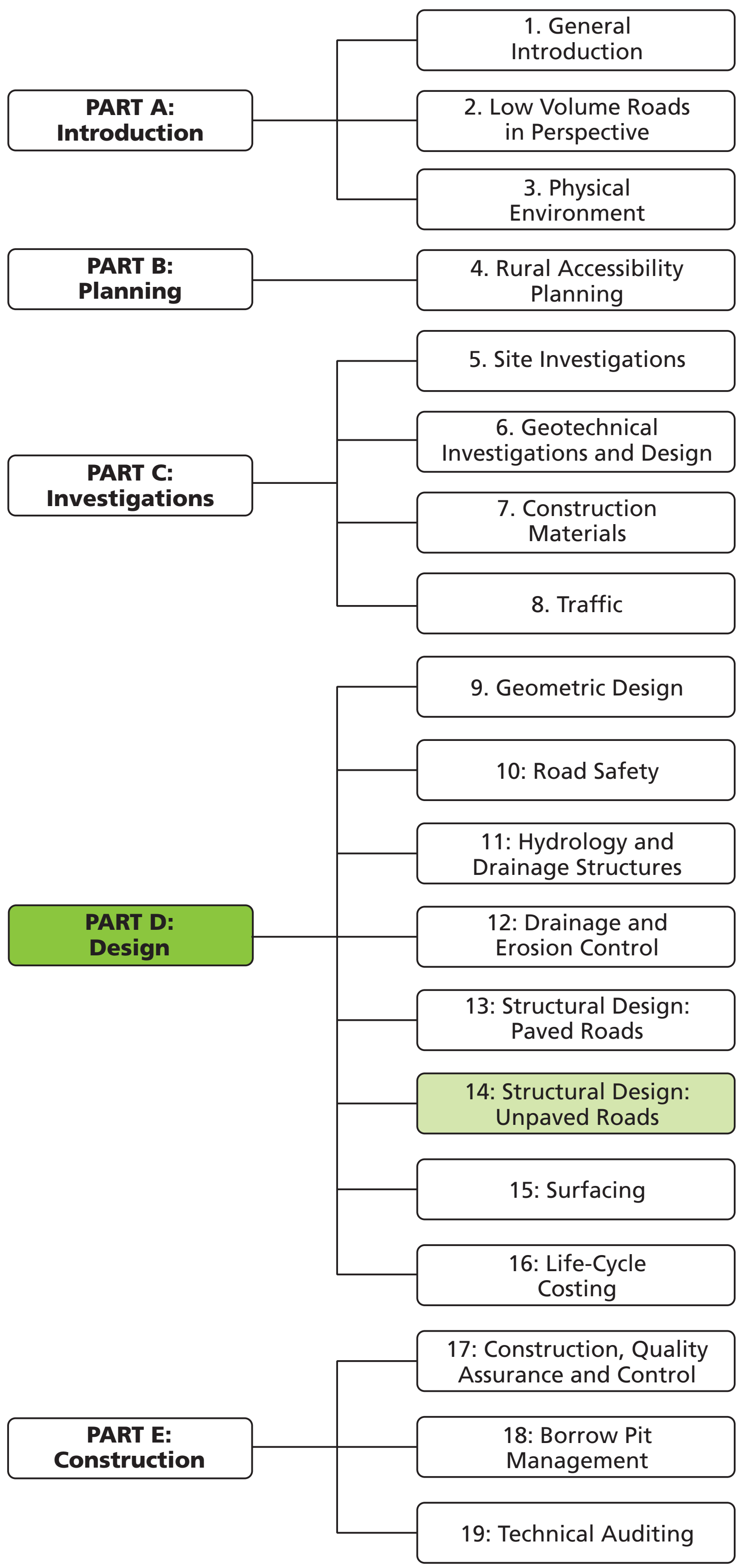
Low Volume Roads Manual



CONTENTS

14.1 INTRODUCTION

14.1.1 Background

More than 90% of the road network in Tanzania consists of unpaved roads. Although often rudimentary, these roads provide communities with access to important services (schools, clinics, hospitals and markets) and are the basis of a thriving market and social environment.

Unpaved roads are defined in this Manual as any road that is not surfaced with a "water proof" surfacing, whether this be bituminous, concrete, interlocking blocks, cobbles or similar surfacings.

In their simplest forms, unpaved roads consist of tracks or earth roads over which goods or persons are moved directly on the in situ material surface. This may in some cases be ripped, shaped and compacted (engineered) but generally the only compaction is that applied by vehicles moving over it (un-engineered).

There comes a point with these "roads" when passability is excessively affected by the weather and vehicles can no longer traverse the road during inclement weather. This problem is best solved by applying a selected material with specific properties over the in situ material to ensure all-weather passability and the roads then become "gravel" roads. Despite this the roads may occasionally become impassable as a result of flooding of parts of the road, in which case vehicles cannot pass because of deep water and not necessarily any reason attributed to the road surface.

Unpaved roads will usually carry a maximum of about 200 to 300 vehicles per day (with less than 10 % being heavy) but in areas where materials are poor, upgrading to paved standard can often be economically justified at traffic volumes much lower than this.

An example of the typical types of low volume road is shown in Figure 14-1.

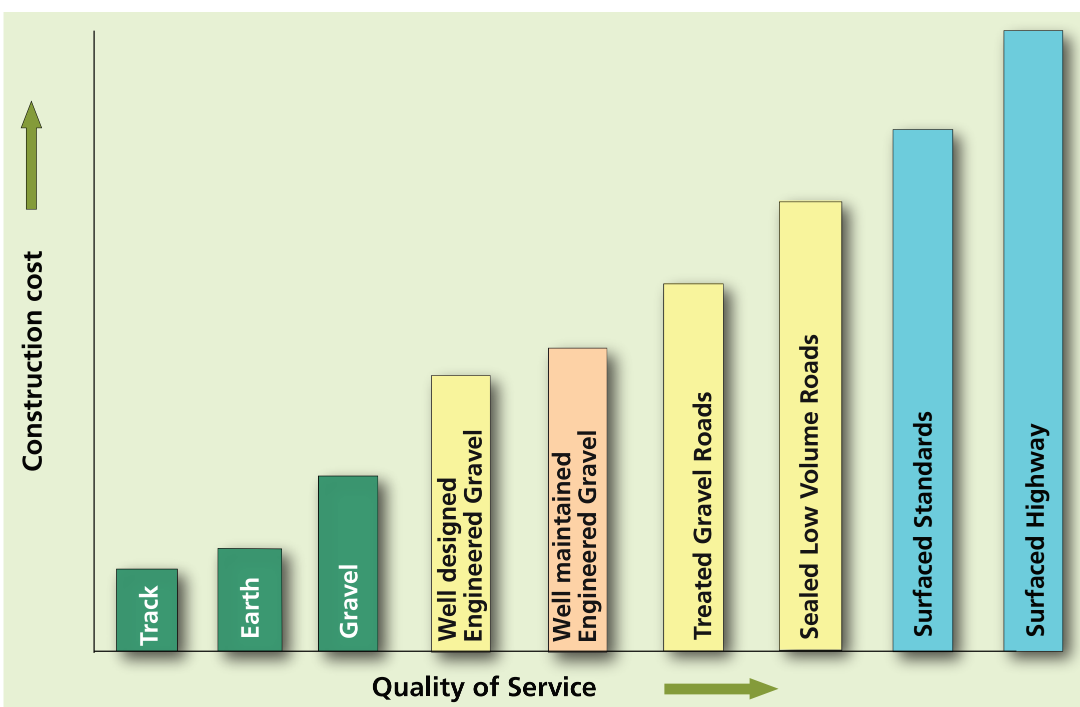


Figure 14-1: Hierarchy of roads showing unpaved and low volume sealed roads

14.1.2 Purpose and Scope

The purpose of this chapter is to provide a framework for the design of unpaved roads in an economic and sustainable manner such that the appropriate levels of quality are produced. The chapter has been developed so as to harmonise with the relevant sections described in the Pavement and Materials Design Manual (MOW, 1999) as far as possible. Innovations have been introduced where considered appropriate.

The chapter covers the design of all levels of unpaved roads from earth roads making use of the in situ soil to engineered and treated gravel roads. Material selection and thickness design are treated in detail.

14.2 EARTH ROADS

14.2.1 General

Earth roads may comprise either un-engineered roads on which traffic travels directly on the in situ material or engineered roads in which some attempt is made to improve the shape of the road, introduce side-drains and usually apply some compaction to the material. The wearing course material is generally obtained from excavation of the side-drains.

14.2.2 Un-engineered

Un-engineered roads usually start as one or two tracks in which the grass and surface vegetation is worn away to expose the in situ material or in some cases the vegetation may be intentionally removed. With time and traffic, these tend to wear down and depressions develop in the natural ground surface. These become areas that collect precipitation or surface run-off and form conduits moving the water, which leads to softening of the material, erosion and ultimately deepening of the channels. At this stage the tracks no longer afford viable routes for traffic and new tracks are formed adjacent to the existing ones, ultimately resulting in a wide “canal”.

The life and effectiveness of earth roads depends on the nature of the in situ material. Often the upper part has humus and clay, which results in some sort of binding of the material, which can be improved by the “reinforcing” effect of any roots. Once these wear away the track will usually deteriorate rapidly.

In some instances, the in situ material may have properties equivalent to those required for conventional wearing course gravels, in which case they may perform reasonably well for a limited period. It should be remembered that these materials are generally not compacted and rely solely on traffic compaction to increase their density, which is accompanied by settlement and some material loss.

Only once the earth road starts deteriorating in riding quality, is it graded and given some shape but the overall structure is usually below natural ground level and the associated drainage problems are not addressed. At this stage the road needs to be improved.

14.2.3 Engineered

Engineered or improved earth roads differ from the un-engineered earth roads described above in that the shape of the road structure is improved. The materials used are the same as the earth road, i.e. the in situ or local material but additional material is excavated from the side of the road to form side drains (at least 150 mm below natural ground level) and this material is added to the road to increase its height and provide a better drained road structure, as shown in Figure 14-2.The material must be shaped to assist with water runoff and compacted to improve its strength, decrease its permeability and reduce maintenance requirements.

The crossfall of carriageway and shoulders for all engineered and gravel roads shall be 4 to 6% depending on local conditions, to prevent potholes developing by ensuring rapid removal of water from the road surface and to ensure that excessive crossfall does not cause erosion of the surface. Although a maximum of 5% is usually recommended, it is often useful to construct a camber of 6%, as 1% is usually lost soon after construction.

The crown height of the improved earth road should be at least 35 cm above the bed of the side drains, which must be graded and lead into regular side and mitre drains to remove water from adjacent to the road as rapidly as possible. The invert of the side drain should be at least 150 mm below the bottom of the wearing course.

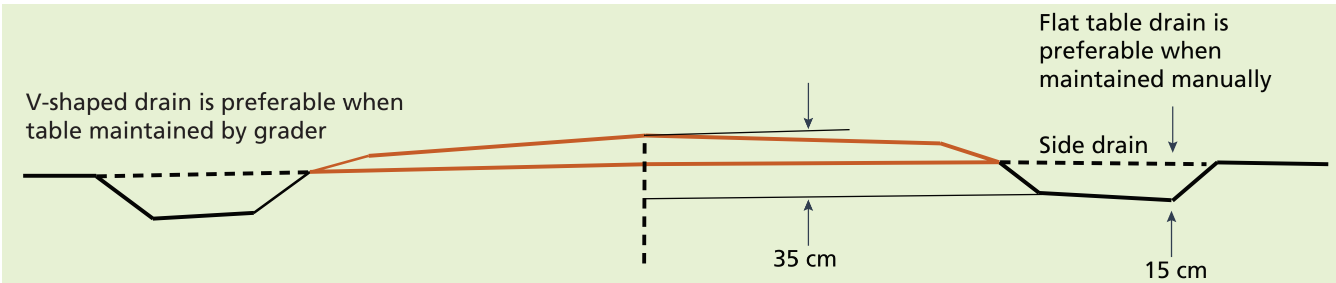


Figure 14-2: Cross section of typical improved earth road

The performance of earth roads is constrained by the quality of the in situ materials, which in many cases is inadequate to provide good all-weather surfaces that require minimal maintenance. A knowledge of the past performance of local materials may, however, allow the use of these even though they do not comply with the required properties for good wearing course gravels. In general, no specific material requirements are applicable to earth roads but if the local materials comply with the requirements for gravel roads, a good performance can be expected.

It is possible to estimate the likely performance of improved earth roads based on an assessment of the traffic carrying capacity of the soils under varying environmental conditions from a knowledge of the bearing capacity (CBR) of the soil, the equivalent single wheel load of the vehicles and the tyre pressures, as shown in Figure 14-3. If the strength of the earth road material is known (in terms of its in-situ CBR), the nomograph permits predictions of the expected number of vehicles that will cause a rut depth of 75 mm.

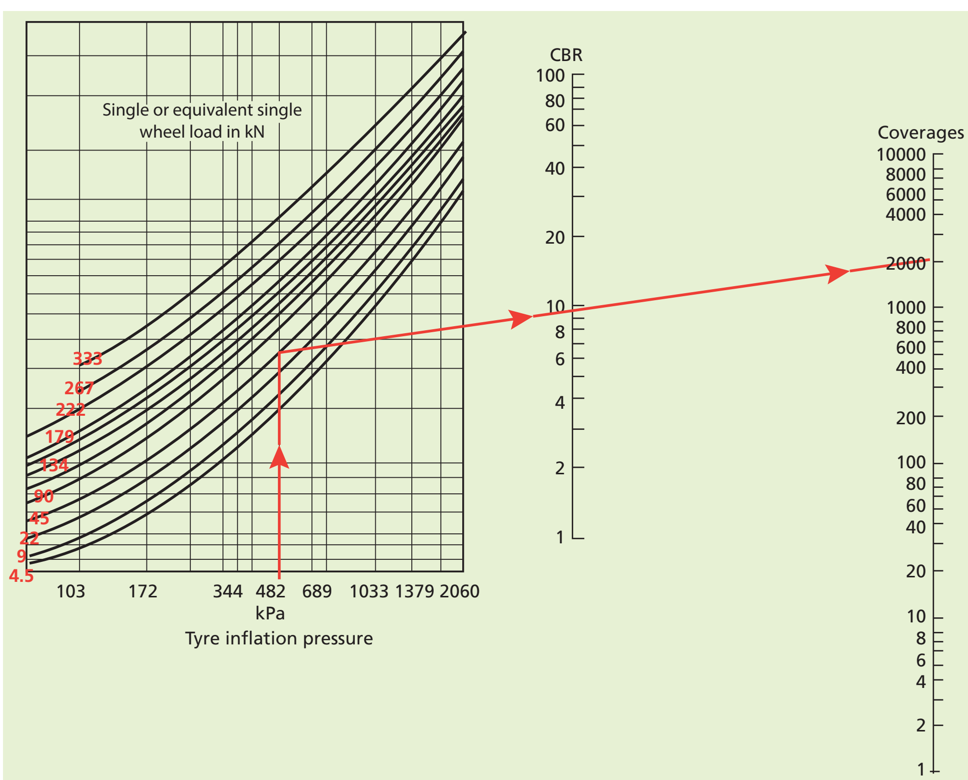


Figure 14-3: Carrying capacity of engineered earth roads

Source: Ahlvin and Hamitt (1975).

As illustrated in Figure 14-3, an engineered earth road with an in-situ CBR of 10% can be expected to provide approximately 2 000 coverages of vehicles with a single wheel load of 45 kN and a tyre inflation pressure of 482 kPa before serious deformation is likely to occur. Since the wheel loads will not be concentrated on exactly the same path, but will wander slightly across the width of a road, one complete coverage is equivalent to the passage of 2.7 vehicles. Thus, 2,000 coverages are equivalent to 5,400 vehicles with the characteristics indicated above.

For a single lane road, the wheel loads will be restricted to narrower channels and therefore the coverages will be different. For example, for a narrow single lane road the number of vehicles that the earth road can accommodate before failure decreases to approximately 1350 vehicles. For a route carrying 50 vpd and assuming 15% of them are relatively heavy (4.54 tonne wheels), this translates into a need to maintain, re-grade or reshape the surface about every 4 to 6 months. For soils with higher CBR this will be longer. It is important for both designers and road managers to appreciate that engineered earth roads have a low initial cost but that they require an ongoing commitment to regularly reshape the surface to keep it in a serviceable condition.

Areas that may have specific problems (usually due to water or to poor subgrade materials) may be treated in isolation, by localised replacement of subgrade, gravelling, installation of culverts, raising the roadway or by installing other drainage measures. This is the basis of a “spot improvement” approach and should be carried out to the best standard possible as these areas will then be in a condition that is suitable for later upgrading to gravel road standard.

Where the topography allows, wide, shallow longitudinal drainage for earth roads is preferred. These minimise erosion, and will not block as easily as narrow ditches. The ditches grass over in time, binding the soil surface and further slowing down the water flow speed, both of which act to prevent or reduce erosion.

Culverts should be installed perpendicular to the route where there is a need to transfer water from one side of the road to the other, for example where the road crosses a watercourse. In flat areas, smaller diameter parallel culverts may be preferable to single large culverts, in order to ensure discharge is at ground level. However, culvert pipes smaller than 750 mm in diameter are not recommended as they are difficult to clean out of silt and debris. The inlet and outlet of the culvert must be protected against erosion.

At some point (usually dictated by the number of vehicles increasing to a certain level and depending on the material quality) the maintenance requirements for earth roads reach a stage that it becomes uneconomical or excessively difficult. At this stage, a decision must be made to construct a traditional gravel road in which materials form a selected borrow source are used for the wearing course.

14.3 GRAVEL ROADS

14.3.1 General

Roads described as gravel roads imply that a number of factors have been taken into account in their design and construction. These include:

- Material of a selected quality is used to provide an all-weather wearing course.
- The structure of the road and strength of the materials is such that the subgrade is protected from excessive strains under traffic loads.
- The shape of the road is designed to allow drainage of water (mainly precipitation) from the road surface and from alongside the road.
- he necessary cross and side drainage is installed.
- The road is constructed to acceptable standards, including shape, compaction and finish.

Although an all-weather wearing course is provided, the road may not necessarily be passable at all times of the years as a result of low level water crossings being flooded periodically. This, however, is not a function of the gravel roads design and is addressed under a different section of this Manual. (*Chapter 11- Hydrology and Drainage Structures).*

14.3.2 Materials

The critical aspect of gravel roads is obviously the material selection. The use of incorrect materials in the wearing course will result in roads that deform, corrugate, become slippery when wet, lose gravel rapidly and generate excessive dust. Table 14-1 summarises the required properties of good wearing course gravels and these are based on materials tested using the standard Tanzanian test methods.

