GUIDELINES FOR THE ENVIRONMENTALLY OPTIMIZED DESIGN OF LOW VOLUME ROADS

Australian Government
Department of Foreign Affairs and Trade

PRADHAN MANTRI GRAM SADAK YOJANA

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TABLE OF CONTENTS

REFERENCES 82

LIST OF ANNEXES

LIST OF FIGURES

LIST OF TABLES

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Foreword

Pradhan Mantri Gram Sadak Yojana (PMGSY) is a major flagship programme of Government of India to connect eligible habitations with black top roads. So far almost 1,31,000 habitations have been connected under PMGSY with a planned road network of 6,40,000 km. Indian Roads Congress came up with separate specifications for rural roads which witnessed low traffic volumes. Resultantly per km cost in PMGSY has been substantially on the lower side.

Several initiatives have been introduced in the largest rural connectivity program, PMGSY (Pradhan Mantri Gram Sadak Yojana) by the National Rural Roads Development Agency, Ministry of Rural Development through mandatory use of innovative technologies for 15 percent of the roads being constructed under the program.

It has been identified that the local and industry waste materials can be effectively used in low volume roads, thereby reducing the logistics cost of transporting road construction material resulting in lower construction costs. In several cases, the prescribed pavement composition is found to be over designed and pavement thickness could be reduced especially where the subgrade is of good quality.

With the support of the World Bank, Korea Green Growth Partnership and the Department of Foreign Affairs and Trade, Australia, the Guidelines for environmentally optimized designs for low volume roads have been prepared to introduce cost-effective and environmentally optimised approach to the design of rural roads. Therefore, by promoting the use of locally available resources and industrial by-products, simplified and environment-friendly pavement design method such as use of dynamic cone penetrometer (DCP) and use of low cost bitumen seals for low volume roads. Maximizing the use of locally available materials, through suitable processing, and optimisation of pavement composition will not only minimize the construction cost but also promote efficient use of the scarce road building materials and fuel, minimization of adverse impacts on environment, and sustainability of the vast road network being built under the PMGSY and other rural road programs.

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The activity was led by Ashok Kumar, and the team comprised Arnab Bandyopadhyay, Reenu Aneja, D.P. Gupta, Rashi Grover, and Lakshmi Narayan. The international panel of experts engaged to prepare these Guidelines included Philip Paige-Green, Mike Pinard, Gerrie Van Zyl and Dr. Arun Kumar. The team gratefully acknowledges the useful insights and guidance received from Karla Gonzalez Carvajal (Practice Manager) and Shomik Raj Mehndiratta (Practice Manager) as well as the support and guidance provided by Eun Joo Allison Yi (KGGTF Program Manager) and Laurent Durix (Consultant).

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TERMINOLOGY

Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt	Same as Bitumen.
Average Annual Daily Traffic (AADT)	The total yearly traffic volume in both directions divided by the number of days in the year.
Average Daily Traffic (ADT)	The total traffic volume during a given period in whole days greater than one day and less than one year divided by the number of days in that time period.
Base Course	The main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Materials stabilised with cement or lime can also be used.
Bitumen	The residue from the refining of crude oil after the more volatile material has been distilled off. It is a viscous liquid comprising many long-chain organic molecules. For use in roads it is practically solid at ambient temperatures but can be heated sufficiently to be poured and sprayed.
Borrow Area	An area within designated boundaries, approved for obtaining borrow material. A borrow pit is the excavated pit in a borrow area.
Borrow Material	Any gravel, sand, soil, rock or ash obtained from borrow areas, dumps or sources other than cut within the road prism and which is used in the construction of the specified work for a project. Does not include crushed stone or sand obtained from commercial sources.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 0.30 m or more.
Capping layer	The top of embankment or bottom of excavation prior to construction of the (selected subgrade) pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better quality subgrade material imported from elsewhere or subgrade material improved by stabilisation (usually mechanical), and may also be considered as a lower quality sub-base.
Carriageway	That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.

Cross fall	The difference in level measured transversely across the surface of the roadway.
Culvert	A structure, other than a bridge, which provides an opening under the carriageway or median for drainage or other purposes.
Cutting	Cutting shall mean all excavations from the road prism including side drains, and excavations for intersecting roads including, where classified as cut, excavations for open drains.
Chippings	Stones used for surface dressing (treatment).
Deformed Bar	A reinforcing bar for rigid slabs conforming to “Requirements for Deformations” in AASHTO Designations M 31 M.
Design Period	The period of time that an initially constructed or rehabilitated pavement structure is intended to perform before reaching a level of deterioration requiring more than routine or periodic maintenance.
Diverted Traffic	Traffic that diverts from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
Dowel	A load transfer device in a rigid slab, usually consisting of a plain round steel bar. Unlike a tie bar, a dowel may permit horizontal movement.
Equivalent Standard Axle (ESA)	A measure of the potential damage to a pavement caused by a vehicle load expressed as the number of 8.2 metric tonnes single axle loads that would cause the same amount of damage. The ESA values of all the traffic are combined to determine the total design traffic for the design period.
Equivalency Factors	Used to convert traffic volumes into cumulative equivalent standard axle loads.
Equivalent Standard Axle Load (ESA)	Summation of equivalent 8.16 tonne single axle loads used to combine mixed traffic to calculate the design traffic loading for the design period.
Escarpment	Escarpments are geological features that are very steep and extend laterally for considerable distances, making it difficult or impossible to construct a road to avoid them. They are characterised by more than 50 five-metre contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 60%.
Expansion Joint	A joint located to provide for expansion of a rigid slab without damage to itself, adjacent slabs, or structures.
Fill	Material of which a man-made raised structure or deposit such as an embankment is composed, including soil, soil-aggregate or rock. Material imported to replace unsuitable roadbed material is also classified as fill.
Flexible Pavements	Include primarily those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.

Formation Level	Level at top of subgrade.
Generated Traffic	Additional traffic which occurs in response to the provision of improved road.
Heavy Vehicles	Those having an unloaded weight of 3000 kg or more.
Hot Mix Asphalt (HMA)	This is a generic name for all high quality mixtures of aggregates and bitumen that use the grades of bitumen that must be heated in order to flow sufficiently to coat the aggregates. It includes Asphaltic Concrete, Dense Bitumen Macadam and Hot Rolled Asphalt.
Longitudinal Joint	A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.
Maintenance	Routine work performed to keep a pavement as nearly as possible in its as-constructed condition under normal conditions of traffic and forces of nature.
Mountainous (Terrain)	Terrain that is rugged and hilly with substantial restrictions in both horizontal and vertical alignment. It is defined as having 26-50 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.
Normal Traffic	Traffic which would pass along the existing road or track even if no improved pavement were provided.
Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a levelling course, to correct the profile of the old pavement, followed by a uniform course or courses to provide needed thickness.
Pavement Layers	The layers of different materials which comprise the pavement structure.
Project Specifications	*The specifications relating to a specific project, which form part of the* contract documents for such project, and which contain supplementary and/or amending specifications to the standard specifications.
Pumping	The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.
Quarry	An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.
Reconstruction	The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.
Recycling	The reuse, usually after some processing, of a material that has already served its first-intended purpose.
Rehabilitation	Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and pre overlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.
Reinforcement	Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.