Table 14-1: Specification requirements for wearing course materials for unpaved roads

Maximum nominal size	37.5 mm
Minimum percentage passing 37.5 mm	95
Shrinkage product (SP)	140 – 400 (260)
Grading coefficient (Gc)	14 – 30
Min DN value (mm/blow)	13.5 at 95% BS Heavy compaction (soaked)
Treton Impact value (%)1	20 – 65

Note 1: The Treton impact value is not a standard Tanzanian test but is described in TMH 1 (1985). It is a simple test and makes use of equipment that can be easily manufactured. No correlation currently exists with the similar BS Aggregate Impact Test.

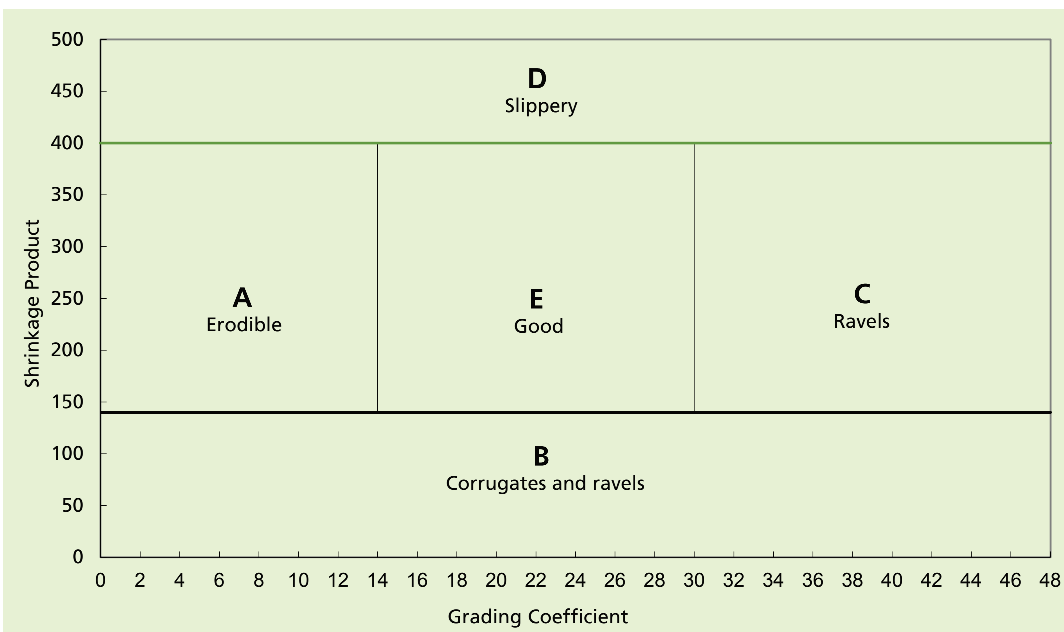


Figure 14-4: Chart showing performance of unpaved road materials

Note: Figure 14-4 is based on CML test methods. Should the results be determined using the CSIR Gravel Roads Test Kit (based on TMH 1 test methods), the chart provided with the kit should be used.

The recommended grading and cohesion (shrinkage) specifications for gravel wearing course materials can also be shown diagrammatically in relation to their predicted performance, as shown in Figure 14-4.

On the chart presented in Figure 14-4, the 5 zones indicated (A to E) show the expected performance of materials as follows:

- Zone A: Fine grained material prone to erosion.
- Zone B: Non-cohesive materials that lead to corrugation and ravelling/loosening.
- Zone C: Poorly graded materials that are prone to ravelling.
- Zone D: Fine plastic material prone to slipperiness and excessive dust.
- Zone E: Optimum materials for best performance.

Requirements for both material and aggregate strength are provided. The material strength is specified as the DCP DN value (13.5 mm/blow) which initially appears very low but investigation of many roads in various countries has shown that material with a strength as low as this will not shear or deform under the passage of an 80 kN axle load (20 kN single tyre load), even when soaked. Materials of significantly higher quality than this should be preserved for later use in paved roads. The Treton impact value differentiates between aggregate particles that will perform well (20 to 65), aggregates that are too soft and will disintegrate under traffic (> 65) and aggregates that are too hard to be broken down by conventional or grid rolling during construction and will result in stony roads if large particles are not removed.

Figure 14-4 can be used to identify potential problems that could affect the road should the materials not fall into Zone E. These can be taken into account and engineering judgement used to override the limits where necessary. For instance, in arid areas where rainfall is rare, the need to limit the upper shrinkage limit can be re-evaluated. Consideration may be given to using a high plasticity material in these areas with appropriate warning signs, provided that the road has no steep grades or sharp bends. Similarly, roads with light, slow moving traffic are unlikely to corrugate and non-cohesive materials could be considered under these conditions or if the application of regular light surface maintenance is possible.

Gravel roads in areas of material scarcity

In situations where natural materials are scarce, performance results have shown that blended materials can work well. Successful blends can be obtained through:

- Mixing non-plastic sand with clayey sand.
- Mixing non-plastic sand with high PI calcrete.
- Mixing clayey material with low plasticity gravels (derived from granite and limestone).

Before blending, laboratory tests should be performed to ensure that the blends produce the required DN values and that the blended materials meet the selection criteria specified in Table 14-1. The laboratory testing should use various blend ratios to determine which are best and these ratios must be carefully adhered to and controlled during construction. The use of material not complying with the specifications can result in severe deformation, rutting and impassability when wet.

14.3.3 Thickness Design

Unlike paved roads, any minor deformation of the support layers beneath the gravel wearing course does not unduly influence the performance of the road. The reason for this is that in paved roads, the cumulative deformation in the subgrade ultimately leads to rutting of the bituminous surfacing over the design or service life of the road, whereas in unpaved roads any minor rutting of deformation (excluding serious shear failures) is made up during routine grader maintenance and traffic wander. Even shear failures, although undesirable, are usually repaired (at least temporarily) during routine grader maintenance.

The need to invest in a series of structural layers is thus seldom warranted for unpaved roads. However, a number of decisions are required during the design to satisfy the following requirements:

- The wearing course must be raised above the surrounding natural ground level to avoid moisture accumulation – the material imported to raise the formation should be of a specified quality.
- Raising the formation to allow pipes and culverts for cross-road drainage to pass beneath/through the road should make use of a specified quality of material.
- Very weak or volumetrically unstable subgrade materials must be taken care of by removing, treating or covering with an adequate thickness of stable material – heave and collapse are seldom significant problems on unpaved roads, being smoothed out during routine maintenance.
- Should regravelling operations be delayed until the gravel has completely worn away (which is a regular occurrence in many countries), a “buffer” layer of reasonable quality material should be in place to avoid vehicles travelling on very weak material.
- The maintenance capacity and frequency are thus important considerations in the pavement design.
- If it is likely that the road will be upgraded to paved standard within six to 10 years after construction, selected materials complying with the requirements for lower layers in the paved road design standards should be used.

Subgrade definition

Notwithstanding the above discussion, it is good practice to assess the subgrade conditions for gravel roads and to base the pavement structure on these in order to get a balanced pavement design. In a similar manner to the Pavement and Materials Design Manual (MOW, 1999), the subgrade should be divided into uniform sections on the basis of the centre-line survey. However, a deviation from the current practice is that this should be done using a Dynamic Cone Penetrometer (DCP) which is much quicker and cheaper than the conventional CBR method. Slightly different methods will be used for a new road and the improvement of an existing earth road.

At least 5 DCP tests to 800 mm depth should be carried out per kilometre of road, alternating between the outer wheel tracks in each direction for an existing road and alternating with 2 m offsets to the left and right of the centre-line for a new road after removing the upper soil layer containing humus, vegetable matter or any other undesirable materials. If the subgrade conditions appear to be highly variable, the frequency of testing should be increased, even up to one test per 50 m if necessary.

Earth Roads

For an earth road or an old gravel road being upgraded the process below should be followed:

- Determine the DCP penetration rate for the upper 150 mm and the 150 -300 mm layers of the existing structure (DN_{150} and $DN_{150\text{-}300}$).
- Determine the DCP structural number (DSN_{800} or number of blows to penetrate 800 mm).
- Plot the data using a cumulative sum (CUSUM) technique to determine uniform sections. If the uniform sections delineated by the three parameters (DN_{150}, $DN_{150\text{-}300}$ and DSN_{800}) differ significantly it is necessary to look at the individual DCP profiles and decide whether the differences are significant. Low DSN_{800} values indicate weak support while low DN150 values indicate that the upper 150 mm of the road is weak.

New Roads

A similar process is carried out for new roads bearing in mind that the upper 150 mm layer will at least be ripped and recompacted as the in situ material and that formation material will usually be imported to raise the level of the road above natural ground level.

DCP testing is carried out at in situ moisture and density conditions. It is recommended that the testing is done at the end of the wet season (i.e. the subgrade is probably in or near its worst moisture condition), but some interpretation (judgement) may be required at the time of the DCP test survey regarding the moisture conditions. It must be noted how the subgrade conditions are expected to relate to their condition in service, i.e. is the subgrade likely to be in a similar state, wetter or drier in service than when the survey was carried out? Areas that are expected to be soaked or flooded periodically must also be noted.

On this basis, the in situ material condition should be divided into uniform sections with a characteristic subgrade strength for each section. The subgrade DN values will be determined as a percentile of the values determined for each uniform section as described in the following section.

Pavement layer design

Once the uniform sections have been defined, the subgrade can be classified in terms of its required strength to carry the expected traffic. This makes use of the following procedure:

- The characteristic subgrade strength for each uniform section is determined by assessing the DN_{150}, $DN_{150-300}$ and DSN_{800} values for each of the identified uniform sections. There should be at least 8 to 30 results for each uniform section for statistical validity.
- Determine the 80th, 50th and 20th percentiles of the DN results for each uniform section in a similar manner to that described for paved roads (Section 13.6).
- Based on the moisture regime at the time of testing the following percentiles of the data shall be used to determine the design strength of the two upper layers, as shown in Table 14-2. The mean (50th percentile) can be used for the less-critical underlying layers (below 300 mm).

Table 14-2: Suggested percentile of minimum in situ DCP penetration rates to be used

Site moisture condition during DCP survey	Percentile of strength profile (maximum penetration rate – DN)	
	Materials with strengths not moisture sensitive*	Materials with strengths that are moisture sensitive*
Wetter than expected in service	20	20 – 50
Expected in service moisture	50	50 – 80
Drier than expected in service	80	80 - 90

* Moisture sensitivity can be estimated by inspecting and feeling a sample of the material – clayey materials (PI > about 12%) can be considered to be moisture sensitive.

- Compare the relevant subgrade strength profiles with the necessary design given in Table 14-3, or the layer strength diagrams shown in Figure 14.5, for the specified traffic categories.

It should be noted that only the upper two layers are critical, the underlying layers being given values in an attempt to improve the pavement balance. It can be seen that the in situ strengths of the third layer (300 – 450 mm) and below range from 19 to 50 mm/blow, which are likely to occur in most situations. If these do not compare adequately (low DSN_{800}), additional thickness of material at the surface may be necessary. It should also be borne in mind that in most cases some formation material is likely to be placed on this in situ profile, this imported material having an in situ DN value of between 14 and 25 mm/blow depending on the traffic.

Table 14-3: Thickness and strength design of support structures for different traffic categories (DN)

Layer, depth and/or structural number	Traffic			
	≤ 2 heavy vehicles per day	2 - 6 heavy vehicles per day	7 - 20 heavy vehicles per day	21 - 60 heavy vehicles per day
	DN (mm/blow)			
Formation or upper 150 mm (≤95% heavy compaction)	25	19	14	14
In situ (Rip and recompact) 150-300 mm (≤95% heavy compaction)	33	25	19	14
300 - 450 mm	50	33	25	19
450 - 600 mm	50	50	33	25
600 - 800 mm	50	50	50	33
DSN_{800}	21	25	33	41

Note: Heavy vehicles are defined as those vehicles classified as HGV and above (Classes F, G and H in Table 8-1, Chapter 8-Traffic).

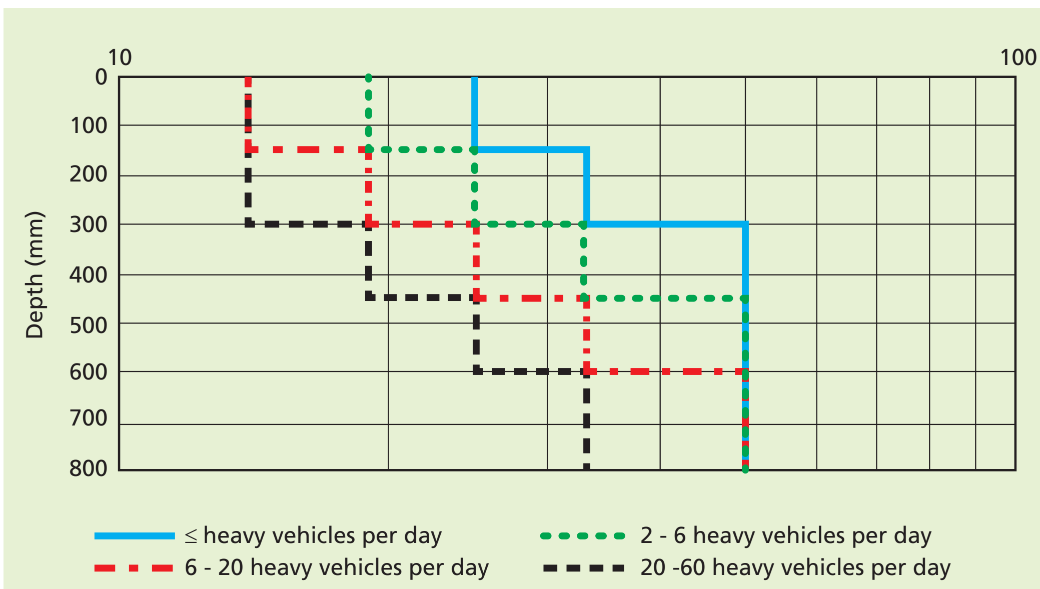


Figure 14-5: Layer strength diagrams of support layers for different traffic categories

If the in situ profiles (selected percentiles) compare adequately with the layer strength diagrams, the wearing course layer can be placed on top. This would normally consist of 150 mm of specified material, as shown in Table 14-1 and Figure 14-4, but if the potential for delayed maintenance (i.e. regravelling) exists, an additional 50 mm should be added as a buffer layer.

The minimum strength of the support layer beneath the wearing course need not be very high and actually becomes equal to the required minimum strength of the wearing course for higher traffic. This material may not have the necessary cohesive or grading properties to provide the necessary performance as a wearing course but must always be present. If this material complies with the requirements of Zone E in Figure 14-4, the total thickness of the upper 150 mm formation and the wearing course can be reduced to 225 mm.

It should be pointed out that the design is based on number of heavy vehicles per day and not cumulative axle loads as traditionally used for paved roads. This is a result of the mode of distress being related to shear failure of the layers under loading as opposed to cumulative deformation with time, which is removed during routine maintenance and regravelling. The reliability of the design is thus accepted as being slightly lower as the repair of any possible failures is much less disruptive than traditional paved road repairs.

Wearing course thickness design

This must take into account the fact that gravel will be lost from the road continuously. Other than the road user costs, this is the single most important reason why gravel roads are expensive, and often unsustainable, in whole life cost terms, especially when traffic levels increase.

Reducing gravel loss by selecting better quality gravels, modifying the properties of poorer quality materials and ensuring high levels of compaction is one way of reducing long term costs. Gravel losses (gravel loss in mm/year/100vpd) are a function of a number of factors: climate, traffic, material quality, road geometrics, maintenance frequency and type etc., and can be predicted using various models. These, however, often need regional calibration but an approximate estimate can be obtained from Table 14-4.

Table 14-4: Typical estimates of gravel loss

Material Quality Zone	Material Quality	Typical gravel loss (mm/yr/100vpd)
Zone A	Satisfactory	20
Zone B	Poor	40
Zone C	Poor	40
Zone D	Marginal	20
Zone E	Good	15

The gravel losses shown in Table 14-4 hold only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the wearing course is reduced in thickness, other developments, such as the formation of ruts or heavy grader maintenance, may also affect the loss of gravel material. However, the rates of gravel loss given above can be used as an aid to the planning for re-gravelling in the future.

The rates of gravel loss increase significantly on gradients greater than about 6% and in areas of high and intense rainfall. Spot improvements should be considered on these sections.

Re-gravelling should take place before the underlying layer is exposed. The re-gravelling frequency, R, is typically in the range 5 - 8 years.

The optimum wearing course thickness = R x GL

Where:

R = re-gravelling frequency in years
GL = expected annual gravel loss.

Where suitable sand is available adjacent to the road, the application of a sand cushion (25 to 40 mm) on top of the wearing course allows low-cost regular maintenance of the road and preserves the wearing course from wear and material loss as long as the sand covers the road.

14.4 TREATED GRAVEL ROADS

It is often difficult to locate suitable materials for unpaved roads or costly to haul them from some distance away. Numerous proprietary chemicals are being marketed that claim to improve almost any soil to a quality suitable for road construction. These chemicals can have mixed results and are very material dependent.

There are essentially two uses of these chemicals – those used for dust palliation and those used for soil stabilization/improvement. Despite these main uses, there can be some overlap in that, for example, dust palliatives may strengthen the upper part of the treated layer and reduce gravel loss.

Application of the products can be through surficial spraying or mixing in. Again, certain products are better and more cost-effectively mixed in (at greater cost) than being sprayed on the surface of the road.

No general guidance on the use of the chemicals can be given as the types, actions and uses can differ widely. However, the following aspects should be considered before using any chemical:

- *Is the use of the chemical going to be cost effective and give some kind of financial, social or environmental benefit that is value for money?* It may often be more cost-effective to import a better material from further away.
- Does the chemical consistently increase the strength of the material, if it is to be used as a stabiliser? This can be checked in a laboratory using traditional CBR testing – however, it has been found that the application rate is critical, some materials react better with chemicals but this may vary considerably within a material source and ongoing testing of the compatibility between material and chemical must therefore be carried out.
- Products used for dust palliation are best tested on short sections of road before full-scale use. It is very difficult to test their effectiveness in the laboratory as a result of the speed and abrasion of vehicles that generate dust.
- Many of the chemical products are costly and where used, it may often be more cost effective to place a bituminous surfacing on the material to conserve it for the full life of the road than to allow it to be lost in the normal gravel loss. The gravel loss may be reduced, but the road is still an unpaved road and will still be subjected to traffic and environmental erosion and material loss.

14.5 EXAMPLE OF DCP DESIGN METHOD

The use of the DCP method for the design of a typical unpaved road being upgraded from an existing track is illustrated below. The expected traffic is between 6 and 20 heavy vehicles per day.

DCP tests were carried out every 500 m (every 200 m would have been preferable) and the results analysed using WinDCP5.1 as described in Section 13.6.1. The results were then tabulated in a spreadsheet as illustrated in Table 14-5, and the CUSUMs calculated for all of the DSN_{800}, $DN_{150\text{-}300}$, $DN_{301\text{-}450}$ and $DN_{451\text{-}800}$ values. This data was then used to identify the uniform sections as illustrated in Figure 14-6. Four distinct uniform sections are shown by the majority of the plots (0 to 2.5 km, 2.5 to 4.5 km, 4.5 to 7.5 km and 7.5 to 9.0 km). The 20^{th} and 80^{th} percentiles of each parameter for each of these uniform sections was then determined using the EXCEL function as shown in Figure 14-5.

Table 14-5: Spreadsheet showing the CUSUM calculation

CUSUM Analysis Unpaved Example																
Test no	Chainage	Position	DSN800			0-150 mm			151-300 mm			301-450 mm			451-800	601-800
			DSN	DSN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN
1	0.000	RHS	179	88.30	88.30	0.68	-18.72	-18.72	2.59	-18.41	-18.41	2.03	-16.78	-16.78	1.23	
2	0.500	RHS	357	266.30	354.60	0.68	-18.72	-37.43	0.58	-20.42	-38.83	0.92	-17.89	-34.68	0.45	
11	1.000	RHS	188	97.30	451.90	4.01	-15.39	-52.82	4.20	-16.80	-55.62	4.54	-14.27	-48.95	6.38	
3	1.500	RHS	160	69.30	521.20	0.56	-18.84	-71.65	4.06	-16.94	-72.56	2.21	-16.60	-65.55	1.06	
4	2.000	RHS	150	59.30	580.50	3.85	-15.55	-87.20	9.10	-11.90	-84.46	22.11	3.30	-62.25	4.80	
115	2.500	LHS	134	43.30	623.80	4.55	-14.85	-102.05	4.10	-16.90	-101.36	8.50	-10.31	-72.57	18.70	
5	3.000	LHS	43	-47.70	576.10	25.95	6.55	-95.49	21.52	0.52	-100.83	22.94	4.13	-68.44	16.07	
7	3.500	RHS	34	-56.70	519.40	54.00	34.60	-60.89	46.80	25.80	-75.03	20.90	2.09	-66.35	17.20	
8	4.000	LHS	52	-38.70	480.70	15.54	-3.86	-64.74	40.00	19.00	-56.03	18.90	0.09	-66.26	12.30	
9	4.500	LHS	33	-57.70	423.00	29.10	9.70	-55.04	21.20	0.20	-55.83	22.10	3.29	-62.98	33.50	
111	5.000	LHS	90	-0.70	422.30	6.40	-13.00	-68.04	8.80	-12.20	-68.02	7.70	-11.11	-74.09	15.60	
14	5.500	LHS	79	-11.70	410.60	5.20	-14.20	-82.23	11.30	-9.70	-77.72	14.10	-4.71	-78.80	21.90	
114	6.000	RHS	83	-7.70	402.90	7.60	-11.80	-94.03	8.50	-12.50	-90.22	16.80	-2.01	-80.81	11.95	
10	6.500	LHS	54	-36.70	366.20	12.30	-7.10	-101.12	11.80	-9.20	-99.42	16.40	-2.41	-83.23	21.20	
12	7.000	LHS	48	-42.70	323.50	31.60	12.20	-88.92	18.10	-2.90	-102.31	12.00	-6.81	-90.04	18.30	
13	7.500	RHS	37	-53.70	269.80	22.30	2.90	-86.02	15.70	-5.30	-107.61	17.80	-1.01	-91.05	29.30	
15	8.000	RHS	36	-54.70	215.10	53.40	34.00	-52.01	46.40	25.40	-82.21	22.20	3.39	-87.66	15.30	
117	8.500	RHS	21	-69.70	145.40	27.70	8.30	-43.71	44.20	23.20	-59.01	47.00	28.19	-59.48	44.40	
116	9.000	LHS	15	-75.70	69.70	45.00	25.60	-18.10	43.00	22.00	-37.00	45.00	26.19	-33.29	75.00	
112	9.500	RHS	21	-69.70	0.00	37.50	18.10	0.00	58.00	37.00	0.00	52.10	33.29	0.00	41.30	
			90.70			19.40			21.00			18.81			20.30	

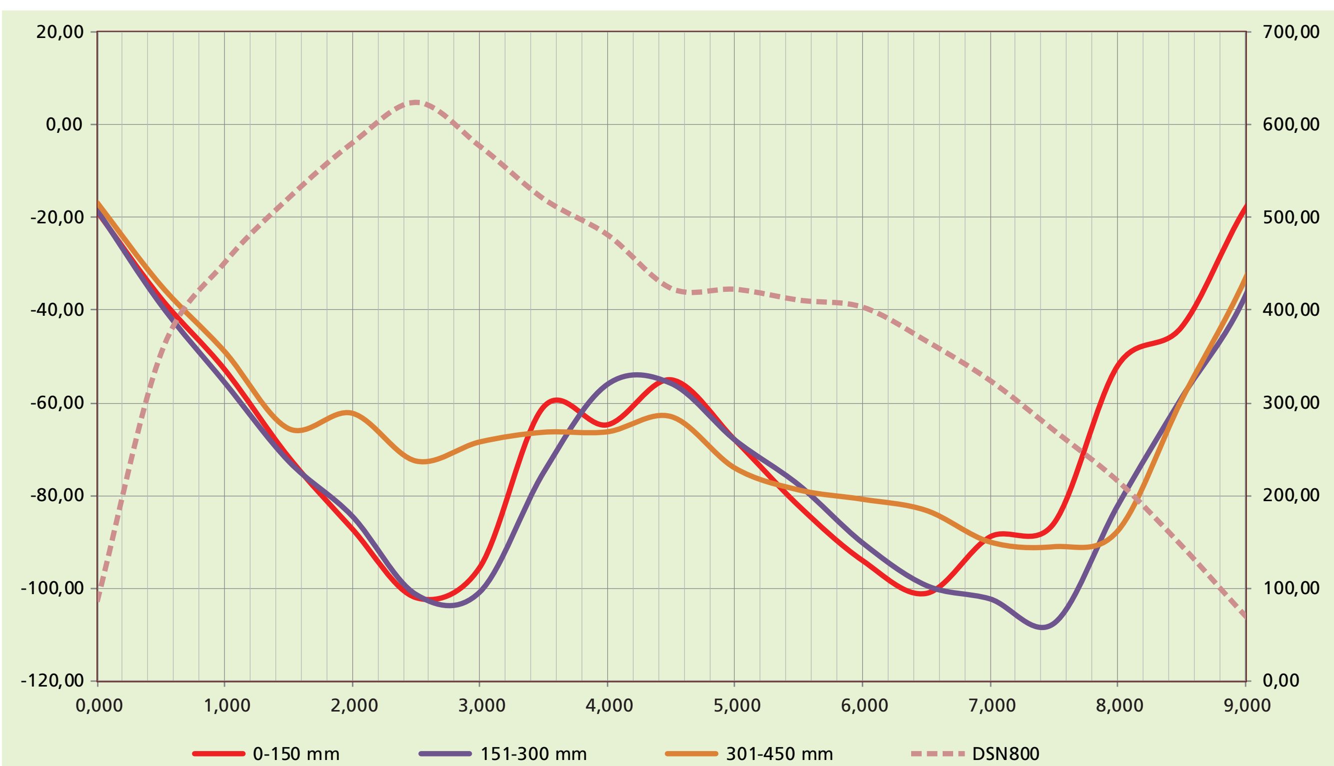


Figure 14-6: Plot of CUSUMs for different parameters

Table 14-6: Summary of percentile values for each parameter in each uniform section

Uniform section	%ile 0-150		%ile 151-300		%ile 301-450		%ile 451-800		%ile DSN800	
	20	80	20	80	20	80	20	80	20	80
0 - 2.5 km	0,7	4,0	2,6	4,2	2,0	8,5	1,1	6,4	188	150
2.5 - 4.5 km	21,8	39,1	21,4	42,7	20,1	22,4	15,6	18,7	90	34
4.5 - 7.5 km	6,4	22,3	8,8	15,7	12,0	16,8	15,3	21,9	79	37
7.5 - 9.0 km	33,6	48,4	43,7	51,0	35,9	49,0	20,3	44,4	37	21

These percentiles were then compared with the required layer strength profiles provided in Table 14-3 as shown in Figure 14-7. As the testing was done when the area was considered to be drier than its expected in situ moisture conditions during the service of the road, the 80th percentile values of the strength were used in the analysis. Different results were obtained for each of the uniform sections as follows:

Table 14-7: Structural properties of each uniform section (tested in dry period)

Design class 6-20 hvpd	Spec. DN/layer mm	Uniform Section 1 km 0+00 - 2+500 80th %-ile	Uniform Section 2 km 2+500 - 4+500 80th %-ile	Uniform Section 3 km 4+500 - 7+500 80th %-ile	Uniform Section 4 km 7+500 - 9+000 80th %-ile
0-150 mm	14	4,01	39,10	22,30	48,40
151-300 mm	19	4,20	42,70	15,70	51,00
301-450 mm	25	8,50	22,40	16,80	49,00
451-800 mm	33	6,40	18,70	21,90	44,40
DSN800	33	150	34	37	21

Marginal/can be improved	Outside spec.	Within spec.

Section 1: The existing conditions are structurally adequate and the road needs only to be shaped and re-compacted. It is still, however, necessary to confirm that the material to be used for the wearing course (upper 150 to 250 mm) complies with the requirements of Table 14-1 to ensure good functional performance.

Section 2: The upper 300 mm of the existing structure is inadequate. As the material is not even strong enough for the support layers, it would need to be removed or improved with some form of mechanical or chemical treatment. Material complying with the requirements of Table 14-1 would need to be imported for the wearing course.

Section 3: The upper 150 mm of the structure is inadequate and would need to be replaced or improved. The remainder of the structure is adequate.

Section 4: This area is a major problem (moist stream bed and marshy area) and would need to be carefully designed with a totally imported structure of at least 800 mm thick This would probably include a rockfill overlain with selected materials and even possibly some geotextile separation and drainage layers. An alternative route bypassing this area would in many cases probably be more cost-effective.

If the DCP survey had been carried out during a wet period, use of the 20th percentile results would be more appropriate and the conditions illustrated in Table 14-8 would be used in the design. It can now be seen that Sections 1 and 3 are adequate, Section 2 would require replacement and the upper 150 mm and reworking of the underlying material between 150 and 300 mm. Section 4 would remain a problem area, with significant challenges.

Table 14-8: Structural properties of each uniform section (tested in wet period)

Design class 6-20 hvpd	Spec. DN/layer mm	Uniform Section 1 km 0+000 - 2+500 20th %-ile	Uniform Section 2 km 2+500 - 4+500 20th %-ile	Uniform Section 3 km 4+500 - 7+500 20th %-ile	Uniform Section 4 km 7+500 - 9+000 20th %-ile
0-150 mm	14	0.68	21.80	6.40	33.60
151-300 mm	19	2.59	21,4	8.80	43.70
301-450 mm	25	2.03	20.10	12.00	35.90
451-800 mm	33	1.06	15.60	15.30	20.30
DSN800	33	188	90	79	37

Marginal/can be improved	Outside spec.	Within spec.

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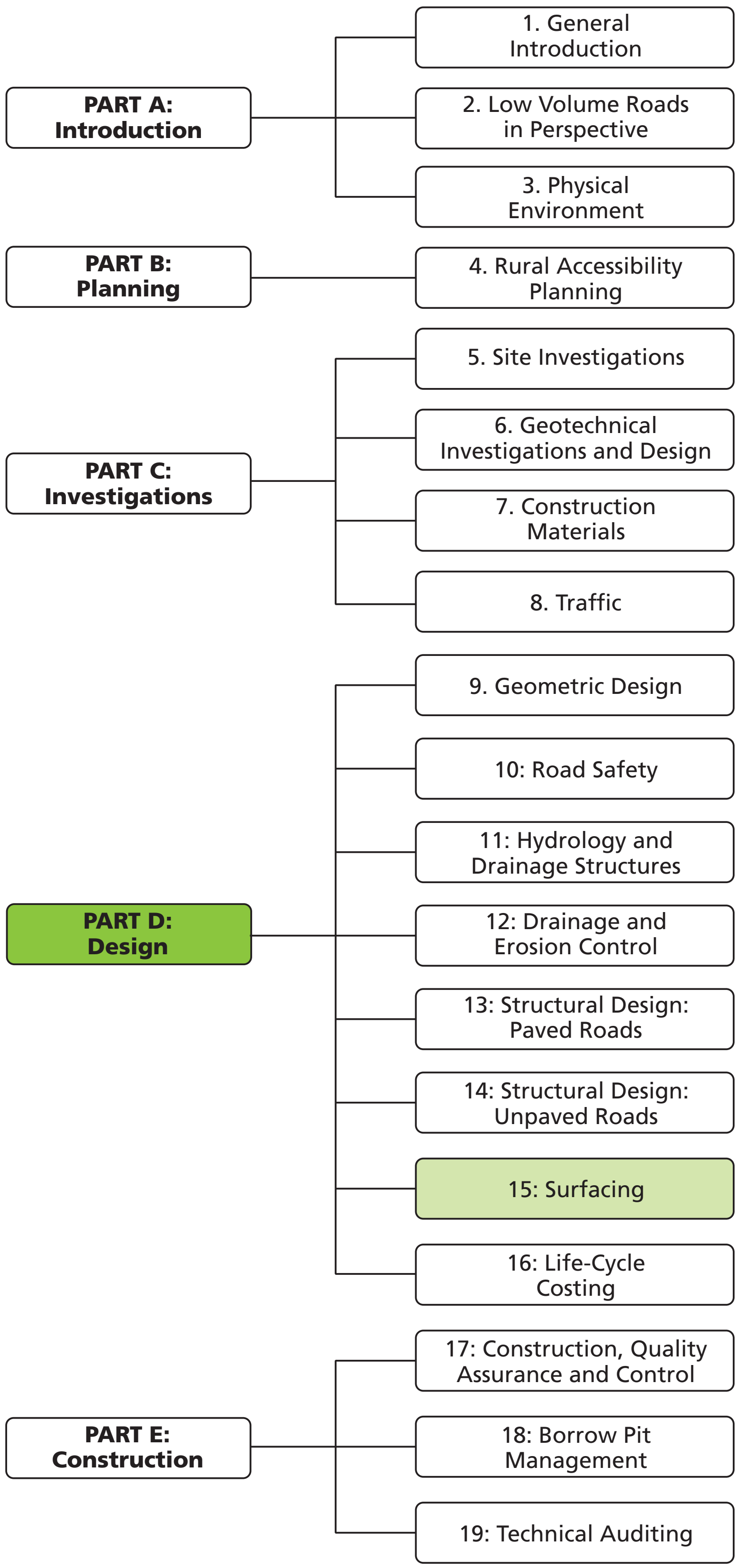
Low Volume Roads Manual



CONTENTS

LIST OF FIGURES

15.1 INTRODUCTION

15.1.1 Background

The surfacing of any road plays a critical role in its long-term performance. It prevents gravel loss, eliminates dust, improves skid resistance and reduces water ingress into the pavement. The latter attribute is especially important for LVR where moisture sensitive materials are often used.

There are a large number of surfacing options, both bituminous and non-bituminous, that are available for use on LVRs. They offer a range of attributes which need to be matched to such factors as expected traffic levels and loading, locally available materials and skills, construction and maintenance regimes, road safety concerns, and the environment. Careful consideration should therefore be given to all of these factors in order to make a judicious choice of surfacing to provide satisfactory performance and minimize life cycle costs.

15.1.2 Purpose and Scope

The main purpose of this chapter is to provide a broad overview of:

- The various types of surfacings that are potentially suitable for use on LVRs.
- The performance characteristics and typical service lives of the various types of surfacings.
- The factors that affect the choice of surfacings.
- The outline design of both bituminous and non-bituminous surfacings.

Thick bituminous surfacings (> 30 mm), due to their relatively high cost, are generally not appropriate for use on LVRs and are not considered in this chapter.

15.2 BITUMINOUS SURFACINGS

15.2.1 General

The term "bituminous surfacings" applies to a wide variety of different types of road surfacings all of which are generally comprised of an admixture of varying proportions of sand, aggregate and bitumen. Such surfacings may be produced in a variety of forms depending on the particular functional and serviceability requirements – single/multiple, thin/thick, flexible/rigid, machine laid/plant processed, etc. Some types, e.g. surface treatments and thin asphalt concrete (<30 mm), do not add any structural strength to the pavement, whilst others, e.g. thick asphalt concrete (> 30 mm) do provide a structural component to the pavement structure. Ultimately, the type of surfacing chosen should be carefully matched to the specific circumstances.

15.2.2 Main types

Terminology for different surfacing types varies for different countries within the region. For purposes of this manual, Figure 15-1 illustrates the main types of bituminous surfacings that are potentially suitable for use in Tanzania.