Rigid Pavement	A pavement structure which distributes loads to the subgrade having, as the main load bearing course, a Portland cement concrete slab of relatively high-bending resistance.
Road base	A layer of material of defined thickness and width constructed on top of the sub-base, or in the absence thereof, the subgrade. A road base may extend to outside the carriageway.
Road Bed	The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.
Road Bed Material	The material below the subgrade extending to such depth as affects the support of the pavement structure.
Road Prism	That portion of the road construction included between the original ground level and the outer lines of the slopes of cuts, fills, side fills and side drains. It does not include sub-base, road base, surfacing, shoulders, or existing original ground.
Roadway	The area normally travelled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.
Rolling (Terrain)	Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment. Defined as terrain with 11-25 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 10 and 25%.
Side Fill	That portion of the imported material within the road prism which lies outside the fills, shoulders, road base and sub-base and is contained within such surface slopes as shown on the Drawings or as directed by the Engineer. A distinction between fills and side fill is only to be made if specified.
Side Drain	Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.
Stabilisation	The treatment of the materials used in the construction of the road bed material, fill or pavement layers by the addition of a cementitious binder such as lime or Portland Cement or the mechanical modification of the material through the addition of a soil binder or a bituminous binder. Concrete and asphalt shall not be considered as materials that have been stabilised.
Sub-base	The layer of material of specified dimensions on top of the subgrade and below the road base. The secondary load-spreading layer underlying the base course. Usually consisting of a material of lower quality than that used in the base course and particularly of lower bearing strength. Materials may be unprocessed natural gravel, gravel-sand, or gravel-sand-clay, with controlled gradation and plasticity characteristics. The sub-base also serves as a separating layer preventing contamination of the base course by the subgrade material and may play a role in the internal drainage of the pavement.

Subgrade	The surface upon which the pavement structure and shoulders are constructed. It is the top portion of the natural soil, either undisturbed (but recompacted) local material in cut sections, or soil excavated in cut or borrow areas and placed as compacted embankment.
Subsurface Drain	Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.
Surface Treatment	The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and crushed stone chippings.
Surfacing	This comprises the top layers(s) of the flexible pavement and consists of a bituminous surface dressing or one or two layers of premixed bituminous material (generally asphalt concrete). Where premixed materials are laid in two layers, these are known as the wearing course and the binder course. Surfacings can also be non-bituminous.
Tie Bar	A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.
Traffic Lane	Part of a travelled way intended for a single stream of traffic in one direction, which has normally been demarcated as such by road markings.
Traffic Volume	Volume of traffic usually expressed in terms of Average Annual Daily Traffic (AADT).
Typical Cross-section	A cross-section of a road showing standard dimensional details and features of construction.
Unbound Pavement Materials	Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.
Wearing Course	The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.
Welded Wire Fabric	Welded steel wire fabric for concrete reinforcement.

LIST OF ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHO	American Association of State Highway Officials (old designation)
AASHTO	American Association of State Highway and Transportation Officials (current designation)
ASTM	American Society for Testing Materials
CBR	California Bearing Ratio
DCP	Dynamic Cone Penetrometer
EOD	Environmentally Optimized Design
FMC	Field Moisture Content
ESA	Equivalent Standard Axles
GGBS	Ground Granulated Blastfurnace Slag
HCV	Heavy Commercial Vehicle
HMA	Hot Mixed Asphalt
HVR	High Volume Road
ISD	Initial Stabiliser Demand
LSP	Layer Strength Profile
LVR	Low Volume Road
MCV	Medium Commercial Vehicle
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
OMC	Optimum Moisture Content
PCC	Portland Cement Concrete
PFA	Pulverised Fly Ash
PMGSY	Pradhan Mantri Gram Sadak Yojana
UCS	Unconfined Compressive Strength
VDF	Vehicle Damage Factor
VPD	Vehicle Per Day, also used in the text as 'vpd'

1. INTRODUCTION

1.1 BACKGROUND

The provision of all-weather road access to all citizens is a key development priority facilitating both accessibility and road connectivity to the many habitations in the rural areas of India, which is being implemented under a national level program, Pradhan Mantri Gram Sadak Yojana (PMGSY) and many sub-national programs. The traffic levels on most of these roads are very low, typically below 300 motorised vehicles per day, consisting of mostly farm tractors and light commercial vehicles with very few heavy commercial vehicles.

Whilst there are potentially significant life-cycle benefits to be achieved from upgrading the existing unpaved low volume rural roads to a paved standard, the cost of doing so following traditional approaches to road design and materials utilization can be prohibitive. This is because such approaches tend to be overly conservative for application to Low Volume Roads (LVRs). This has led to a need to develop an alternative design procedure with the objective of enhancing the efficiency and effectiveness of LVR provision and, by extension, reducing the cost of providing much needed connectivity in the rural areas of the country.

1.2 PURPOSE

The main purpose of these Guidelines are to provide practitioners with the requisite tools for undertaking an environmentally optimized approach to the design of LVRs in India that takes account of the many locally prevailing road environment factors that impact on the design of such roads. Such an approach is aimed at providing appropriate and cost-effective designs for LVRs, bearing in mind their practical implementability within the available resources and level of expertise in rural areas. To the extent possible, the use of locally available materials in their natural state, or after suitable processing, must be maximized to not only reduce construction costs but, also, to minimize adverse environmental impacts regarding the use of non-renewable resources (aggregate and gravel). The availability of plant and equipment for construction and maintenance, as well as the level of quality control that can be effectively exercised in the field should also be considered.

1.3 SCOPE

The Guidelines take account of the many advances in LVR technology based on a number of international research and investigation projects that have been carried out in environments similar to those prevailing in India [1,2,3,4,5]. The corroborative findings of this work provide a wealth of performance-based information that has advanced previous knowledge on various aspects of LVR technology. This has allowed state-of-the-art guidance to be provided in the Guidelines including more extensive use of local and by-product materials, simplified, environmentally optimized pavement design methods, optimization of pavement composition and use of a range of potentially suitable bituminous surfacing options.

The Guidelines incorporate the latest approaches to the provision of LVRs that mirror the sequential activities that are typically undertaken in designing such roads, i.e. activities that progress from the surveys and investigations stage, through to the structural design of existing and new roads, selection of appropriate surfacing type, attention to drainage as well as aspects of construction dealing with quality assurance and control.

The Guidelines complement and link to relevant aspects of the latest versions of other manuals in India including:

- IRC Guidelines for the Design of Flexible Pavements for Low Volume Rural Roads [6].
- New Technology Initiatives under PMGSY [7].
- IRC Guidelines of Road Drainage [8].
- MORD Specifications for Rural Roads [9].
- MORD Quality Assurance Handbook for Rural Roads [10].
- IRC Recommendations for road construction in areas affected by water-logging, flooding and/or salt infestation [11].

The main differences between these *Guidelines for Environmentally Optimized Design of Low Volume Roads* and the above guidelines are:

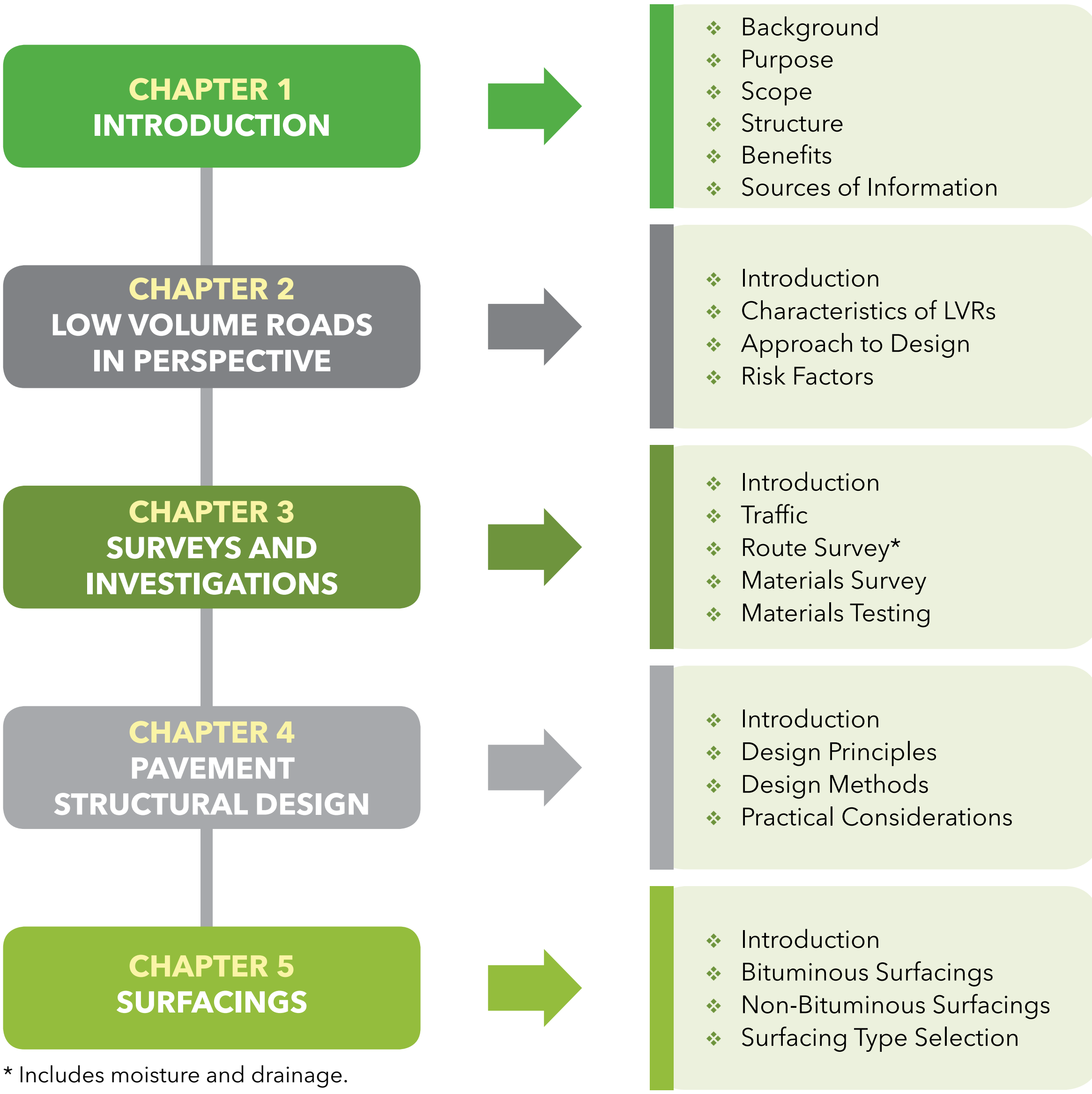
- It moves away from the more traditional, empirically developed, CBR design approach, which provides an indirect measure of the strength of a material, to a more direct method of measuring in situ shear strength based on the use of the Dynamic Cone Penetrometer (DCP).
- It focuses on the use of the DCP for evaluating in situ road conditions and, by integrating the design strength profile optimally with the in situ strength profile, for designing LVR pavement structures in a cost-effective manner that minimizes the use of imported materials.
- It facilitates the greater use of local, more abundant, and therefore less expensive, locally available and by-product materials in the road pavement by a variety of techniques for improving strength of these materials and for evaluating their properties in the laboratory using the DCP.
- It offers a wider array of relatively low-cost bituminous surfacings in addition to the more traditionally used surface dressing and premix carpet.
- It also allows improved consideration of factors such as soaked or unsoaked conditions of pavements.

In essence, therefore, the Guidelines provide an alternative approach to the design of LVRs that, in some respects complements, and in other respects enhances, traditional approaches adopted in other IRC guidelines.

1.4 STRUCTURE

The Guidelines are structured as shown in Figure 1.

FIGURE 1: STRUCTURE OF GUIDELINES



* Includes moisture and drainage.

A brief description of each chapter is given below:

CHAPTER 1 – INTRODUCTION: Discusses the background, purpose and scope of the Guidelines.

CHAPTER 2 – LOW VOLUME ROADS IN PERSPECTIVE: Places in broad perspective the various factors that affect the provision of LVRs, including the definition and particular characteristics of LVRs, the environmentally optimized design philosophy and various sustainability and implementation considerations.

CHAPTER 3 – SURVEYS AND INVESTIGATIONS: Details the procedures to be followed in obtaining the basic inputs to the design of the road pavement, including traffic, route and materials surveys as well as moisture and drainage investigations and materials testing.

CHAPTER 4 – PAVEMENT STRUCTURAL DESIGN: Includes the principles of LVR pavement design and the methods available, with a focus on the use of the Dynamic Cone Penetrometer method. Also considers several related practical considerations including compaction, drainage and shoulders. Includes an Annex with a typical design example.

CHAPTER 5 - SURFACINGS: Provides an overview of the various types of bituminous and non-bituminous discrete-element surfacings that are potentially suitable for use on LVRs, their performance characteristics, and the factors that may govern their selection. Concrete surfacings are dealt with in a separate Guidelines.

The Guidelines do not address other complementary aspects of road design such as geometric and drainage design, road safety, construction issues or special engineering measures required for hill roads or roads in flood-prone areas, which are addressed in other IRC guidelines.

A number of explanatory Annexes are also included in the Guidelines as follows:

- Annex I: Determination of Design Traffic.
- Annex II: Determination of Uniform Sections from CUSUM Analysis.
- Annex III: Determination and Choice of DN Percentile Values.
- Annex IV: Comparison of DCP-DN and IRC CBR Catalogues.
- Annex V: DCP Design Example.

1.5 BENEFITS

There are several benefits to be gained from adopting the approaches advocated in the Guidelines. These include providing LVRs that:

- Are less expensive, in life-cycle cost terms, to build and to maintain through the adoption of more appropriate, environmentally optimized designs.
- Maximise the use of locally available, often unprocessed materials, including industrial by-product materials and, by so doing, avoiding long haulage of more expensive processed materials.
- Provide well balanced pavement structures that are relatively less sensitive to vehicle overloading.
- Offer improved design reliability due to the much larger data set of DCP-DN measurements for statistical analysis and pavement design based on discrete uniform sections rather than general blanket designs.
- Integrate sustainability aspects (social, environmental and economic success) into the design, delivery and operation of road infrastructure assets (ref. Sections 2.3.4 and 2.3.5).

1.6 SOURCES OF INFORMATION

In addition to providing general information and guidance, the Guidelines also serve as a valuable source document because of its comprehensive list of references from which readers can obtain more detailed information to meet their particular needs. A bibliography can be found at the end of the Guidelines.

2. LOW VOLUME ROADS IN PERSPECTIVE

2.1 INTRODUCTION

2.1.1 Background

The traditional approaches to the provision of LVRs in many tropical and sub-tropical countries tend to be based on technology and research carried out in external environments that are not reflective of those that prevail in these countries. While these "standard" approaches might still be appropriate for much of the main trunk road network, they remain conservative, inappropriate and too costly for application on much of India's rural road network. Thus, in facing the challenges of improving and expanding the country's LVR network, more appropriate approaches need to be considered.