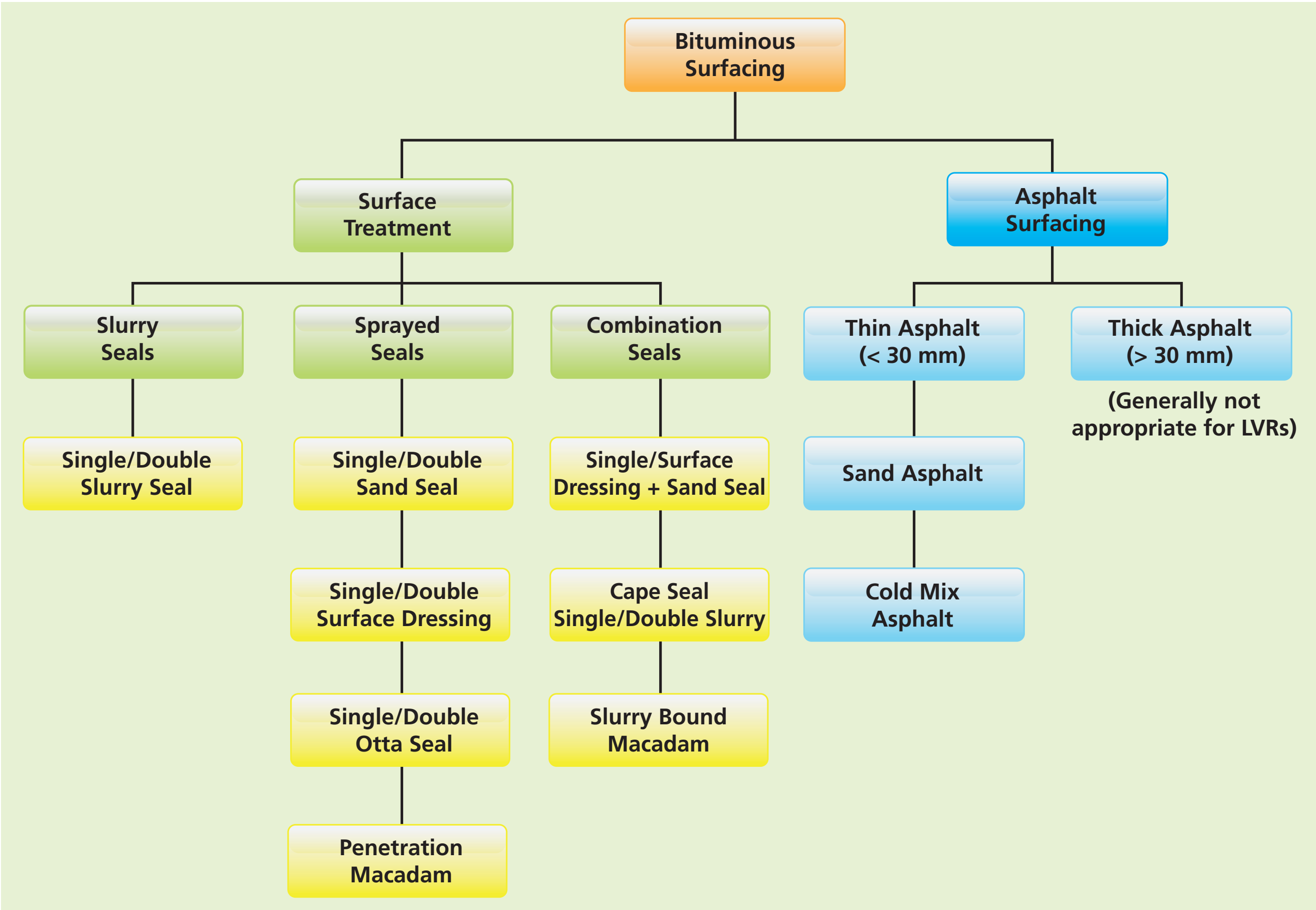


Figure 15-1: Terminology and categorisation of bituminous surfacings

Some of the typical types of bituminous surfacings used on LVRs are shown in Figure 15-2.

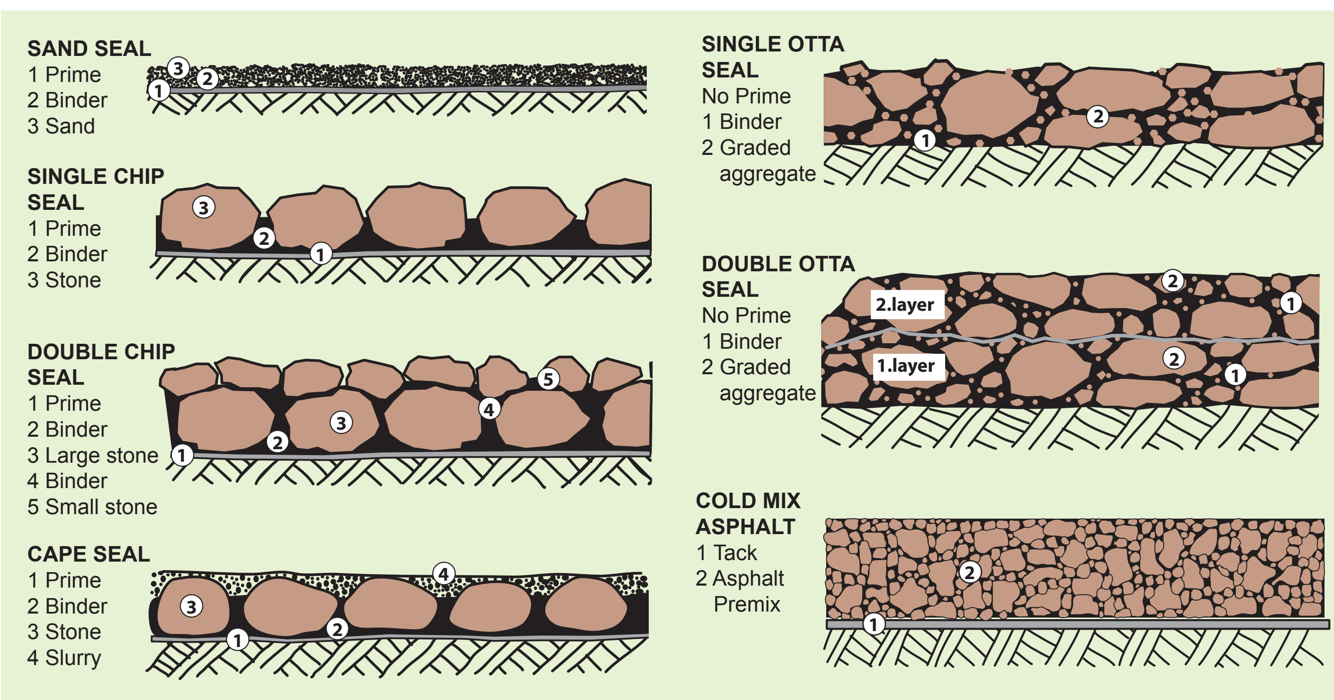


Figure 15-2: Common types of bituminous surfacings

15.2.3 Performance Characteristics

The various types of bituminous surfacings may be placed in two categories as regards their mechanism of performance which is illustrated in Figure 15-3.

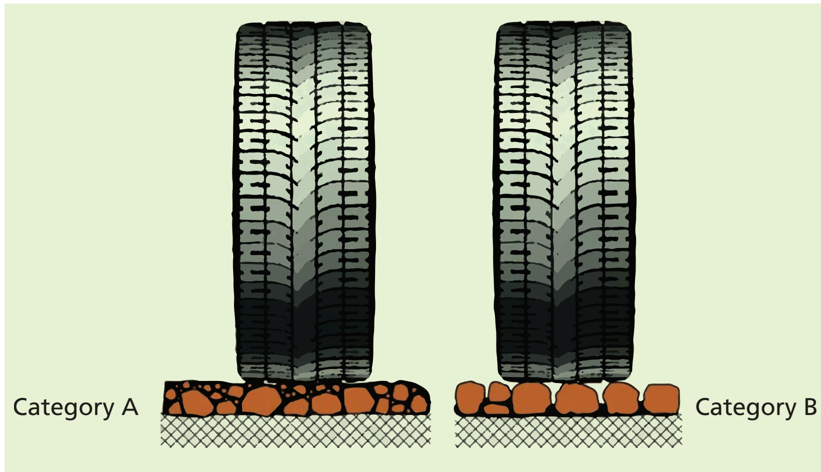


Figure 15-3: Differing mechanisms of performance of bituminous surfacings

Category A (e.g. Sand Seal, Otta Seal, Cold Mix Asphalt): These seal types, like hot mix asphalt, rely to varying extent on a combination of mechanical particle interlock and the binding effect of bitumen for their strength. Early trafficking and/or heavy rolling are necessary to develop the relatively thick bitumen film coating around the particles. Under trafficking the seal acts as a stress-dispersing mat comprising the bitumen/aggregate mixture.

Category B (e.g. Surface Dressing): These seal types rely on the binder to "glue" the aggregate particles to the primed base course. Where shoulder to shoulder contact between the stones occurs, some mechanical interlock is mobilised. Under trafficking, the aggregate is in direct contact with the tyre and requires relatively high resistanc to crushing and abrasion to disperse the stress without distress. Should the bitumen/aggregate bond be broken by traffic or should there be poor aggregate/binder adhesion, insufficient material strength, or oxidation and embrittlement of the binder, then "whip-off" of the aggregate is almost inevitable.

Table 15-1. indicates the relative difference in required properties between the various surfacing types.

Table 15-1: Differences in required properties of main types of bituminous surfacings

Parameter	Category A	Category B
Aggregate quality	Less stringent requirements in terms of strength, grading, particle shape, binder adhesion, dust content, etc. Allows extensive use to be made of natural gravels.	More stringent requirements in terms of strength, grading, particle shape, binder adhesion, dust content, etc. Allows limited use to be made of locally occurring natural gravel.
Binder type	Relatively soft (low viscosity) binders or emulsion are required.	Relatively hard (high viscosity) binders are normally used.
Design	Empirical approach. Relies on guideline and trial design on site. Amenable to design changes during construction.	Rational approach. Relies on confirmatory trial on site. Not easily amenable to design changes during construction.
Construction	Less sensitive to standards of workmanship. Labour based approaches relatively easy to adopt if desired.	Sensitive to standards of workmanship. Labour based approaches less easy to adopt if desired.
Durability of seal	Enhanced durability due to use of relatively soft binders and, in the case of the Otta Seal, a dense seal matrix.	Reduced durability due to use of relatively hard binders and open seal matrix.

15.2.4 Typical Service Life

The life of a surface treatment depends on a wide range of factors such as the quality of the design, climate, pavement strength, binder durability, standard of workmanship, adequacy of maintenance etc. As a result, the service life of the surfacing can vary widely. In general, however, thin seals, which are typically used as temporary or holding measures in a phased surfacing strategy, have much shorter service lives (generally < 10 years) than double/combination seals (generally > 10 years).

Table 15-2 provides a broad indication of the relative service lives of different types of surface treatments which, together with other factors, could assist in the selection of the type of surfacing in the context of a life- cycle cost analysis. For further information refer to *Chapter 16 – Life-Cycle Costing*.

Table 15-2: Typical lives of bituminous surfacings

Type of surfacing	Typical service life (years
(a) Thin seal/phased strategy	
● Single Sand Seal	2 - 3
● Double Sand Seal	3 - 6
● Single Slurry Seal	3 - 5
● Single Surface Dressing	5 - 7
(b) Double/combination seal strategy	
● Single Surface Dressing + Sand Seal	6 - 8
● Double Surface Dressing	8 – 10
● Cold Mix Asphalt	8 - 10
● Single Otta Seal	8 - 10
● Single Otta Seal + Sand Seal	10 – 12
● Cape Seal (13 mm + single slurry)	10 - 12
● Cape Seal (19 mm + double slurry)	12 - 15
● Double Otta Seal	15 – 18
● Penetration Macadam	8 – 12
● Slurry Bound Macadam	8 – 10
● Sand Asphalt	8 - 10
● Thin Asphalt < 30 mm	8 - 10

15.2.5 General Characteristics

The general characteristics of the different types of bituminous surfacings are summarized in Table 15-3.

Table 15-3: General characteristics of bituminous surfacings

Surfacing	Characteristics
Sand Seal	– Empirical design. – Consists of a film of binder (cutback bitumen or emulsion) followed by a graded natural sand or fine sand-sized machine or hand-broken aggregate (max. size typically 6 – 7 mm) which must then be compacted. – Single sand seals are not very durable but performance can be improved with the application of a second seal after 6-12 months, depending on traffic. Will then last for another 6-7 years before another seal can be added. – Especially useful if good aggregate is hard to find. – Very suitable for labour-based construction, especially where emulsions are used, and requires simple construction plant. – Need to be broomed back into the "worn" wheel tracks. There is an extended curing period (typically 8 – 12 weeks) between the first and second seal applications to ensure complete loss of volatiles and thus prevent bleeding. During this period, the sand may need to be broomed back into the "worn" wheel tracks.

Surfacing	Characteristics
Penetration Macadam	– Rational design with both simplified and detailed approaches. – Consists of a mixture of fine aggregates, Portland cement, emulsion binder and additional water to produce a thick creamy consistency which is spread to a thickness of 5-15 mm. – Can be used on LVRs carrying only light traffic. More typically used for re-texturing surface dressings prior to resealing or for constructing Cape seals. – Very suitable for labour-based construction using relatively simple construction plant (concrete mixer) to mix the slurry. – Thin slurry (5 mm) is not very durable; performance can be improved with the application of a thicker (15 mm) slurry.
Otta Seal	– Empirical design. – Consists of a low viscosity binder (e.g. cutback bitumen, MC 3000 or 150/200 penetration grade bitumen) followed by a layer of graded aggregate (crushed or screened) with a maximum size of up to 19 mm, (normally 16 mm) – Thickness about 16 mm for a single layer. – Due to the fines in the aggregate, requires extensive rolling to ensure that the binder is flushed to the surface. – May be constructed in a single layer or, for improved durability, with a sand seal over a single layer or in a double layer. – Fairly suitable for labour-based construction but requires relatively complex construction plant (bitumen distributor + binder heating facilities).and extended aftercare (replacement of aggregate and rolling).
Penetration Macadam	– Empirical design – Constructed by first applying a layer of rolled coarse (e.g. 40/60 mm aggregate) followed by the application of emulsion or penetration grade binder. Next, the surface voids in the coarse aggregate layer are filled with finer aggregate (e.g. 10/20 mm aggregate) to lock in the coarse aggregate followed by an additional application of emulsion binder which is then covered with fine aggregate (e.g. 5/10 mm) and rolled. – Very suitable for labour-based construction as aggregate and emulsion can be laid by hand. – Produces a stable interlocking, robust layer after compaction but the cost is relatively high for LVRs due to the very high rate of application of bitumen (7-9 kg/m^2). Not considered appropriate for use on LVRs in Tanzania.
Single Surface Dressing + Sand Seal	– Partly rational (surface dressing) and partly empirical design. – Consists of a single 13 mm or 9.5 mm surface dressing followed by a single layer of Sand Seal (river sand or crusher dust). – The primary purpose of the sand seal is to fill the voids between the chips to produce a tightly bound, close-textured surfacing. – Fairly suitable for labour-based construction and, when emulsion is used, requires relatively simple construction plant. – More durable than a Single Surface Dressing.
Cape Seal	– Partly rational (surface dressing) and partly empirical (slurry seal) design. – Consists of a single 19 mm or 13 mm surface dressing followed by two layers or one layer respectively of slurry. The primary purpose of the slurry is to fill the voids between the chips to produce a tightly bound, dense surfacing. – Fairly suitable for labour-based construction and, when emulsion is used with the surface dressing; can be constructed with relatively simple plant. – Produces a very durable surfacing, particularly with the 19 mm aggregate + two slurry applications (life of 12–15 years).
Slurry Bound Macadam	– Empirical design. – Consists of a layer (about 20- 30 mm thick) of single size aggregate (typically 13 mm or19 mm), static roller compacted and grouted with bitumen emulsion slurry before final compaction with light pedestrian roller (vibrating at low amplitude and high frequency). A fine slurry is normally applied after curing of the penetration slurry. – Acts simultaneously as a base and surfacing layer. – Very suitable for labour-based construction as aggregate and emulsion can be laid by hand. – Produces a stable interlocking, robust layer after compaction but the performance is sensitive to single sized aggregate and all voids being filled with slurry. The cost is relatively high for LVRs due to the high rate of application of bitumen and may not be appropriate for use on LVRs in Tanzania.

Surfacing	Characteristics
Sand Asphalt	– Empirical design. – Consists of 30-50 mm thick admixture of sand and bitumen at high temperature (130 – 140 degrees Celsius) which is spread and rolled when the temperature has reduced to 80 degrees Celsius. – Performance not yet proven, so not considered for use on LVRs in Tanzania.
Cold Mix Asphalt	– Empirical design. – Consists of an admixture of graded gravel (similar to an Otta seal) and a stable, slow-breaking emulsion which is mixed by hand or in a concrete mixer. After mixing the material is spread on a primed road base and rolled. Thickness about 15 mm. – Very suitable for labour-based construction; requires very simple construction plant; reduces the potential hazard of working with hot bitumen; does not require the use of a relatively expensive bitumen distributor.
Thin Asphalt < 30 mm	– Rational design. – Consist normally of 4.74 mm crushed aggregate mixed in asphalt hot mix plant and placed by a paver.

15.2.6 Design

General: The design of a particular type of surface treatment is usually project specific and related to such factors as traffic volume, climatic conditions, available type and quality of materials. Various methods have been developed by various authorities for the design of surface treatments. Thus, the approach to their design as described in this section is generic, with the objective of presenting typical binder and aggregate application rates for planning or tendering purposes only. Where applicable, reference has been made to the source document for the design of the particular surface treatment which should be consulted for detailed design purposes.

Prime coat: This is used to provide an effective bond between the surface treatment and the existing road surface or underlying pavement layer and is essential for good performance of a bituminous surfacing. This generally requires that the non-bituminous road surface or base layer must be primed with an appropriate grade of bitumen before the start of construction of the surface treatment. However, for a double Otta seal and Penetration Macadam a prime coat is normally not required.

Typical primes are:

Bitumen primes: Low viscosity, medium curing cutback bitumens such as MC-30, MC-70, or in rare circumstances, MC-250, can be used for prime coats.

- **Emulsion primes:** Bitumen emulsion primes are not suitable for priming stabilized bases as they tend to form a skin on the road surface and to not penetrate this surface.
- **Tar primes:** Low-viscosity tar primes such as 3/12 EVT are suitable for priming road surfaces but are no longer in common use because of their carcinogenic properties which are potentially harmful to humans and the environment.

The choice of prime depends principally on the texture and density of the surface being primed. Low viscosity primes are necessary for dense cement or lime stabilized surfaces while higher viscosity primes are used for untreated, coarse-textured surfaces. Emulsion primes are not recommended for saline base courses.

The grade of prime and the nominal application rates to be used on the various types of road bases are presented in Table 15-4.

Table 15-4: Typical prime application rates in relation to road base type

Pavement surface	Prime	
	Grade	Rate of application (l/m²)
Tightly bonded (light primer)	MC-30	0.7 – 0.8
Medium porosity (medium primer)	MC-30 / MC-70	0.8 – 0.9
Porous (heavy primer)	MC-30 – MC 70	0.9 – 1.1

Adhesion agent: The successful performance of a bituminous seal depends not only upon the strength of the two main constituents – the binder and the aggregate – but also upon the attainment of adhesion between these materials - a condition that is sometimes not achieved in practice.

The main function of an adhesion agent is to facilitate the attainment of a strong and continuing bond between the binder and the aggregate. However, if the aggregate is dusty, the adhesion agent will be ineffective and in such a case the aggregate should be pre-coated.

Pre-Coating Materials: Surfacing aggregates are often contaminated with dust on construction sites and, in that condition, the dust tends to prevent actual contact between the aggregate and the binder. This prevents or retards the setting action of the binder which results in poor adhesion between the constituents. This problem can be overcome by sprinkling the aggregate with water or, alternatively, by using an appropriate pre-coating material which increases the ability of the binder to wet the aggregate and improve adhesion between binder and aggregate.

Sand Seal

Design: There are no formal methods for the design of Sand Seals with the binder and aggregate application rates being based on local experience.