The approach to the design of LVRs follows the general principles of any good road design. However, there are several important differences from the traditional approaches that need to be appreciated by the designer to provide designs that will meet with the multiple social, economic and environmental requirements in a sustainable manner.

2.1.2 Purpose and Scope

The main purpose of this chapter is to place in broad perspective the various factors that may govern the provision of LVRs. To this end, the chapter addresses the following topics:

- The particular characteristics of LVRs.
- The LVR design philosophy.
- Various implementation considerations.

2.2 CHARACTERISTICS OF LOW VOLUME ROADS

2.2.1 Definition

A common understanding of the definition of an LVR is crucially important as it will dictate the approach to undertaking the design of such roads in relation to their characteristics and the related criteria to be used in providing them at an appropriate level of service and minimum life cycle cost.

There is no internationally accepted definition of an LVR. In developed countries such as the USA, roads carrying about 400 Vehicles Per Day (VPD) are defined as very low volume roads. The figure that is currently, typically used is about 300 VPD or a design traffic loading not exceeding about 1 Million Equivalent Standard Axles (MESA). However, neither of these definitions provides a complete picture of the unique characteristics of an LVR in that there are many other characteristics that need to be considered in their design as discussed below. For traffic loading of more than 1 MESA, standard pavement design procedures should be followed.

2.2.2 Special features

The following specific features of LVRs affect the manner of their provision and need to be fully appreciated by the designer:

- They are constructed mostly from naturally-occurring, often "non-standard", moisture-sensitive materials.
- Pavement deterioration is driven primarily by environmental factors, particularly moisture, with traffic loading being a relatively lesser influential factor, and drainage being of paramount importance.
- The alignment may not necessarily be fully "engineered", especially at very low traffic levels, with most sections following the existing alignment.
- A need to cater for a significant amount of non-motorized traffic, especially in urban/ peri-urban areas, coupled with a focus on the adoption of a range of low-cost road safety measures.
- Variable travelling speeds that will seldom exceed 60 km/h, as dictated by local vehicle characteristics and prevailing topography.

A holistic appreciation of the attributes that characterize LVRs will guide designers in producing more appropriate designs with an emphasis on using a fit-for-purpose, context sensitive, environmentally optimized approach to their design and construction. This will place an onus on the design engineer to provide a road that meets the expected level of service at least life-cycle cost, based on a full understanding of the local environment and its demands, and to turn these to a design advantage.

2.3 APPROACH TO DESIGN

2.3.1 General approach

Whilst the approach to the design of LVRs follows the general principles of any good road design practice, the level of attention and engineering judgement required for optimal provision of such roads tends to be higher than that required for the provision of other roads. This is because optimizing a design requires a multi-dimensional understanding of all of the project elements and in this respect all design elements become context specific. This will require:

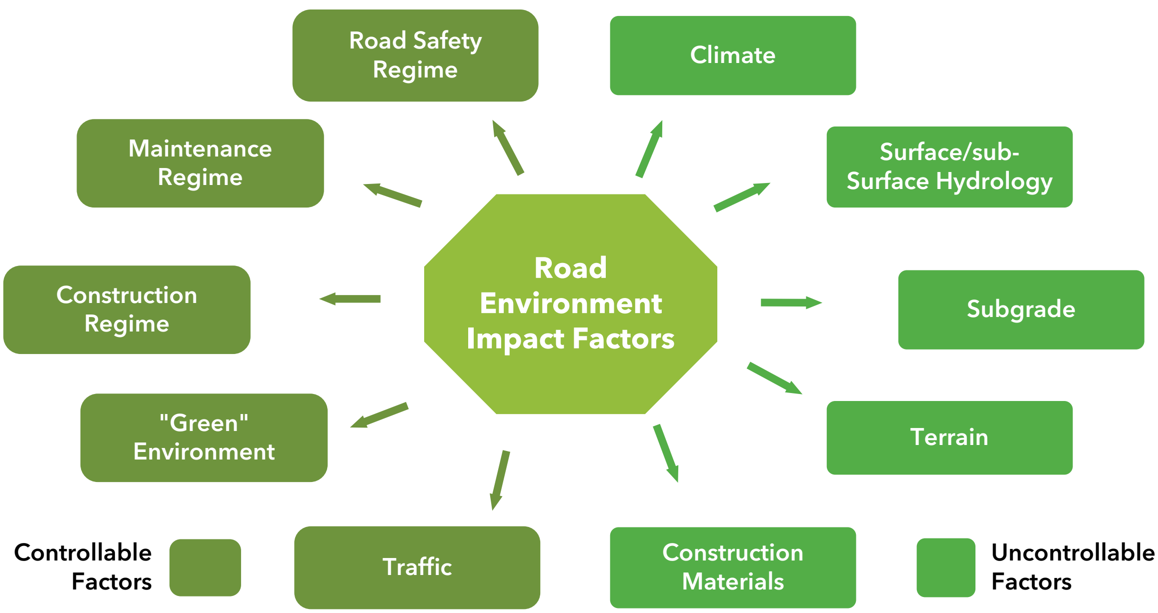
- A full understanding by the design engineer of the local environment (physical and social).
- Recognition and management of risk.
- Innovative and flexible thinking through the application of appropriate engineering solutions rather than following traditional thinking related to road design.

2.3.2 Influence of road environment

The term "road environment" is an all-encompassing one that includes both the natural or bio-physical environment and the human environment. It includes the interaction between the different environmental factors and the road structure. Some of these factors are uncontrollable, such as those attributable to the natural environment, including the interacting influence of climate (e.g. wind, rainfall and intensity), local hydrology and drainage, terrain and gradient. Collectively, these will influence the performance of the road and the design approach needs to recognize such influence by providing options that minimize the negative effects. Other factors, such as the construction and maintenance regime, safety and environmental demands, and the extent and type of traffic, are largely controllable and can be more readily built into the design approach.

Typical road environment factors that impact on the LVR design process are presented in Figure 2 and are covered in more detail in various parts of the Guidelines.

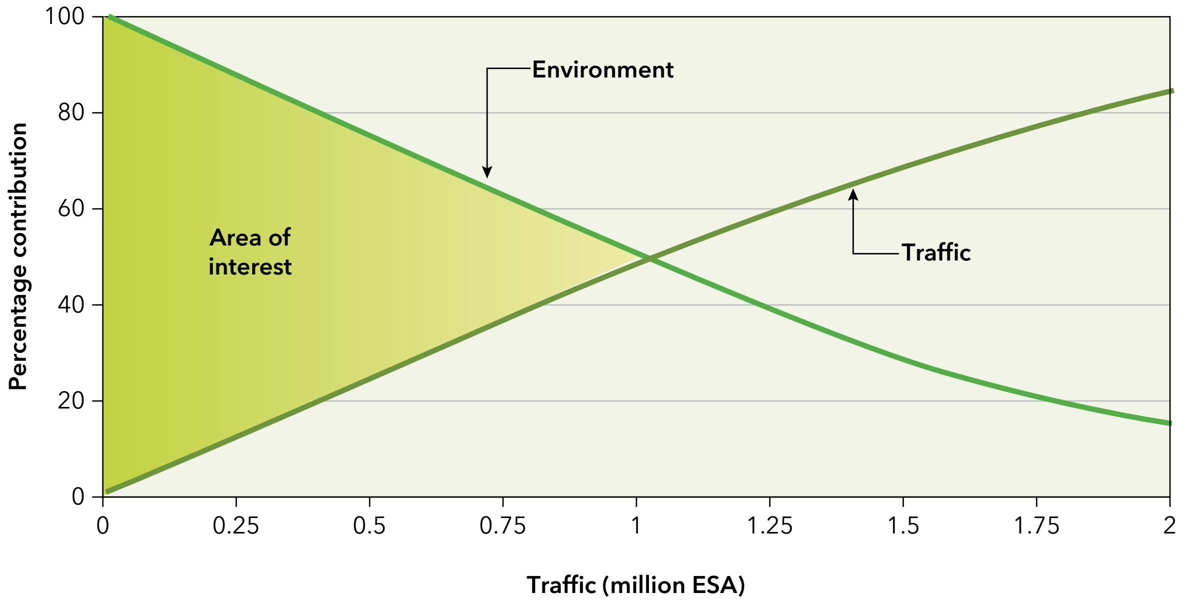
FIGURE 2: VARIOUS ROAD ENVIRONMENT FACTORS AFFECTING DESIGN



2.3.3 Road deterioration factors

Research carried out in many countries has shown that the relative influences of road deterioration factors are significantly different for LVRs compared with HVRs. A critical observation is that for LVRs carrying less than 1.0 MESA, pavement deterioration is controlled mainly by how the road responds to environmental factors, such as moisture changes in the pavement layers, fill and subgrade, rather than to traffic, as illustrated in Figure 3. Thus, particular attention needs to be paid to the influence of moisture in the design of LVR pavements and the adoption of appropriate drainage.

FIGURE 3: TRAFFIC LOADING VERSUS DOMINANT MECHANISM OF PAVEMENT DISTRESS (SCHEMATIC) [1]



2.3.4 Towards Sustainable Roads

The expectation to integrate sustainability aspects (social, environmental and economic success) into the design, delivery and operation of road infrastructure assets is growing rapidly. Moreover, a sustainable approach could lead to more cost-effective and environmentally sound solutions. Efficient material use, lean design and finding the shortest lead distances for construction can be important design criteria. Designs should consider the use of industry "waste", local materials, and reuse of building materials.

Construction materials

In constructing LVRs (for several thousand kilometres as planned in India), large quantities of materials, energy and water are required. The resources of most concern are those that are non-renewable. Consideration should be given to the following while designing roads and pavements:

- **Water:** Fresh water is becoming increasingly scarce and excessive consumption threatens ecosystem function. The aim of road assets at the design and construction cycle should thus be first to minimise water consumption, and then to replace potable water with effective reuse and recycling of locally appropriate alternative water sources.
- **Materials:** Road construction typically involves the consumption of large quantities of materials, a significant portion of which are derived from natural resources. The supplies of some of these resources are limited and are becoming increasingly scarce. The design and practice should encourage selection of materials that minimizes the consumption of precious resources such as crushed rock, which are generally transported over long distances. An example could be the use of locally available materials and their improvement on site for construction.

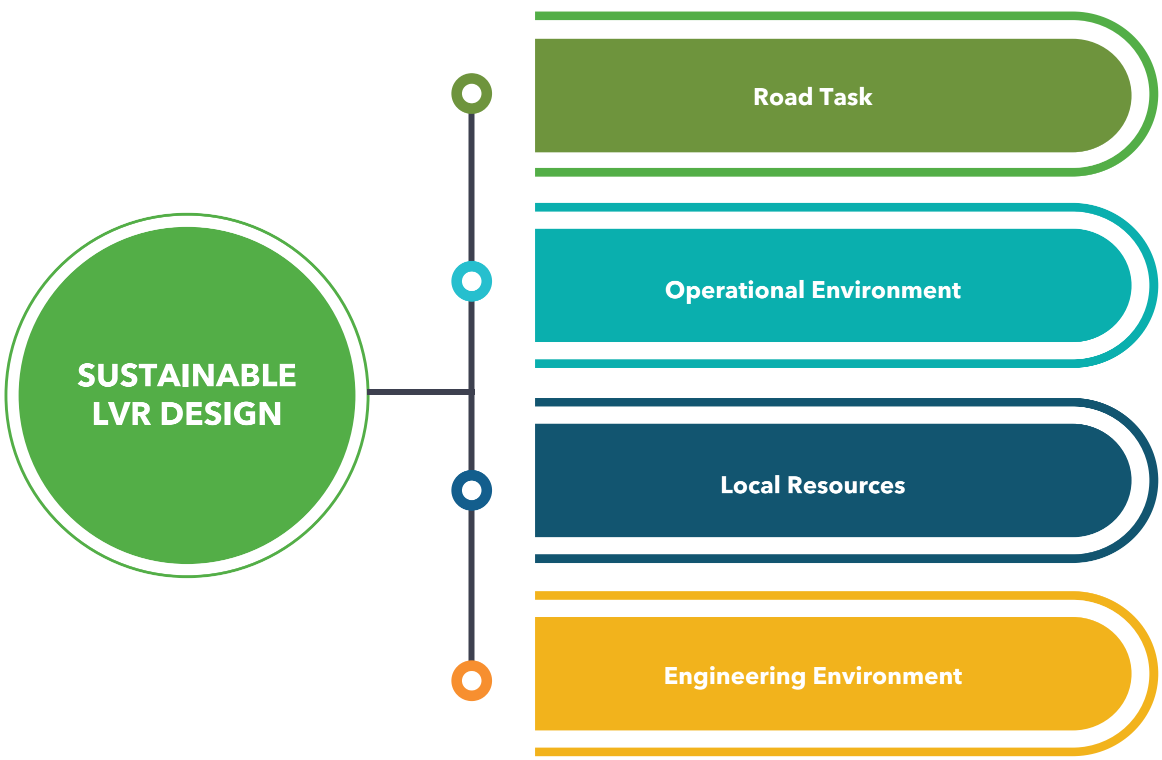
- **Waste materials:** Waste generation is increasing and recycling and reuse of the products are not increasing at the same rate. Waste from construction and demolition is significant and considerable amount is disposed to landfill. In the design, consideration should be given to the use of waste material through recycling, re-use and design optimisation.
- To achieve the best environmentally sustainable outcomes from investments in road infrastructure, sustainability implications should be considered at each phase of design, construction and maintenance of LVR.

2.3.5 Environmentally optimized design

To obtain optimal results from investments in road infrastructure in India, it is important to adopt an approach that is guided by appropriate local standards and conditions. In this regard, international research has highlighted the benefits of applying the principles of "Environmentally Optimized Design" (EOD) to the provision of LVRs in a manner that is compatible with the local road environment as outlined below and illustrated in Figure 4. The essence of the EOD approach is that it is:

1. **Task based:** LVRs must suit their identified function and the nature of the traffic (the people as well as the vehicles) which will pass along them, by applying appropriate standards.
2. **Environmentally compatible:** Suitable for, and where necessary, adapted to the local road environment factors.
3. **Local resource based:** The design of the LVR must be compatible with the construction materials that are readily available within appropriate specifications,

FIGURE 4: LVR IMPLEMENTATION WITHIN AN EOD CONTEXT



and within the capacities of the engineers and technicians who will design the roads, and the contractors who will construct them, and within the means of the roads agency to maintain them, involving local communities, where possible.