Typical constituents for sand seals are:

Binder: The following grades of binder are typically used:

- MC-800 cut-back bitumen.
- MC-3000 cut-back bitumen.
- Spray-grade emulsion (65% or 70% of net bitumen).

Aggregate: The grading of the sand may vary, but the conditions of Table 15-5 must be met. However, in the case of a relatively high proportion of motorbikes, a coarser sand grit may be considered (max. 8 mm) in order to improve the skid resistance when wet.

Table 15-5: Grading of sand for use in Sand Seal

Sieve size (mm)	Grading, (% passing)	
	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

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

For planning or tender purposes, typical binder and aggregate application rates for Sand Seals are shown in Table 15-6.

Table 15-6 shows typical binder and aggregate application rates for Sand Seals.

Table 15-6: Binder and aggregate application rates for Sand Seals

Application	Hot spray rates of MC3000 cut back bitumen (l/m²)	Aggregate application rate (m³/m²)
Double Sand Seal used as a permanent seal	1.2 per layer	0.01 – 0.012 per layer
Single Sand Seal used as a cover over an Otta Seal or Surface Dressing	0.8 – 1.0	0.01 – 0.012
Single Sand Seal used as a maintenance remedy on an existing road	0.6 – 1.0	0.01 – 0.012

Slurry Seal

Design: The detailed design of a Slurry Seal surfacing is presented in the Sabita Manual 28 – Best Practice for the Design and Construction of Slurry Seals, June 2010. The design is based on semi-empirical methods or experience with the exact proportions of the mix being determined by trial mixes.

Binder: The binder typically used is an anionic or cationic emulsion or quick setting cationic emulsion produced from 80/100 pen. grade base bitumen. Stable grade anionic and cationic emulsions are used when the slurry mixes are being laid by hand. If the crusher dust used in the slurry comes from acidic rocks a cationic emulsion is preferred.

Aggregate: The aggregate grading for conventional slurry mixes is presented in Table 15-7.

Table 15-7: Aggregate grading for conventional slurry mixes

Sieve size (mm)	Grading, (% passing)	
	Fine type	Coarse type
10	100	100
5	85-100	85-100
2 1.18	85-100 20-90	50-90 32-70
0.425 0.150	32-60 10-27	20-44 7-20
0.075	4-12	2-8

Laboratory test CML 1.7 is rederred to.
Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

For planning or tender purposes, the typical composition of the slurry may be based on the mass proportions indicated in Table 15-8.

Table 15-8: Nominal Slurry Seal mix components

Material	Proportion (Parts)
Fine aggregate (dry)	100
Cement (or lime)	1.0 – 1.5
60% stable grade emulsion	20
Water	+ / - 15

Surface Dressing

Design: Design methods for both single and double Surface Dressings are presented in Overseas Road Note 3 (2nd edition, 2000): *A guide to surface dressing in tropical and sub-tropical countries.* The design is based on the concept of partially filling the voids in the covering aggregate. This is controlled by the natural orientation of the chippings as they lie on the road surface with their 'least dimension' in the vertical direction. Thus, the Average Least Dimension (ALD) of the chippings is the parameter that mainly determines how much bitumen is required. Corrections to the spray rate need to be subsequently carried out to take account of site conditions as described in the guide. These conditions include traffic level, hardness of existing road surface (controlling embedment of the chippings), shape and condition of chippings, downhill or uphill road gradient, grade of bitumen, and climate.

Typical constituents for Surface Dressings are:

Binder

The bituminous binder can consist of any of the following:

- 80/100 or 150/200 penetration grade bitumen.
- MC 3000 grade cutback bitumen.
- Spray grade anionic (60) or cationic (65 or 70).
- Modified binders (polymer modified and bitumen rubber).

Aggregate: The aggregate for a Surface Dressing shall be durable and free from organic matter or any other contamination. Typical aggregate grading requirements for Surface Dressings are given in Table 15-9.

Table 15-9: Aggregate grading requirements for bituminous Surface Dressings

Material property Sieve size (mm)	Nominal aggregate size			
	20 mm	14 mm	10 mm	7 mm
	Grading, (% passing)			
25	100			
20	85-100	100		
14	0-30	85-100		
10	0-5		85-100	100
6.3	-	0-5	0-30	0-5
5	-	-	0-5	< 1.5
2.36	-	-	-	0-5
0.425	< 0.5	< 1.0	< 1.0	< 1.5
0.075	< 0.3	< 0.5	< 0.5	< 1.0
Flakiness Index	max 20	max 25		max 30
TFV_{dry}	Min 150		Min 120	
$TFV_{soaked\ 24\ hrs}$	min 75% of the corresponding TFV_{dry}			

CML tests 2.4 and 2.7 are referred to.
Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

For planning purposes, typical binder and aggregate application rates for single bituminous Surface Dressings are given in Table 15-10.

Table 15-10: Specifications for total binder application rates for Double Surface Dressing

Traffic (vehicles/lane/day)	Total binder application rate (l/m²)	
	20/10 mm	14/10 mm
<25	2.60	2.40
25-75	2.50	2.30
75-150	2.35	2.20

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

These specifications apply in situations where the surfacing stone meets conventional specifications. In situations where the materials are marginal the following design procedure should be mandatory.

Design procedure:

Laboratory tests

1. Sample surfacing stone from the selected quarry or quarries and carry out laboratory tests. Compare the laboratory tests with the specifications given in Table 15-19.
2. Calculate application rates.

a. Determine the Average Least Dimension (ALD) from the chart (Figure 15-4) using the value of the measured flakiness index and the nominal size value obtained from the sieve analysis. The intercept of ALD line and a straight line drawn from the aggregate size scale to the flakiness scale should be read as the ALD value of the surfacing stone. Alternatively place a random sample of 100 stones on a flat table. The ALD is their vertical height measure from their most stable face. Measure this for each stone with callipers and take the average.

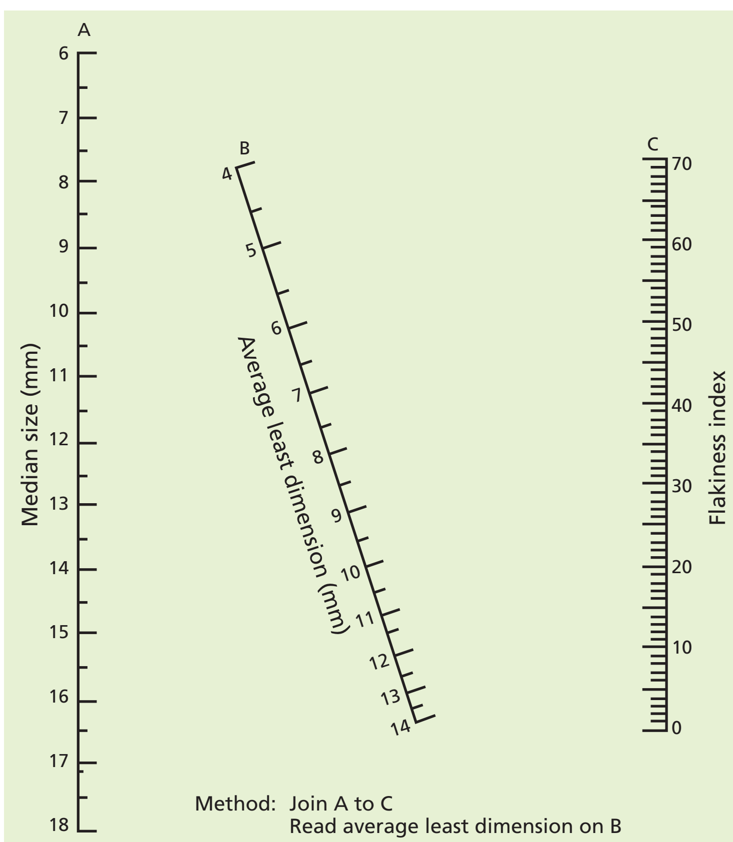


Figure 15-4: Determination of Average Least Dimension (ALD)

b. Determine the weighting factor, the sum of the individual factors given in Table 15-11.

c. Determine the binder application rate using the following formula (equation 15-1)

$$R = 0.625 + (0.023xF) + [0.0375+ (0.0011 \times F)] \times ALD \qquad \text{Equation 15-1}$$

Where: F = Overall weighting factor

ALD = Average least dimension

R = Rate of binder application in kg/m^2

This formula is correct for MC3000 grade bitumen. Adjustment factors for different cut-back bitumens, penetration grade bitumens and emulsions are required (see TRL's ORN 3 for a more detailed description).

d. Calculate the application rate for the chippings or surfacing stone.

A rough estimate of the application rate for the chippings can be obtained using the following formula assuming the density of loose aggregate to be approximately 1.35 kg/litre.

Chipping application rate = 1.364 x ALD

The weighting factor, F, is obtained from Table 15-11.

Table 15-11: Determination of weighting factor (F)

Description	Factor (F)
Traffic vehicles/lane/day	
Very light (0 – 20)	+8
Light (20 – 100)	+4
Medium light (100 – 250)	+2
Medium (250 – 500)	0
Existing surface	
Untreated or primed	+6
Very lean bituminous	+4
Lean bituminous	0
Average bituminous	-1
Very rich bituminous	-3
Climatic conditions	
Wet and cold	+2
Tropical (wet and hot)	+1
Temperate	0
Semi-arid (hot and dry)	-1
Arid (very hot and very dry)	-2
Type of chippings	
Round/dusty	+2
Cubical	0
Flaky	-2
Pre-coated	-2

Source: TRL, UK. (2000) A Guide to Surface Dressing in Tropical and Sub-tropical Countries, ORN 3.

Conversions from hot spray rates in volume (litres) to tonnes for payment purposes must be made for the bitumen density at a spraying temperature of 180°C. For planning purposes, a hot density of 0.90 kg/l should be used until reliable data for the particular bitumen is available.

Shoulders and steep grades: The design of bituminous surfacings for shoulders or steep grades (typically > 5%) follows, in most respects, the same general principles as that for the road carriageway. However, because of the much reduced trafficking of the shoulders, and the tendency for the surfacing to dry out more quickly than on the carriageway, higher bitumen spray rates are required, typically of the order of + 10% of that used on the carriageway. In contrast, because of the slower moving traffic on steep grades, lower bitumen application rates are required, typically of the order of – 10% of that used on flat grades.

Otta Seal

Design: The design of an Otta Seal relies on an empirical approach in terms of the selection of both an appropriate type of binder and an aggregate application rate. Full details of the design methods are given in Ministry of Works, Tanzania (1999) Pavement and Materials Design Manual and the Ministry of Transport and Communications, Botswana, *Guideline No. 1: The Design, Construction and Maintenance of Otta Seals (1999) and the* Norwegian Public Roads Administration, Publication No.93 - *A Guide to The use of Otta Seals (1999).*

Binder: The choice of binder in relation to traffic and aggregate grading is given in Table 15-12.

Table 15-12: Choice of Otta Seal binder in relation to traffic and grading

AADT at time of construction	Type of bitumen		
	Open Grading	Medium Grading	Dense Grading
>100	150/200 pen grade	150/200 pen grade in cold weather	MC3000 MC800 in cold weather
<100	150/200 pen grade	MC 3000	MC 800

Shaded cells = preferred grading in relation to traffic.

Table 15-13 gives the recommended hot spray rates for primed base courses.

Table 15-13: Nominal binder application rates for Otta Seal

Type of Otta Seal	Open (l/m²)	Medium (l/m²)	Dense (l/m²)	
			AADT < 100	AADT > 100
Double 1st layer 2nd layer	 1.7 1.6	 1.8 1.6	 1.8 2.0	 1.7 1.9
Single with sand cover 1st layer Fine sand Crusher dust/Coarse river sand	 1.7 0.8 0.9	 1.8 0.7 0.8	 2.0 	 1.9 0.9 0.8
Single	1.8	1.9	2.1	2.0
Maintenance reseal	1.7	1.8	2.0	1.8

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

The following points should be noted with regard to the binder application rates:

- Hot spray rates lower than 1.6 l/m² should not be allowed.
- Binder for the Sand Seal cover shall be MC 3000 for crusher dust or coarse river sand and MC 800 for fine sand.
- Where the aggregate has a water absorbency of more than 2%, the hot spray rate should be increased by 0.3 l/m².

Aggregate: Both crushed and uncrushed material and a mixture of both can be used. The grading of the aggregate should fall within, and should desirably be parallel to, the grading envelope. Although the envelope is relatively wide, the preferred maximum size is 16 mm (19 mm can be tolerated for the Double Otta Seal) and the maximum amount of fines (material passing the 0.075 mm sieve) should preferably not exceed 10%. The recommended grading in relation to traffic level is indicated in Table 15-14.

Table 15-14: Alternative Otta Seal grading requierments

AASHTO Sieve (mm)	Preferred Open Grading AADT < 100	Preferred Medium Grading AADT >100	Preferred Medium Grading AADT >100
	% passing	% passing	% passing
20	100	100	100
14	60 - 82	68 - 94	84 - 100
10	36 – 58	44 - 73	70 - 98
5	10 - 30	19 - 42	44 - 70
2	0 - 8	3 - 18	20 - 44
1.18	0 - 5	1 - 14	15 - 38
0.425	0 - 2	0 - 6	7 - 25
0.075	0 - 1	0 - 2	3 – 10

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

The aggregate shall meet the specification requirements shown in Table 15-15.

Table 15-15: Specifications for Otta Seal aggregate

Test	Requirement	
	< 100 vpd	> 100 vpd
10% FACT (kN)	Min.90	Min. 110
Wet/dry 10% FACT ratio	Min. 0.60	Min. 0.75
Water absorption (WA) (%)	Max. 2.0 In the case higher (WA) increase hot spray rate with 0,3 l/m²	

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

The aggregate application rate for different gradings is presented in Table 15-16.

Table 15-16: Nominal Otta Seal aggregate application rates

Type of Seal	Aggregate Application Rates (m^3/m^2)		
	Open Grading	Medium Grading	Dense Grading
Otta Seals	0,015 – 0,017	0.015 – 0,017	0.018 – 0.022
Sand Cover Seals	0.012 – 0,014		

Source: MOW, Tanzania (1999) Materials and Pavement Design Manual.

The following points should be noted with regard to the aggregate application rates:

- Sufficient amounts of aggregate should be applied to ensure that there is some surplus material during rolling (to prevent aggregate pick-up) and through the initial curing period of the seal.
- Aggregate embedment will normally take about 3 – 6 weeks to be achieved where crushed rock is used, after which any excess aggregate can be swept off. Where natural gravel is used the initial curing period will be considerably longer (typically 6 – 10 weeks).

Rolling: In the construction of Otta Seals the following factors should be given particular attention.

- As a rule of a thumb, it should be assumed that a good result will be achieved when the bitumen can be seen being pressed up in-between the aggregate particles, sparsely distributed in the wheel tracks of the chip spreader or truck wheels.
- Sufficient rolling of the Otta Seal must be achieved. A minimum of two pneumatic tyred rollers with a minimum weight of 12 tonnes or more is essential. Such rollers are particularly well suited to kneading the binder upwards into the aggregate particles, and to apply pressure over the entire area. A minimum of 30 passes with a pneumatic tyred roller is required over the entire surface area, shoulders included, on the day of construction.
- After the initial rolling is completed (on the day of construction) it may be an advantage to apply one pass with a 10-12 tonne static tandem steel roller to improve the embedment of the larger aggregate. During this process any weak aggregate will be broken down and will contribute to the production of a dense matrix texture. Table 15-17 summarises the minimum rolling requirements.

Table 15-17: Minimum rolling requirements for an Otta Seal

Rolling after treatment	Minimum requirements
On the day of construction.	30 passes with pneumatic roller (weight > 12 tonnes) + 1 pass with a static steel roller.
For each of the next three days after construction.	30 passes with pneumatic roller (weight > 12 tonnes).
2-3 weeks after construction.	Sweep off any excess aggregate.

- Commercial traffic should be allowed on the surfaced area immediately following completion of the initial rolling with the pneumatic roller(s). This will assist further in the kneading of the binder/aggregate admixture.
- A maximum speed limit of 40 - 50 km/hour should be enforced immediately after construction and sustained for minimum 3 - 4 weeks when any excess aggregate should be swept off.

Figure 15-5: Extensive rolling of an Otta Seal- essential to achieve a good result

Cape Seal

Design: The design of a Cape Seal is a combination of a Single Surface Dressing plus a Slurry Seal. The design is similar to that for a Surface Dressing and Slurry Seal as described above.

Typical constituents for Cape Seals are:

- **Binder:** A variety of binder types may be used for constructing a Cape Seal.
- **Aggregate:** The same requirements are required as for Surface Dressings and Slurry Seals.

For planning purposes, typical binder and aggregate application rates for Single Surface Dressings are as shown in Table 15-18.

Table 15-18: Binder and aggregate application rates for a Cape Seal

Nominal size of aggregate (mm)	Nominal rates of application for tendering purposes	
	Binder (litres of net cold bitumen per m^2)	Aggregate (m^3/m^2)
14 20	0.6 1.1	0.011 0.0075

Cold Mix Asphalt

Design: The Cold Mix Asphalt (CMA) is, in many respects, similar to an Otta Seal in that a graded aggregate is used. However, the binder used is an emulsion, rather than a hot-applied penetration grade or cut-back binder.

Binder: Three types of emulsion may be used for different circumstances. Table 15-19 describes their use and Table 15-20 shows the grading requirements of the aggregates. Nevertheless, care must be taken to plan the CMA works to prevent washout by rain before the emulsion has set properly.

Table 15-19: Use of emulsion types

Type of emulsion	Description	Purpose
SS60/70 (K3-60/70)	Slow breaking or slow setting emulsion. Allows adequate time for operations.	The mixing and placing should go as one continuous operation to ensure that spreading is done before breaking and setting commences and the mix is still fluid. This will ensure some degree of "self-compaction" of the mix and result in a denser layer.
MS60/70 (K2-60/70	Medium setting emulsion.	This is NOT commonly used but it is suitable in situations where construction is relatively quick.
RS60/70 (K1-60/70)	Rapid setting or quick setting emulsion. This type of emulsion breaks quickly and is difficult to work with.	This emulsion is NOT commonly used in Tanzania but it is useful when construction is carried out in wet weather where rapid breaking prevents wash away of emulsion by rain water or on steep slopes were rapid breaking prevent runoff of the emulsion.

Note: Bitumen emulsions are an area where technological progress is still being made to meet the requirements of pavement engineering. Anionic emulsions were first developed. However, they are less favored than the Cationic emulsions, as Cationic emulsions coat aggregates more efficiently due to their positive load and have therefore better adhesion properties.

"Breaking" of emulsions is the loss of water from the emulsion. Determining whether an emulsion has broken is very easily: the colour turns from brown to black. The "breaking" process is influenced by the environmental conditions in the following order: the incident wind velocity, humidity and temperature. At high bitumen content, the bitumen particles are more likely to come into contact with each other, resulting in an increase in the "breaking" time. For more information refer to the SHELL Bitumen Handbook of 2015.