EOD can be described as a strategy for utilising the available resources of budget and materials in the most cost-effective manner to counter the variable factors of traffic, terrain, materials and subgrade that may exist along an alignment. To be successful and sustainable, LVR technology needs to be implemented within the framework of an EOD strategy. Moreover, if the LVR project is to be sustainable in the long run, several strategic objectives should be satisfied, including:

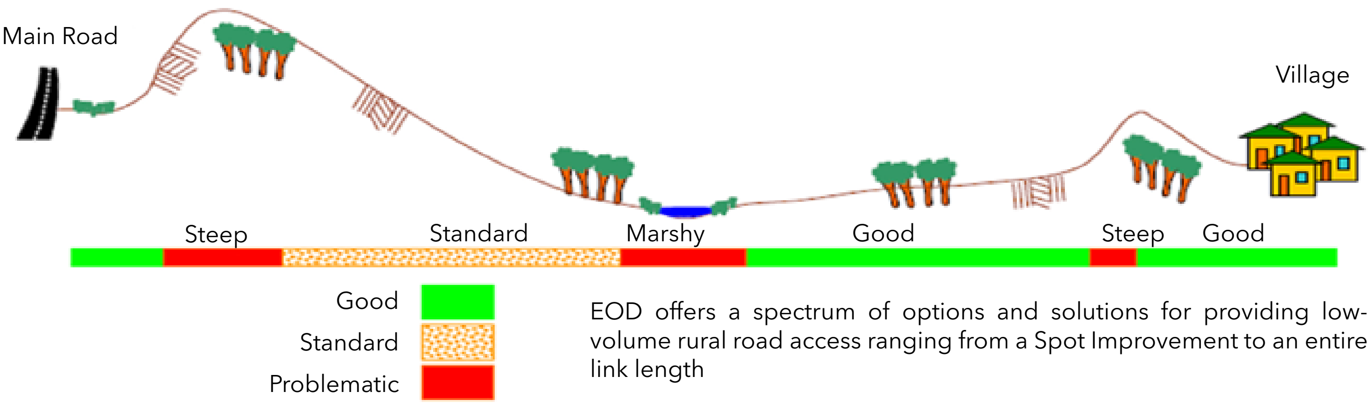
- Practical implementability of the recommended designs within the available resources and level of expertise in rural areas.
- Use of design standards and materials specifications that should aim at achieving an appropriate level of serviceability which should not fall below the minimum acceptable level during the design life.
- Availability of equipment/plant for construction and maintenance as well as the level of quality control that can be effectively exercised in rural areas.
- Maximum use of local labour and skills.
- Maximum use of locally available or processed materials.
- In-built maintenance considerations in the design such as provision of adequate drainage, resistance to soil erosion along the side slopes, adequate lateral support from shoulders etc. as would minimize subsequent maintenance requirements.

The EOD strategy should be applied with the overall aim of ensuring that each section of a road is provided with the most suitable pavement type for the specific circumstances prevailing along the road. This requires analysis of a broad spectrum of solutions to improve different road sections, depending on their individual requirements, ranging from engineered natural surfaces to bituminous pavements. The chosen solution must be achievable with materials, plant and contractors available locally.

The EOD approach ensures that specifications and designs support the functions of different road sections - assessing local environment and limited available resources. EOD assesses whether the standard design is sufficient for problematic areas and whether it is necessary for good areas. An under-design of poor sections can lead to premature failure and an over-design will often be a waste of resources which would be better applied on the problematic sections. Short problematic sections should be handled as described in Section 4.3.2.2, Step 7. The EOD principle is illustrated in Figure 5.

The use of the DCP method of design, which is highlighted in these Guidelines, combines the concepts of environmentally optimized design with in situ environmental and material conditions in a manner whereby specifications are adapted to suit the local road environment. This approach facilitates greater use of local, more abundant, and therefore less expensive, materials. By so doing, it reduces the need to import large quantities of virgin material by only adding a new layer (s), if necessary, to cater for the design traffic. This often results in reduced life cycle costs of LVR provision.

FIGURE 5: ENVIRONMENTALLY OPTIMIZED AND SPOT IMPROVEMENT DESIGN [SCHEMATIC]



2.4 RISK FACTORS

The departure from well-established, conservative material quality specifications may carry some increased level of risk of failure for an LVR. However, such a risk should be a calculated one and not a gamble and must consider not just materials but the whole pavement and its environment. Thus, in any pavement design strategy, it is necessary to be aware of the main risk factors which could affect the performance of LVRs so that appropriate measures can be taken to minimize them. These factors are summarized below:

- Quality of the materials (strength and moisture susceptibility).
- Construction control (primarily compaction standard and layer thicknesses).
- Environment (particularly drainage).
- Maintenance standards (drainage, surfacing and shoulders).
- Vehicle loads (overloading).

The risk of premature failure will depend on the extent to which the above factors are negative – the greater the number of factors that are unsatisfactory, the greater the risk of failure. However, this risk can be greatly reduced by adhering to the prescribed material specifications, by ensuring that the construction quality is well controlled and that drainage measures are strictly implemented and, probably most importantly, that maintenance is carried out in a timely manner and vehicle overloading is reasonably well controlled.

3. SURVEYS AND INVESTIGATIONS

3.1 INTRODUCTION

3.1.1 Background

Various surveys and investigations are required to collect and process the information required for the actual pavement design. These include the main input parameters for the pavement design:

- Traffic
- Subgrade conditions
- Material types and availability
- Moisture/drainage conditions

Each of these is critical to the final pavement design and must be carried out accurately and comprehensively to ensure the most cost-effective pavement structure is designed and to minimize the potential for construction delays due to unforeseen circumstances.

3.1.2 Purpose and Scope

The process and requirements of the traffic survey, subgrade investigation and details regarding material availability and moisture/drainage conditions are discussed in this chapter.

The purpose of this chapter is to outline the procedures to be followed in determining these parameters as a basis for designing the road pavement. The chapter considers types of surveys that provide the inputs for determining the design traffic loading, which requires the data to be sufficiently accurate to select the correct traffic category for structural design from the six classes appropriate to LVRs. Simplified methods of accomplishing this are described.

The chapter also covers the process of defining the subgrade conditions using the DCP apparatus to provide the input into the structural design decisions. The potential construction materials required for layer works in all classes of LVR are discussed. These can be obtained from borrow sources and “waste dumps” and hauled for use on the roads. Mechanisms for improving the quality of local materials are also highlighted. Finally, the need to identify sources of construction water during the site survey is briefly discussed and issues regarding material testing highlighted.

3.2 TRAFFIC

3.2.1 General

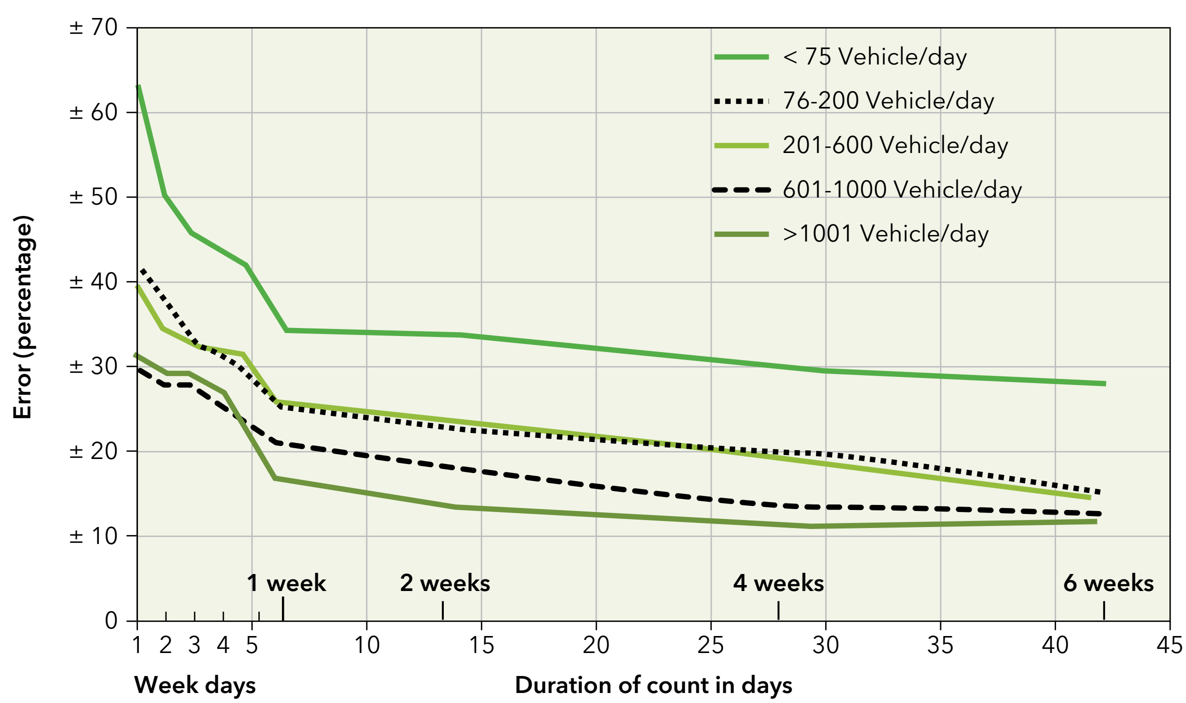
Reliable data on traffic volumes and characteristics is essential for both pavement and geometric design and assists in the planning of road safety measures as summarized below:

- **Pavement design:** The deterioration of the pavement is influenced by both the magnitude and frequency of individual axle loads. The large number of bicycles, motor cycles and pneumatic-tired animal drawn carts are of little consequence, and only commercial vehicles of gross laden weight > 3 tonnes are to be considered.
- **Geometric design:** The volume and composition of traffic, both motorized and non-motorized, influence the cross-section design (carriageway and shoulders).
- **Road safety:** The volume, type and characteristics of the traffic using the road will all influence the type of road safety measures required to ensure a safe road environment.

In view of the above, a reliable estimate of existing (base line) and future traffic statistics is required to undertake the design of the road in an appropriate manner. Aspects related to geometric design and road safety are not covered in this document.

Different traffic survey requirements are necessary for upgrading of an existing road and for a new road. For an existing road, the timing, frequency and duration of traffic surveys should be given very careful consideration in terms of striking a balance between cost and accuracy. As indicated approximately in Figure 6, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.

FIGURE 6: POSSIBLE ERRORS IN ADT ESTIMATES FROM RANDOM COUNTS OF VARYING DURATION [12]



In the case of a new road, an approximate estimate should be made of traffic that would use the road considering the number of villages and their population along the road environment and other socio-economic parameters. This can be achieved by carrying out traffic counts on an existing road, as described above, in the vicinity with similar conditions and knowing the population served as well as agricultural/industrial produce to be transported. Likely traffic on the new road can also be estimated from Origin-Destination (O-D) surveys along the nearby existing roads which presently serve the villages proposed to be connected (see section 3.2.2.2). However, such surveys for LVRs might be restricted to special areas such as industrial or business hubs.

For either a new road or an existing road, due consideration also needs to be given to the anticipated "Diverted" and "Generated" traffic because of the development of the proposed road, land use of the area served, the probable growth of traffic and the design life (see Section 3.2.3.3).

3.2.2 Traffic Surveys

The following types of traffic survey are typically carried out in the project area where the road is located:

- Classified Traffic Surveys
- Origin-Destination Surveys
- Axle Load Surveys

3.2.2.1 Classified traffic survey

A classified traffic count is one of the most important items of data for both geometric and pavement structural design as well as for planning purposes in terms of evaluating social and economic benefits derived from construction of LVRs. In most cases of LVRs, it would be sufficient to carry out only the classified traffic counts. It is necessary to ascertain the volume and composition of current and future traffic in terms of motorcycles, cars, light, medium and heavy goods vehicles, buses, and, importantly, non-motorized vehicles and pedestrians.

The most common types of surveys for counting and classifying the traffic in each class are:

- Manual Traffic Survey
- Automatic Traffic Surveys
- Moving Observer Methods

Although the methods of traffic counting may vary, the objective of each method remains the same - essentially to obtain an estimate of the Annual Average Daily Traffic (AADT) using the road, disaggregated by vehicle type. Prediction of such traffic is notoriously imprecise, especially where the roads serve a predominantly developmental or social function and when the traffic level is low.

Reducing errors in estimating traffic for LVRs

Errors in estimating traffic flow can be reduced, where possible, by:

- Counting for seven consecutive days.
- On some days counting for a full 24 hours, preferably with one 24-hour count on a weekday and one during a weekend; on other days, 16 hour counts (typically 06.00–22.00 hours) should be made and expanded to 24-hour counts using a previously established 16:24 hour expansion ratio. It is suggested that each state develop indices for this ratio, as these are likely to vary from state to state.
- Avoiding counting at times when road travel activity increases abnormally; for example, just after the payment of wages and salaries, or at harvest time, public holidays or any other occasion when traffic is abnormally high or low. However, if the harvest season is during the wet season (often the case, for instance, in the timber industry), it is important to obtain an estimate of the additional traffic typically carried by the road during these periods. This should also be adapted for local conditions as different crops can be harvested in different seasons.

Care should be exercised in selecting appropriate locations for conducting the traffic counts to ensure a true reflection of the traffic using the road and to avoid under- or over-counting. Local knowledge should be used to help with this.

Ideally, the accuracy of traffic counts can be improved by increasing the count duration or by counting in more than one period of the year. Improved accuracy can also be achieved by using local knowledge to determine whether there are days within the week or periods during the year when the flow of traffic is particularly high or low. It may also be possible to develop data relating the cumulative standard axles to the population and economic activities served in different districts or states.

Adjustments for season

An appropriate, weighted average adjustment will need to be made according to the season in which the traffic count was undertaken and the length of the wet and dry seasons, as illustrated in Figure 7.

Although the number and duration of harvesting seasons can vary from one region to another, typically two harvesting seasons each year are shown in Figure 7. If T is the average number of commercial vehicles of a given category, plying per day during the lean season, the enhanced traffic during the peak season can be denoted by nT, over and above the lean season traffic T, the value of n varying widely from one region to the other. Typically, it takes about 40% of the duration of a harvesting season (t) to build up from lean season level T to the peak. The peak traffic may continue for about 20% of the duration of the harvesting period before returning to the lean season traffic level. This usually takes about 40% of the total duration of the harvesting season.

Vehicle classification

Table 1 shows the vehicle classification system used for compiling the results of the traffic survey described above.