Aggregate: The grading requirements for the aggregate are shown in Table 15-20.

Table 15-20: Grading requirements for Cold Mix Asphalt

Sieve size (mm)	Percentage Passing sieve		
	Minimum	Recommended	Maximum
14	100	100	100
10	85	96	100
6.3	62	70	78
5	46	52	60
2	28	34	40
1.18	16	21	26
0.425	7	10	13
0.300	5	7.5	10
0.150	2	4	6
0.075	1	2	3

The coarse aggregates (6/10 mm stone) shall meet the minimum aggregate strength specification as shown in table 15-21. Care should be taken to avoid overly dusty aggregates with too high fines content. For every new source of aggregate, trial mixes should be done before surfacing operations start, and if necessary the mix proportions adjusted. It is important that all aggregates are evenly moist before the emulsion is added.

Table 15-21: Minimum aggregate strength requirements for Cold Mix asphalt

Aggregate strength requirements	AADT at time of construction	
	<100	>100
Min Dry 10% FACT	90kN	110kN
Min Wet/Dry strength ratio	0.60	0.75
Flakiness Index	Maximum 30%	

Mix proportions: For tendering purposes the following mix proportions shall be used:

Maximum batch volume: 40 litres.

Aggregates: 6/10 stone: 12 litres.

0/6 crusher dust: 28 litres.

0/2 fine sand: 3 litres (this will depend on the amount of fines (fraction less than 2 mm) in the crusher dust. If the crusher dust is coarser, more fine sand may need to be added).

K3 65 Cationic Emulsion: 6 litres.

Water: 1 litre (when using dry aggregate).

When the mixing is completed, the mix must quickly be placed on the road in between the guide rails and levelled to the top of the guide rails before the emulsion starts to break (turn from brown to black), after which point the mix gets sticky and difficult to spread.

Figure 15-6: Levelling and compaction of Cold Mix Asphalt

Compaction should be undertaken with a double drum steel roller type i.e. Bomag 75 or, similar as shown in figure 15-6. Rolling can commence once the guide rails have been removed and the initial breaking of the asphalt has commenced for the full depth of the layer. This period will be affected by the ruling weather conditions, but can normally be done within ½ hour.

The first compaction is done with the roller in static mode. After 2-3 hours the final compaction is done with the roller in vibrating mode. Rolling is continued until the 20 mm loose layer has been compacted to a thickness of approximately 14-15 mm.

Different compacted layer thicknesses can be achieved by using guide rails of different dimensions:

- 20 mm guide rail gives compacted thickness of +/- 15 mm.
- 25 mm guide rail gives compacted thickness of +/- 19 mm.

The thicker option is preferable particularly if the surface of the base is not perfectly even across the width to prevent thin spots that will eventually be the start of a pothole.

15.2.7 Suitability for Surface treatment on LVRs

The choice of the appropriate surfacing type in a given situation will depend on the relevance or otherwise of a number of factors, including the following:

- Traffic (volume and type).
- Pavement (type – strength and flexural properties).
- Materials (type, quality and availability).
- Environment (climate – temperature, rainfall, etc.).
- Operational characteristics (geometry – gradient, curvature, etc.).
- Safety (skid resistance - surface texture, etc.).
- Construction (techniques and contractor experience).
- Maintenance (capacity and reliability).
- Economic and financial factors (available funding, life cycle costs, etc.).
- Other external factors.

The suitability of various types of surfacings for use on LVRs, in terms of their efficiency and effectiveness in relation to the operational factors outlined above, is summarized in Table 15-22.

Whilst not exhaustive, the factors listed in the table provide a basic format which can be adapted or developed to suit local conditions and subsequently used to assist in making a final choice of surfacing options. These options can then be subjected to a life cycle cost analysis and a final decision made with due regards to prevailing economic factors and be compatible with the overall financial situation.

Table 15-22: Suitability of various surfacings for use on LVRs

SSS-Single Sand Seal, **DSS**-Double Sand Seal, **SLS**-Slurry Seal, **SSD**-Single Surface Dressing, **SOS**-Single Otta Seal, **DSD**-Double Surface Dressing, **DOS**-Double Otta Seal, **CS**-Cape Seal 13/19 mm+Single/Double SLS, **CMA**-Cold Mix Asphalt

Surface attributes	Thin seal/phased strategy				Double/Combination seal strategy							
	SSS	DSS	SLS	SSD	SSD+SS	DSD	SOS	SOS+SS	DOS	CS 13 mm	CS 19 mm	CMA
Ease of design	Very good	Very good	Very good	Reasonable	Reasonable	Reasonable	Good	Very good	Very good	Reasonable	Reasonable	Good
Ease of construction	Very good	Very good	Very good	Reasonable	Reasonable	Reasonable	Good	Very good	Good	Reasonable	Reasonable	Very good
Service life	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Good	Reasonable	Very good	Very good	Good	Very good	Good
Suitability for LBM	Very good	Very good	Very good	Good	Good	Good	Reasonable	Reasonable	Reasonable	Good	Good	Very good
Risk of poor mtce capability	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Reasonable	Very good	Very good	Very good	Very good	Good
High skid resitance	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Good	Very good	Reasonable	Reasonable	Reasonable	Good	Very good	Reasonable
Early road marking	Reasonable	Reasonable	Very good	Very good	Reasonable	Very good	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Very good	Very good
Suitablity for turning action	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Good	Good	Good	Good	Good	Good
Sensitivity to material quality	Reasonable	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Very good	Very good	Poor/not suited	Poor/not suited	Good
Constr. sensitivity to gradient (>8%) *	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Good

Very good Good Reasonable Poor/not suited

15.3 NON-BITUMINOUS SURFACINGS

15.3.1 General

There are a number of situations in which bituminous surfacings are unsuitable for use on LVRs, for example, on very steep grades (>8%), very flexible subgrades or in marshy areas. In such circumstances, some type of more rigid, structural/semi-structural, surfacing would be more appropriate. There are a number of such surfacings which are potentially suitable for use on LVRs as described below.

While these non-bituminous surfaces have the potential to provide all-season accessibility, some have safety concerns. Design engineers should use their professional judgement to weigh up the benefits of improving access with the dis-benefits of increased safety risks. Examples of such safety risks, as well as potential mitigations, are described later in this chapter.

15.3.2 Main Types

The main types of non-bituminous surfacings are summarized in Figure 15-7.

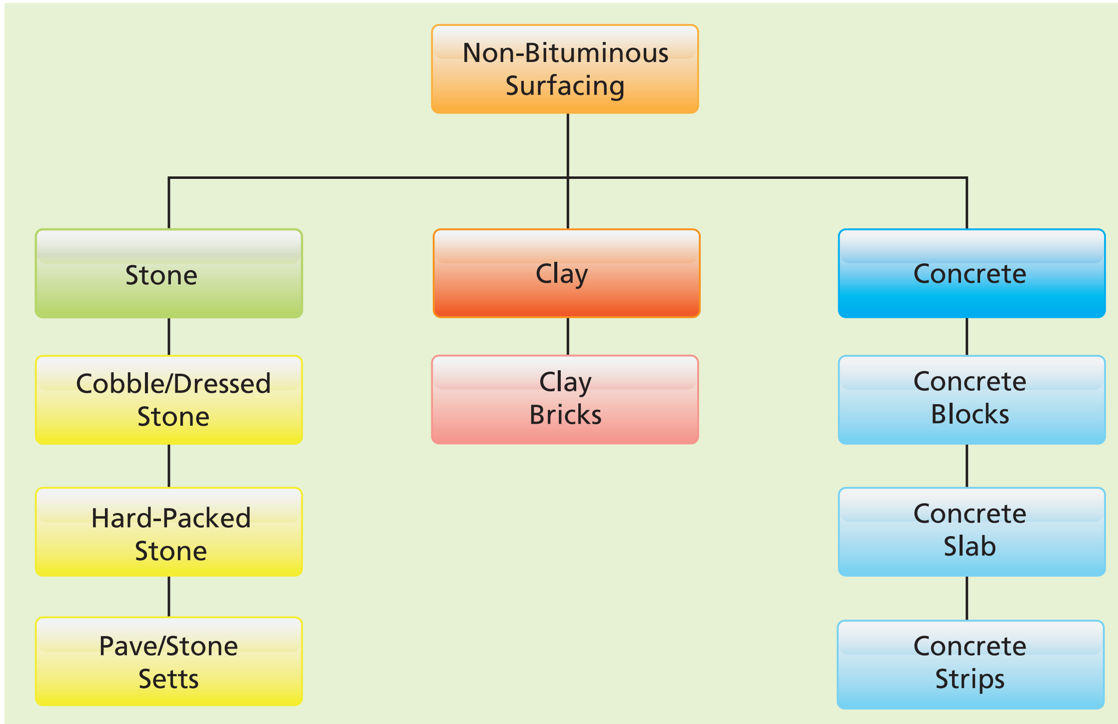


Figure 15-7: Terminology and categorization of non-bituminous surfacings types

Some of the typical types of non-bituminous surfacings are shown in Figure 15-8.

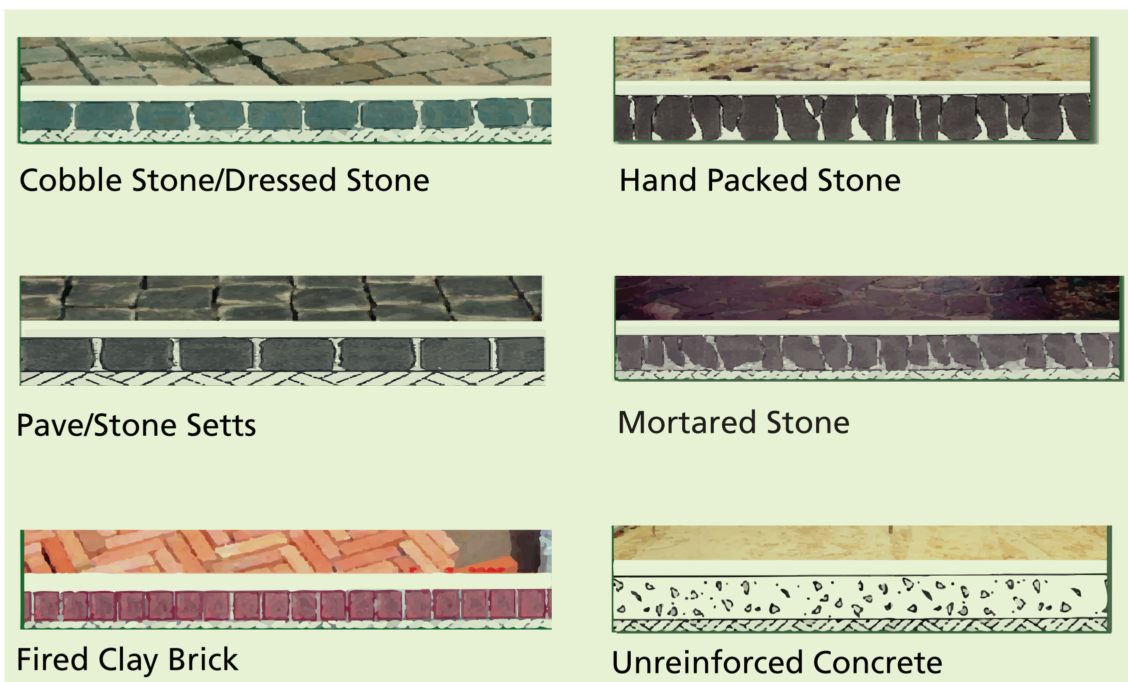


Figure 15-8: Common types of non-bituminous surfacings

15.3.3 Performance Characteristics

The non-bituminous surfacings described above all act simultaneously as a surfacing and base layer and provide a structural component to the pavement because of their thickness and stiffness. They all require the use of a sand bedding layer which also acts as a load transfer layer for the overlying construction. In some cases they act additionally as a drainage medium.

In some circumstances (e.g. on steep slopes in high rainfall areas and in areas with weak subgrades and/or expansive soils) it may be advantageous to use mortared options. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone (or Dressed Stone), and Fired Clay Brick pavements. The construction procedure is largely the same as for the un-mortared options except that cement mortar is used instead of sand for bedding and joint filling.

The behavior of mortared pavements is different to that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on mortared option, although empirical evidence indicates that inter-block cracking may occur. For this reason the option is currently only recommended for the lightest traffic divisions up to TLC2. Reference is made to *Chapter 13, sub-section 13-4, Design of Roads with Non Discrete Surfaces* until further locally relevant evidence is available.

All the non-bituminous surfacings are well suited for use on steep grades in situations where the more traditional types of bituminous surfacings would be ill-suited.

15.3.4 Typical Service Lives

The service life of a non-bituminous surfacing is relatively much longer than for a bituminous surfacing. This is due largely to the superior durability of the surfacing material, mostly natural stone, which is very resistant to the environment. Provided that the foundation support and road drainage are adequate, non-bituminous surfacings require relatively little maintenance and will last almost indefinitely on LVRs as long as they are well constructed and maintained. Thus, for life-cycle costing purposes, the service life of a non-bituminous surfacing can generally be assumed to be at least as long as the design life of a typical LVR pavement.

15.3.5 General Characteristics

The general characteristics of a range of non-bituminous surfacings that may be considered for use in Tanzania are summarized in table 15-23.

Table 15-23: General characteristics of non-bituminous surfacings

Surfacing	Characteristics
Cobble Stone/ Dressed Stone	– Consists of a layer of roughly rectangular dressed stone laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. Individual stones should have at least one face that is fairly smooth, to be the upper or surface face when placed. – Each stone is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregates is brushed into the spaces between the stones and the layer then compacted with a roller (normally by a vibratory tamper or vibratory plate compactors). – Generally 150 mm thick and dressed stones generally 150-200 mm thick. – Joints sometimes mortared.
Hand Packed Stone	– Consists of a layer of large broken stone pieces (typically 150 to 300 mm thick) tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel. – Hand-packing achieves a degree of interlock which should be assumed in the design. – Requires a capping layer when the subgrade is weak and a conventional Subbase of G30 material or stronger. – Normally bedded on a thin layer of sand (SBL) which normally is compacted by a by a vibratory tamper, or vibratory plate compactors. – An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.

Surfacing	Characteristics
Pave/Stone Setts	– Consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. – Individual stones should have at least one face that is fairly smooth to be the upper or surface face when placed. – Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. – Sand or fine aggregate is brushed into the spaces between the stones and the layer is then compacted with a roller (normally by a vibratory tamper or vibratory plate compactors).
Fired Clay Brick	– Consists of a layer of high quality bricks, typically each 10 cm x 20 cm and 7-10 cm thick, laid by hand on a sand bed with joints also filled with a sand and lightly compacted or bedded and jointed with cement mortar. – Kerbs or edge restraints are necessary and can be provided by sand-cement bedded and mortared fired bricks. – Normally laid in herringbone or other approved pattern to enhance load spreading characteristics. (Good practice is to lay the bricks with narrow face up to improve strength). – Un-mortared brick paving is compacted with a plate compactor and jointing sand is topped up if necessary. For mortar-bedded and joint-fired clay brick paving, no compaction is required.
Concrete Blocks	– Consists of pre-cast concrete blocks in moulds typically 10 cm x 20 cm x 7 cm. – Laid by hand, side-by-side on a 3-5 cm sand bed with gaps between blocks filled with fine material and lightly compacted to form a strong, semi-pervious layer with a vibrating plate compactor. – Well suited to labour based construction with modest requirement for skilled workforce.
Un-reinforced Concrete	– Involves casting slabs of 4.0 to 5.0 m in length between formwork with load transfer dowels between them to accommodate thermal expansion. – Provides a strong durable pavement with low maintenance requirements. – More suited to areas with good quality subgrade; in areas of weakness, reinforcement may have to be considered. – Suited to small contractors as concrete can be manufactured using small mixers.
Lightly Reinforced Concrete	– Similar to NRC but with light mesh reinforcement which provides added strength to counteract the wheel loading as traffic moves onto the end slab from the adjacent surfacing. – Well suited in areas of relatively weak subgrade to improvement strength, preventing excessive stress and cracking. – Using mesh reinforcement 6 mm @ 200 mm is a good practice independent from the subgrade condition.
Concrete Strips	– Consists of parallel 0.9 m wide, 3.0 m (max) in length and 0.20 m in thickness, unreinforced concrete strips spaced in a distance from centre to centre shall be 1.55 m so that both sets of vehicle wheels would run on the strips. The end of the strip on adownward slope should be thickened to act as a dowel. – Strips contain transverse concrete strips between the wheel tracks to help stop excessive erosion down the centre of the strips.

15.3.6 Design

General: The design approach for non-bituminous surfacings is similar to that of the more traditional bituminous surfacings, in that design inputs are principally traffic volume, subgrade soil condition and other environmental factors.

A number of design catalogues have been developed based on a combination of experience gained in LVR trials in the Southern African region; existing published design details; engineering judgment and, where relevant, correlation with bituminous LVR design catalogues in terms of equivalent structural number.

Cobble Stone/Dressed Stone

The thickness designs are dealt with in *Chapter 13 – Structural Design for Paved Roads.*

Stone Setts/Pave

Suitable thickness designs for Stone Setts or Pave are similar to Cobble Stone and are dealt with in *Chapter 13 – Structural Design for Paved Roads.*

Fired Clay Bricks

Suitable thickness designs for Fired Clay Bricks are similar to Cobble Stone, Stone Setts or Pave and are dealt with in *Chapter 13 – Structural Design for Paved Roads.*

Hand Packed Stone

Suitable thickness designs for HPS are dealt with in *Chapter 13 – Structural Design for Paved Roads.* Since a degree of interlock is achieved in practice, this has been taken into account in the designs. The structures also require a capping layer when the subgrade is weak and a conventional subbase of G30 material or stronger.

Concrete Blocks

Suitable thickness designs for concrete blocks are dealt with in *Chapter 13 – Structural Design for Paved Roads.* The blocks are required to have an average compressive strength of 25 MPa with a minimum strength for individual blocks of 20 MPa. Full design details may be obtained from *Draft UTG2. Structural design of segmental block pavements for Southern Africa, 1987.*

Figure 15-9: Concrete strip road with dangerous edge drop

Unreinforced/Lightly Reinforced Concrete

Suitable thickness designs for unreinforced concrete pavements are dealt with in *Chapter 13: Structural Design for Paved Roads.*

Concrete Strips

Suitable thickness designs for Concrete Strip roads are similar to that for Unreinforced/lightly reinforced road pavements and are dealt with in *Chapter 13 – Structural Design for Paved Roads.*

The non–bituminous surfacings and their typical thicknesses and strength requirements are given in Table 15-24.

Table 15-24: Common thicknesses and strength requirements for non–bituminous surfacings

Type of surfacing	Typical Thickness (mm)	Crushing strength (MPa)
Cobble Stone/Dressed Stone	150 - 200	20
Hand Packed Stone	150 - 300	20
Pave Stone Setts	100	20
Fired Clay Brick [1)]	70 - 100	20
Concrete Blocks	70	25
Un-Reinforced Concrete (URC)	150 -200	20
Lightly Reinforced Concrete	150 - 200	25
Concrete Strips	150 - 200	25
Mortared Stone	70	20

NOTES: 1) Water absorption: < 16% of their weight of water after 1 hour soaking.

15.3.7 Suitability for Use on LVRs

Non-bituminous surfacings of one type or another are particularly suitable for use on LVRs in the following situations:

- Relatively steep gradients where high tyre traction is required.
- High rainfall areas where slipperiness may be a problem on steep grades.
- Severely stressed areas, such as near market places.
- Oil spillage is likely to occur.
- Junctions with heavy turning vehicles.
- Parking bays with prolonged static loading.
- Waiting lanes for weigh bridges or toll booths.
- Areas with subsurface facilities that requires frequent access.
- Very low maintenance capability is likely.
- Very long service life is required.
- Poor/weak subgrades prevail.
- Natural stone is in plentiful supply.
- Long lines of sight are available.

15.3.8 Safety Risks associated with Non-Bituminous Surfacings

Of the non-bituminous surfaces, the concrete strips pose the greatest safety concerns, especially for motorbikes when they are forced to leave or re-join a strip, for example when encountering a four-wheeled vehicle or when overtaking another motorcycle. In order to mitigate the potential road safety risks associated with the use of concrete strips, the following provisos should be applied:

- Relatively low traffic situations with maximum 30 four-wheeled vehicles and 270 two-wheeled vehicles (motor bikes). Such a situation would result in very few passing occurrences/km/day.
- Relatively short, straight sections of road.
- The width of the road, including shoulders, is sufficient to allow a motorcycle to pass a four-wheeled vehicle safely *(refer to Chapter 9 - Geometric Design).*
- Ditch side slopes (not less than 1V:3H).

- The un-surfaced part of the road is adequately maintained to prevent edge-drops developing, and to keep them clear of vegetation and loose and oversize material.
- The gravel area between the two strips should be maintained to prevent edge-drops developing, and to prevent the transverse concrete strips or chevrons from becoming a hazard.

Other concrete surfaces also pose safety risks. Mitigations include ensuring that:

- Their width should be sufficient to allow a motorcycle to pass a four-wheeled vehicle safely.
- Shoulders should be maintained to prevent edge-drops developing.
- The surface should be scoured (roughened) to provide adequate texture thereby increasing skid resistance, but the scouring should not leave the surface overly rough as this can impart vibrations through the hands of motorcyclists, creating the risk of loss of control.
- In a transition between a concrete surface and an earth or gravel surface, the end of the concrete surface should be bevelled downwards to reduce the risk of erosion creating a drop down from the concrete to the earth.
- Where two different types of surfacing adjoin each other, there is a need to ensure that this point does not occur where it cannot be seen by a motor cyclist, such as at the brow of a hill or on a sharp curve.

In general, the use of non-bituminous surafcings, particularly concrete slab or strip surfaces, requires adequate maintenance to be carried out, in the absence of which, road safety problems are likely to be a serious issue.

Figure 15-10: Example of two different adjoining surfacings with dangerous edge drop

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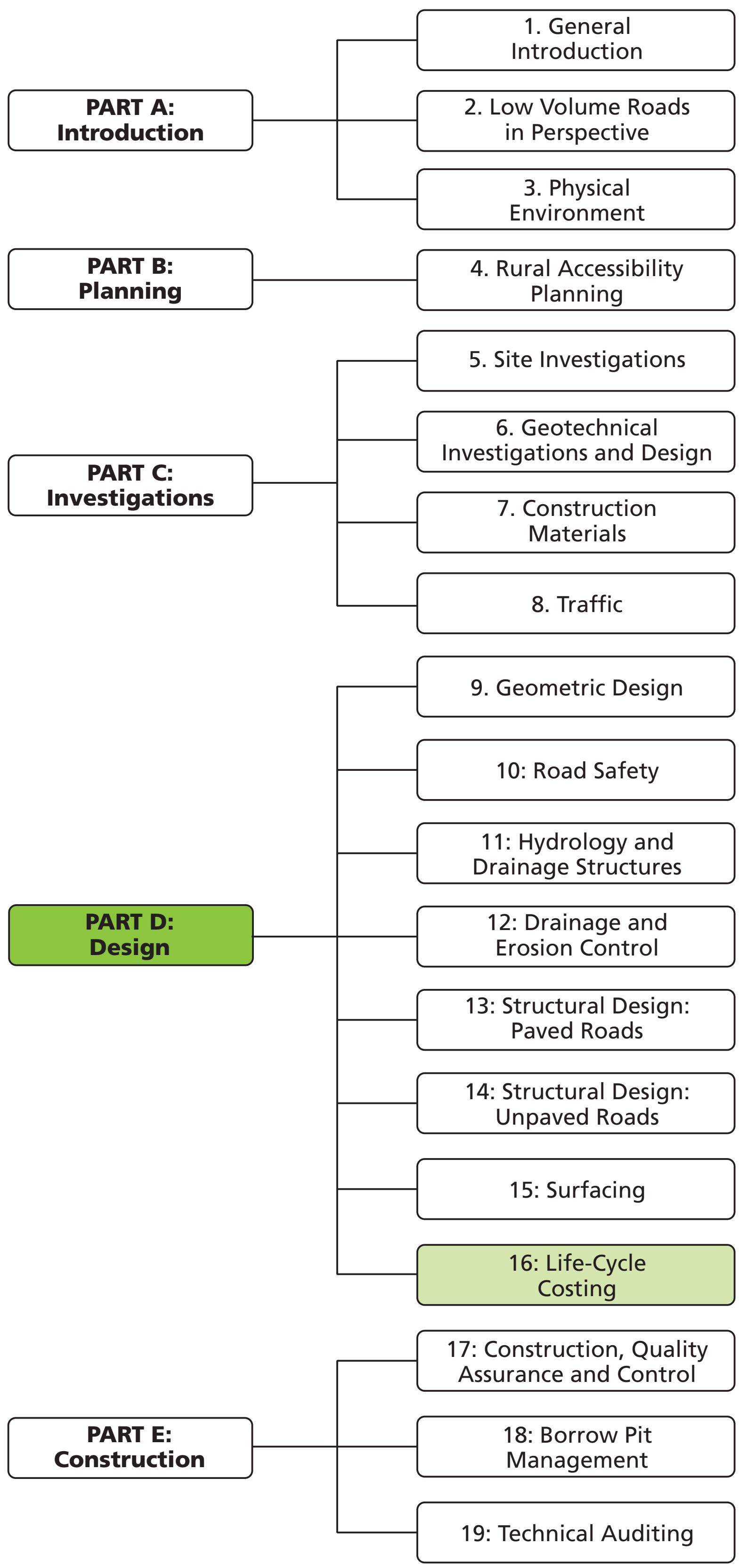

CONTENTS

16.1 INTRODUCTION

16.1.1 Background

There are always a number of potential alternatives available to the designer in the design of new roads or the rehabilitation of existing ones, each capable of providing the required performance. For example, as illustrated in Figure 16-1, for a given analysis period, one alternative might entail the use of a relatively thin, inexpensive pavement which requires multiple strengthening interventions (Alternative B) whilst another alternative might entail the use of a thicker, more costly pavement with fewer interventions (Alternative A).

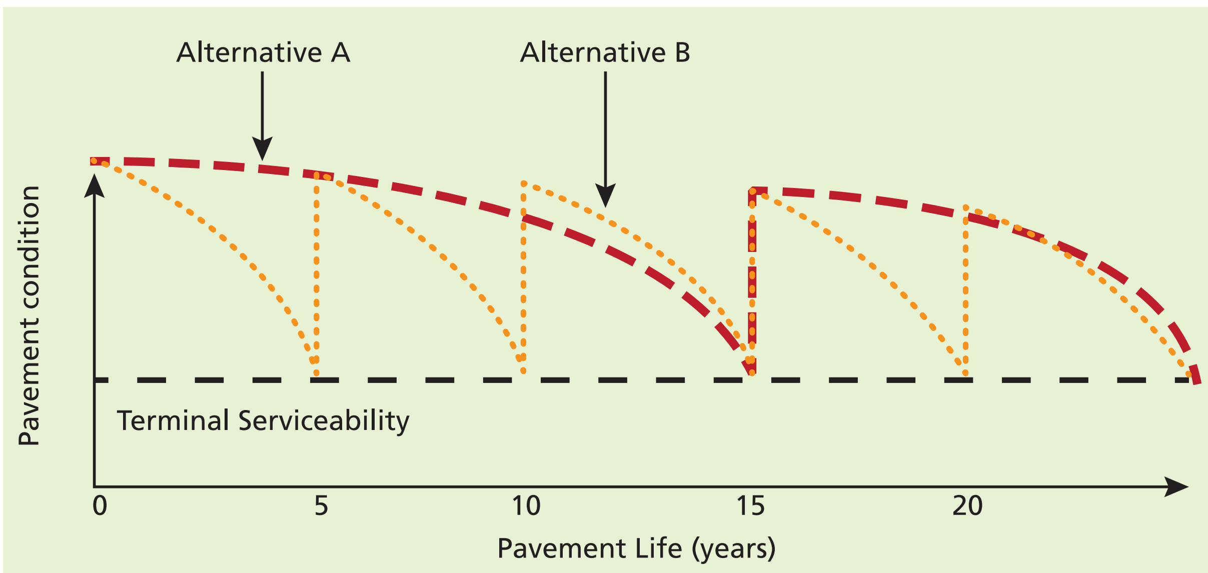

Figure 16-1: Alternative pavement options

In order to make the most effective use of the available resources, the designer is required to find which alternative will serve the needs of road users for a given level of service at the lowest cost over time. Such a task can be achieved through the use of a life-cycle economic evaluation, often referred to as "life-cycle" or "whole-of-life" costing.

16.1.2 Purpose and Scope

The main purpose of this chapter is to outline the procedure to be followed in undertaking a life-cycle cost (LCC) analysis to compare alternative pavement options over their design lives in order to arrive at the most cost-effective solution. The chapter outlines the methods of carrying out an LCC analysis and consider the necessary inputs to the analysis including such factors as construction, maintenance and road user costs, salvage value, discount rate and analysis period.

The focus of the chapter is on LCC analysis of road pavements/surfacings/upgrading. However, the principles of this analysis can also be applied to comparing road projects involving alternative alignments, or alternative maintenance strategies, etc., which are outside the scope of this chapter.

16.2 LIFE-CYCLE COST ANALYSIS

16.2.1 General

In the roads context, a LCC analysis is defined as a process for evaluating the total economic worth of a road project by analysing initial construction/rehabilitation costs and discounted future costs and benefits, such as maintenance, user and reconstruction costs and benefits over the life of the road (or analysis period) of the project. The analysis requires identifying and evaluating the economic consequences of various alternatives over time, primarily according to the criterion of minimum total (life-cycle) costs.

As indicated in Figure 16-2, the principal components of a LCC analysis are the initial investment or construction cost and the future costs of maintaining or rehabilitating the road, as well as the benefits due to savings in user costs over the analysis period selected. An assessment of the residual value of the road is also included so as to incorporate the possible different consequences of construction and maintenance strategies for the pavement/surface options being investigated.

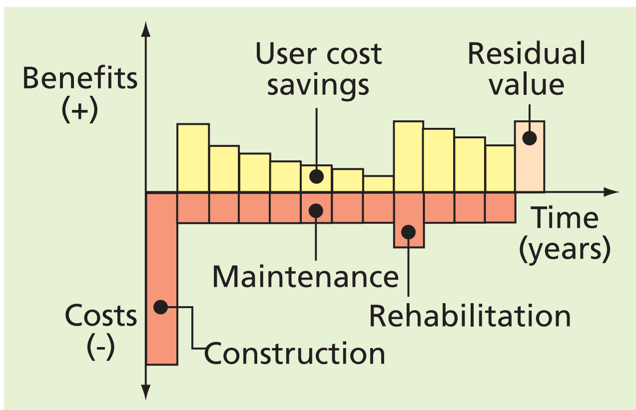

Figure 16-2: Distribution of costs and benefits during the life cycle of a road

16.2.2 Method of Economic Comparison

In the life-cycle analysis process, alternative pavement/surface options are compared by converting all the costs and benefits that may occur at different times throughout the life of each option to their present day values. Such values are obtained using discounted cash flow techniques involving the use of an appropriate discount rate, to determine the Net Present Value (NPV) of the pavement/surface options. Costs and benefits are usually estimated in constant local currency terms to eliminate the effects of inflation. Indirect taxes are usually excluded from costs and benefits.

$$NPV = C + \sum_i M_i (1+r)^{-X_i} - S(1+r)^{-z} \qquad \text{...Equation 16-1}$$

The NPV can be calculated as follows:

Where:

NPV	=	present worth costs
C	=	present cost of initial construction
M_i	=	cost of the i^{th} maintenance and/or rehabilitation measure
r	=	real discount rate
X_i	=	number of years from the present to the i^{th} maintenance and/or rehabilitation measure, within the analysis period
z	=	analysis period
S	=	salvage value of pavement at the end of the analysis periode expressed in terms of present values.

The NPV method is generally preferred over other methods of evaluating projects, such as the Internal Rate of Return (IRR). One of its main advantages is that it can be used to evaluate both independent and mutually exclusive projects whilst the IRR method cannot be relied upon to analyse mutually exclusive projects – this method can lead to conflicts in the ranking of projects. In many cases, the project with the highest IRR may not be the project with the highest NPV. Nonetheless, the IRR, which is defined as the rate of discount which equates the present worth of the costs and benefits streams, may be computed by solving for the discount rate that makes the NPV of a project equal to zero. This may be done graphically or by iteration. On this basis, an independent project would be viable whose IRR is greater that the project cost of capital. However, this value gives no indication of the size of costs or benefits of a project, but acts as a guide to the profitability of the investment. The higher the IRR the better the project.

16.2.3 Components of a LCC analysis

The components of a LCC analysis associated with a particular design alternative are listed below and illustrated in Figure 16-3.

- Analysis period.
- Structural design period.
- Construction/rehabilitation costs.
- Maintenance costs.
- Road user costs.
- Salvage value.
- Discount rate.

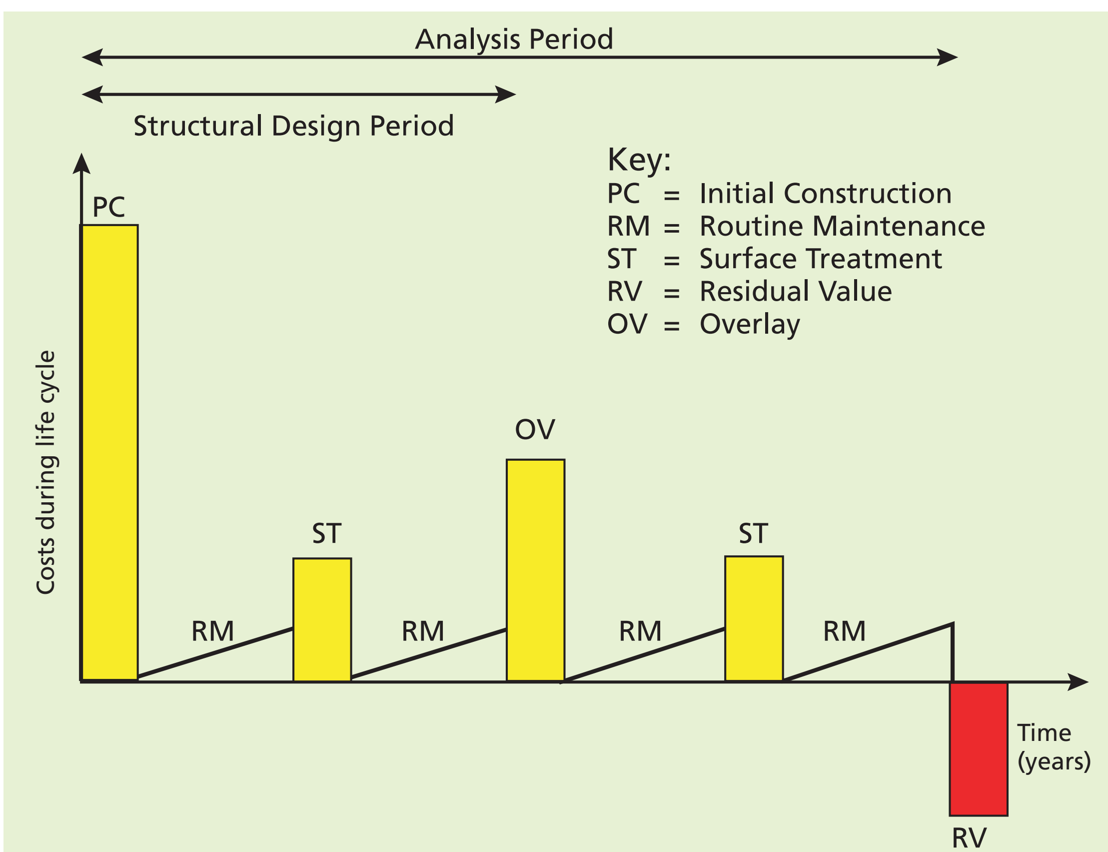

Figure 16-3: Components of a typical life cycle cost analysis

As indicated in Figure 16-4, the optimum road design standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs. Thus, as illustrated in Figure 16-5, the optimum road design standard, in terms of the pavement structural capacity, for a relatively low traffic pavement would incur lower initial construction costs but, within its life cycle, this would be balanced by higher maintenance and VOC. Conversely, a higher traffic pavement would incur higher initial construction costs but lower maintenance and VOC.

Analysis period
This period is the length of time for which comparisons of total costs are to be made. It should be the same for all alternative strategies and should not be less than the longest design period of the alternative strategies.

Structural design period
This is the design life of the road at which time it would be expected to have reached its terminal serviceability level and to require an appropriate intervention such as an overlay.

Construction costs
Unit costs for alternative pavement designs will vary widely depending on such factors as locality, availability of suitable materials, scale of project and road standard. Other factors that would typically warrant consideration include:

- Land acquisition costs.
- Supervision and overhead cost.
- Establishment costs.
- Accommodation of traffic.
- Relocation of services.

Maintenance costs
The nature and extent of future maintenance will be dependent on pavement composition, traffic loading and environmental influences. An assessment needs to be made of future annual routine maintenance requirements, periodic treatments such as reseals, and rehabilitation such as structural overlays.

Road user costs
These are normally not considered in a LCC analysis, as the pavement designs are considered to provide "equivalent service" during the analysis period. However, when evaluating the viability of costly measures to improve or maintain a high roughness level, e.g. treatments for expansive clays, the savings for the road user (vehicle operating costs) compared with the cheapest option are treated as benefits and should be incorporated as one of the components in the LCC analysis (ref. Figure 16-2). Vehicle operating costs (VOC) are related to the roughness of the road in terms of its International Roughness Index (IRI) and will change over the life of the road due to changes in surface condition and traffic. Relationships can be developed for main vehicle types which relate VOCs to variations in road surface conditions (IRI) under local conditions.

Road user costs are normally excluded from a LCC analysis that is confined to comparing alternative pavement/surfacing options, as the pavement options are considered to provide "equivalent service" during the analysis period. However, when evaluating the viability of upgrading a gravel road to a paved standard, the savings for the road user (primarily vehicle operating costs) on the latter versus the former option can be significant and are treated as benefits which should be incorporated as one of the components in the LCC analysis (ref. Figure 16-2).

Salvage value
The value of the pavement at the end of the analysis period depends on the extent to which it can be utilized in any future upgrading. For example, where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the subbase layer for the subsequent project, then the salvage value would be equal to the cost in current value terms for construction in future to subbase level discounted to the evaluation year.

Discount rate
This rate must be selected to express future expenditure in terms of present values and cost. It is usually based on a combination of policy and economic considerations.

LCC Procedure
The procedure that is followed typically in undertaking a LCC analysis of mutually exclusive projects, i.e. the selection of one project precludes selection of the other project.

1. Establish alternative project options.
2. Determine analysis period.
3. Estimate agency (construction and maintenance) costs.
4. Estimate road user costs.
5. Develop expenditure stream diagrams (similar to Figure 16-3).
6. Compute NPV of both options.
7. Analyse results, including sensitivity analysis, if warranted.
8. Decide on preferred option, i.e. the option with the highest NPV.

16.2.4 Selection of Road Design Standard

The selection of an appropriate pavement design standard requires an optimum balance to be struck between construction/rehabilitation, maintenance and road user costs, such as to mimimise total life cycle costs, as illustrated in Figure 16-4. Such an analysis can be undertaken using an appropriate techno-economic model, such as the World Bank's Highway Design and Maintenance Standards (HDM) model or, preferably, the Low Volume Road Economic Decision (RED) model which is customised to the characteristics of LVRs.

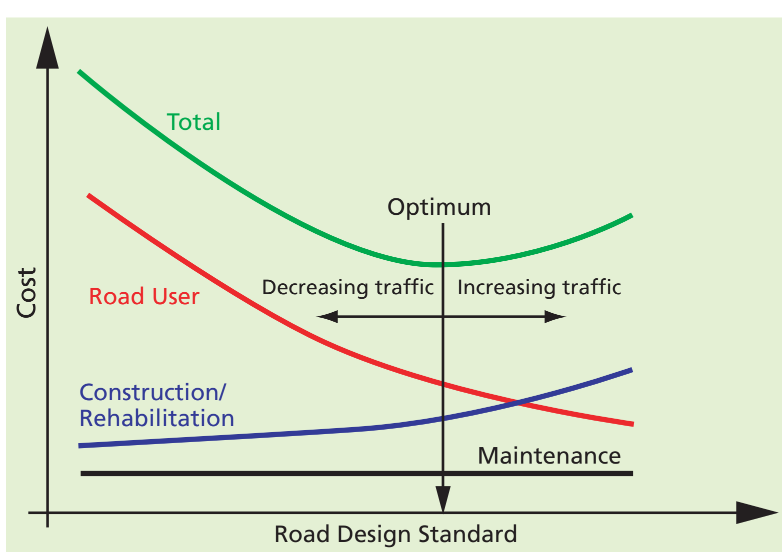

Figure 16-4: Economic analysis of optimum road design standard

As indicated in Figure 16-4, the optimum road design standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs. Thus, as illustrated in Figure 16-5, the optimum road design standard, in terms of the pavement structural capacity, for a relatively low traffic pavement would incur lower initial construction costs but, within its life cycle, this would be balanced by higher maintenance and VOC. Conversely, a higher traffic pavement would incur higher initial construction costs but lower maintenance and VOC.

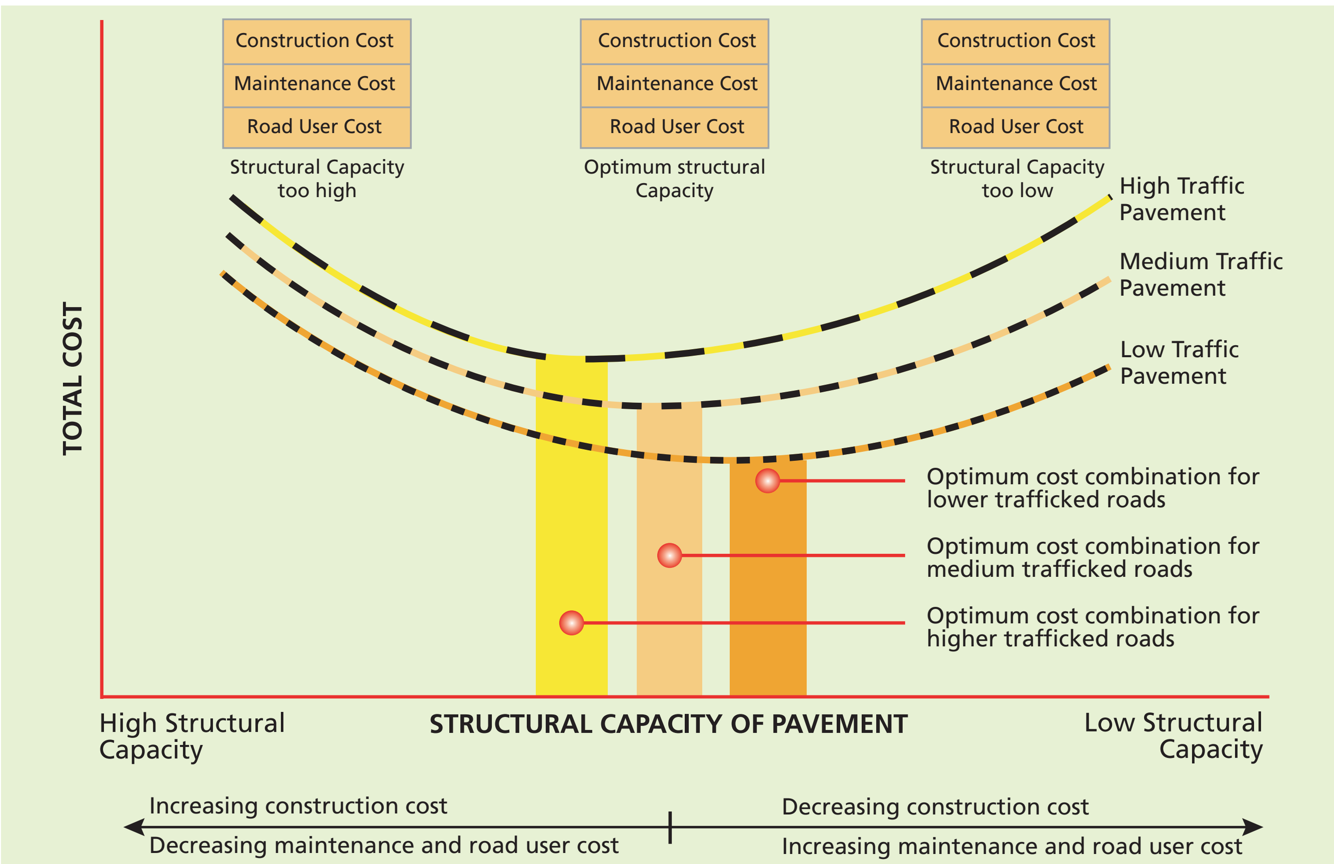

Figure 16-5: Combined cost for various pavement structure capacities

16.2.5 Gravel versus paved road comparison

A typical situation faced by a road agency is – when is it economically justified to upgrade a gravel road to a paved standard. As illustrated in Figures 16-6 and 16-7, both the gravel and paved road options would have a have a different relative mix of construction, maintenance and road user costs. In such a situation, a LCC analysis can be undertaken to determine the viability of upgrading a gravel road to a paved standard.

The typical components of the LCC analysis are illustrated in Figure 16-8 and could be undertaken using an appraisal model such as RED in which the VOC relationships may need to be calibrated for local conditions. The option with the higher NPV would be the preferred one.

Figure.16-6: Gravel road option
(Lower construction costs, higher maintenance and road user costs)

Figure 16-7: Paved road option
(Higher construction costs, lower maintenance and road user costs)

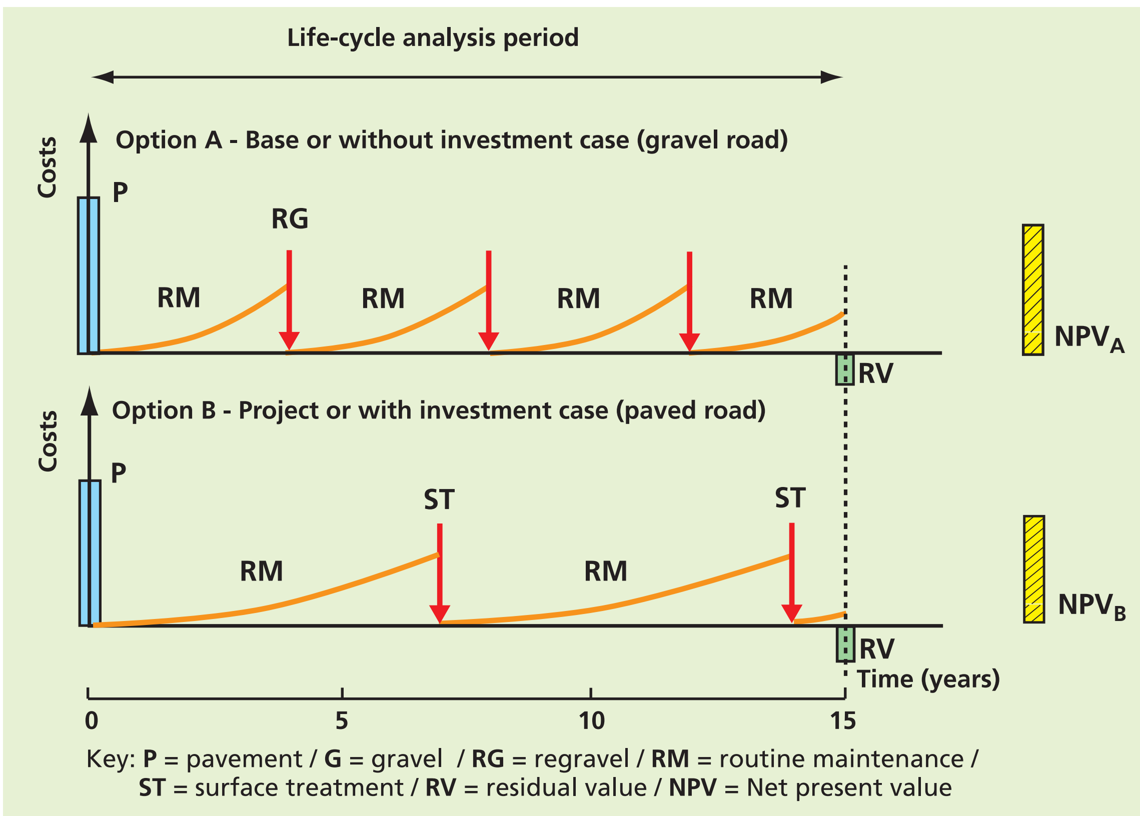

Figure 16-8: Typical components of a LCC: gravel versus paved road

16.2.6 Selection of surfacing option

A LCC analysis can also be undertaken to determine the most cost-effective type of surfacing to use on a LVSR. Such an analysis entails comparing the construction and maintenance costs of alternative surfacings during the life of the road for which the main inputs to the analysis would typically include:

- Assumed service life of surfacing.
- Construction cost for surfacing options.
- Maintenance cost for surfacing options.
- Discount rate.

The analysis assumes that the vehicle operating costs imposed by the various options are similar due to very small differences in their roughness levels.

Figure 16-9 and Tables 16-1 and 16-2 illustrate the manner of undertaking a LCC analysis for two typical types of bituminous surfacings by comparing the PV of all costs and maintenance interventions that occur during a given analysis period. The example is a hypothetical one used for illustrative purposes only and does not necessarily reflect a real life situation.

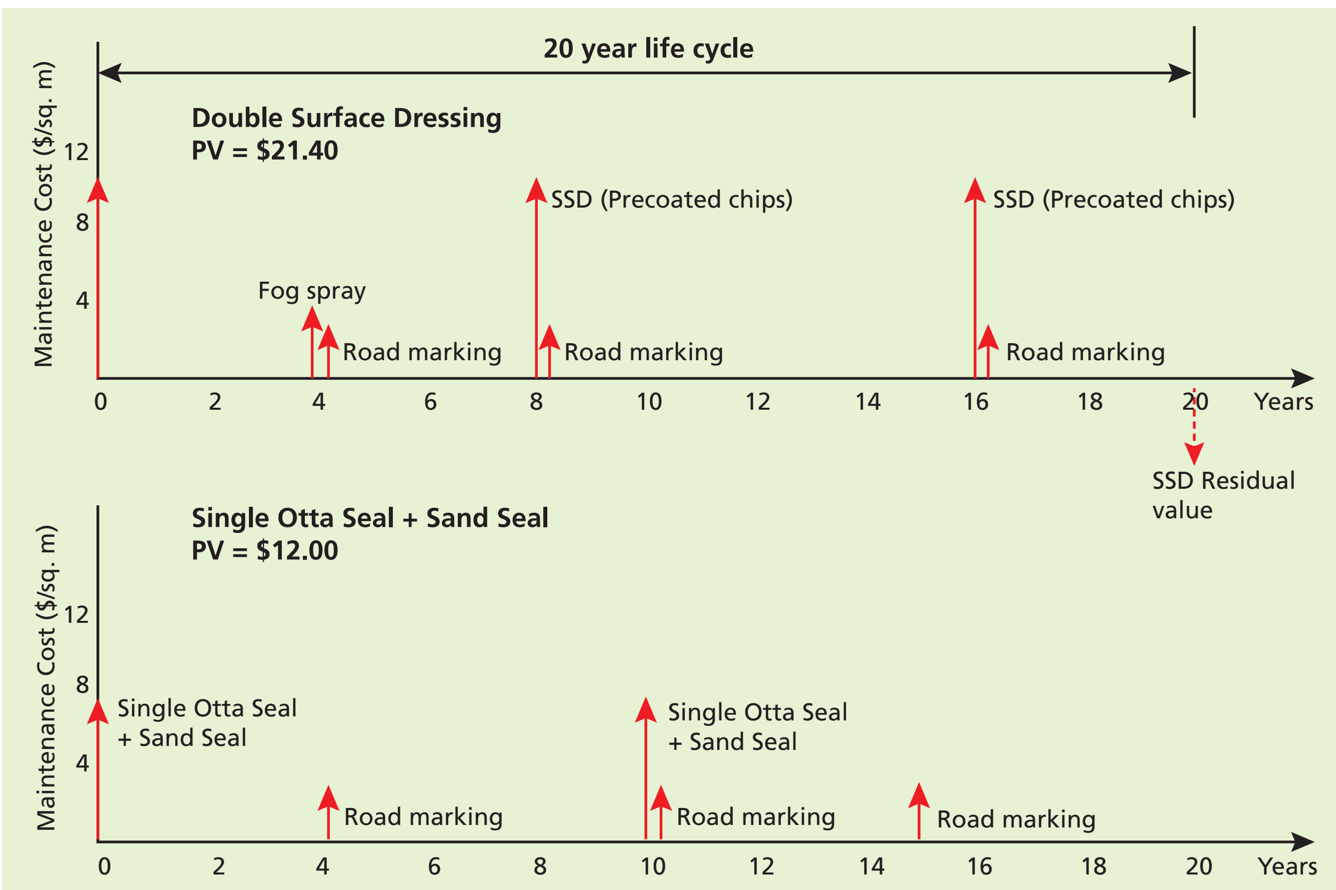

Figure 16-9: Cost components of a LCC comparison between a single Otta seal + Sand Seal and a Double Surface Dressing

Table 16-1: Life cycle cost analysis for Double Surfacing

Activity	Years after construction	Base Cost/m² ($)	8% Discount Factor	PV of Costs/m² ($)
1. Construct Double Chip Seal	-	10.00	1.0000	10.00
2. Fog spray	4	02.00	0.7350	1.47
3. Road marking	4	00.96	0.7350	0.71
4. Single Chip Seal (pre-coated)	8	10.00	0.5403	5.40
5. Road marking	8	00.96	0.5403	0.52
6. Fog spray	12	2.00	0.3971	0.79
7. Road marking	12	00.96	0.3971	0.38
8. Single Chip Seal (pre-coated)	16	10.00	0.2919	2.92
9. Road marking	16	00.96	0.2919	0.28
10. Residual value of surfacing	20	(5.00)	0.2145	(1.07)
				Total 21.40/m²

Table 16-2: Life cycle cost analysis for single Otta Seal + Sand Seal

Activity	Years after construction	Base Cost ($)	8% Discount Factor	PV of Costs ($)
1. Construct single Otta Seal + Sand Seal	-	7.25	1.00	7.25
2. Road marking	5	0.96	0.6806	0.65
3. Single Otta reseal	10	7.25	0.4632	3.36
4. Road marking	10	0.96	0.4632	0.44
5. Road marking	15	0.96	0.3152	0.30
Assume life span of 20 years. Thus, no residual value.				0.00
				Total 12.00/m²

Note: It is assumed in the above example that the underlying pavement structures are identical in both options.

16.3 IMPLICATIONS OF IMPLEMENTING REVISED APPROACHES

16.3.1 General

The implications of using the revised approaches recommended in this Guideline are to significantly reduce both the initial construction and longer terms maintenance costs. This factor, coupled with an investment model, such as the World bank's Roads Economic Decision (RED) model or the Sabita Manual 7 - Economic warrants for Surfacing Unpaved Roads SuperSurf) , which are able to capture and quantify important socio-economic benefits, is to reduce the threshold level at which it may be economically justified to pave an earth/gravel road as illustrated in Figure 16-7.

16.3.2 Factors Influencing Traffic Threshold for Upgrading

Some of the factors which continue to be identified and quantified through research and which are changing the traffic threshold for upgrading gravel roads are given in Table 16-3.