FIGURE 7: SEASONAL VARIATIONS IN RURAL TRAFFIC [6]

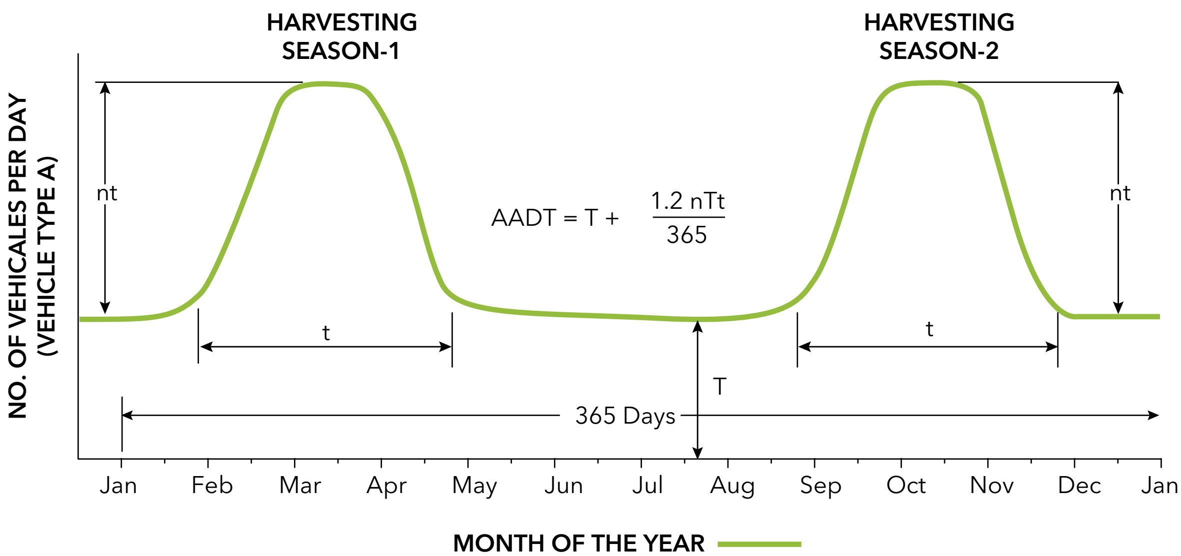


TABLE 1: VEHICLE CLASSIFICATION SYSTEM

Class	Type of Vehicle	Use
1	Car, Jeep, Van	Capacity analysis for geometric (cross section) design
2	Auto Rickshaw	
3	Scooter/Motor Bike	
4	Mini-bus (LCV)	
5	Bus (loaded, unloaded, overloaded)	Traffic loading analysis for pavement design
6	Truck	
7	Tractor with trailer	
8	Tractor without trailer (LCV)	Capacity analysis for geometric (cross section) design
9	Cycle	
10	Cycle Rickshaw/Hand Cart	
11	Horse Cart/Bullock Cart	
12	Pedestrian	

3.2.2.2 Origin-Destination (O-D) surveys

Origin-Destination (OD) surveys in case of rural roads are not normally required except for special areas such as industrail, business hubs etc. Such surveys can be undertaken using a variety of survey techniques. They are carried out to establish the nature of travel patterns in and around the area of enquiry and would normally be carried out as part of a regional planning exercise rather than for an individual road project.

3.2.2.3 Axle load surveys

Axle load surveys provide critical and essential information that is required for both cost-effective pavement design as well as preservation of existing roads, and are recommended particularly in areas where additional heavy vehicles are likely to use the new road. The importance of this parameter is highlighted by the well-known “fourth power law” which exponentially relates increases in axle load to pavement damage (e.g. an increase in axle load of 20 per cent produces an increase in damage of more than 200 per cent). Information about the loading of vehicles is essential for pavement design and for overload control. Simplified methods of acquiring vehicle load data are described below.

Full axle load surveys

The type of equipment which may be used for axle load surveys varies widely and includes:

- Static or dynamic weighing equipment.
- Manual or automatic recording of loads.
- Portable or fixed installation.

The quality of the data obtained will depend on the type of equipment used, the duration of the survey and the degree of quality control performed. In general, the higher the quality of the data, the greater will be the resources required to collect it. However, axle load surveys can be expensive and are unlikely to be undertaken for an individual LVR project for which simplified methods are required.

Simplified axle load surveys

If a full axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator merely records, for every heavy vehicle in the heavy vehicle classes, the state of loading (full, partial or empty), and the type of load (heavy, medium, or light).

These are particularly important in areas where the traffic carries bulk quantities such as sugar cane, brick-works, quarry areas, etc.

3.2.3 Procedure for Determining Design Traffic

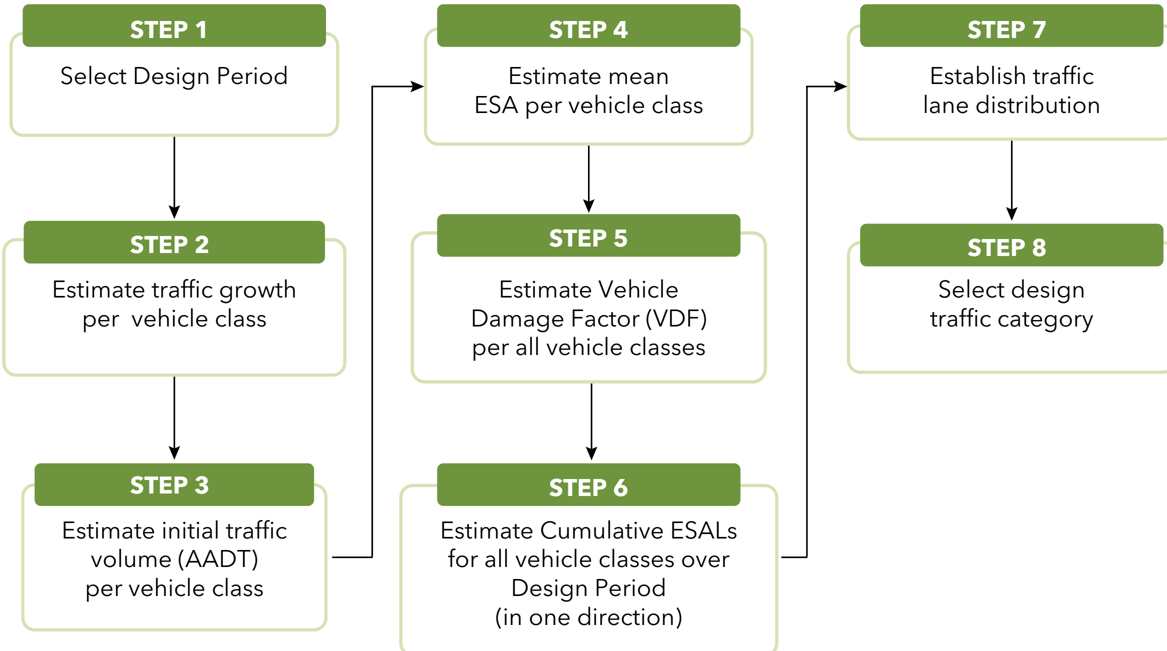
3.2.3.1 General

The procedure for determining the traffic loading for pavement design purposes is summarized in Figure 8.

3.2.3.2 Select Design Period

A structural design period must be selected over which the cumulative axle loading is determined as the basis of designing the road pavement. The design period is defined as the time span in years considered appropriate for the road pavement to function before reaching a terminal value of serviceability after which major rehabilitation or

FIGURE 8: PROCEDURE FOR ESTABLISHING DESIGN TRAFFIC CATEGORY



reconstruction would be required. It will be required to carry out a condition survey at least every 2 years, so that the nature and rate of change of condition will help identify as to when the pavement will require strengthening.

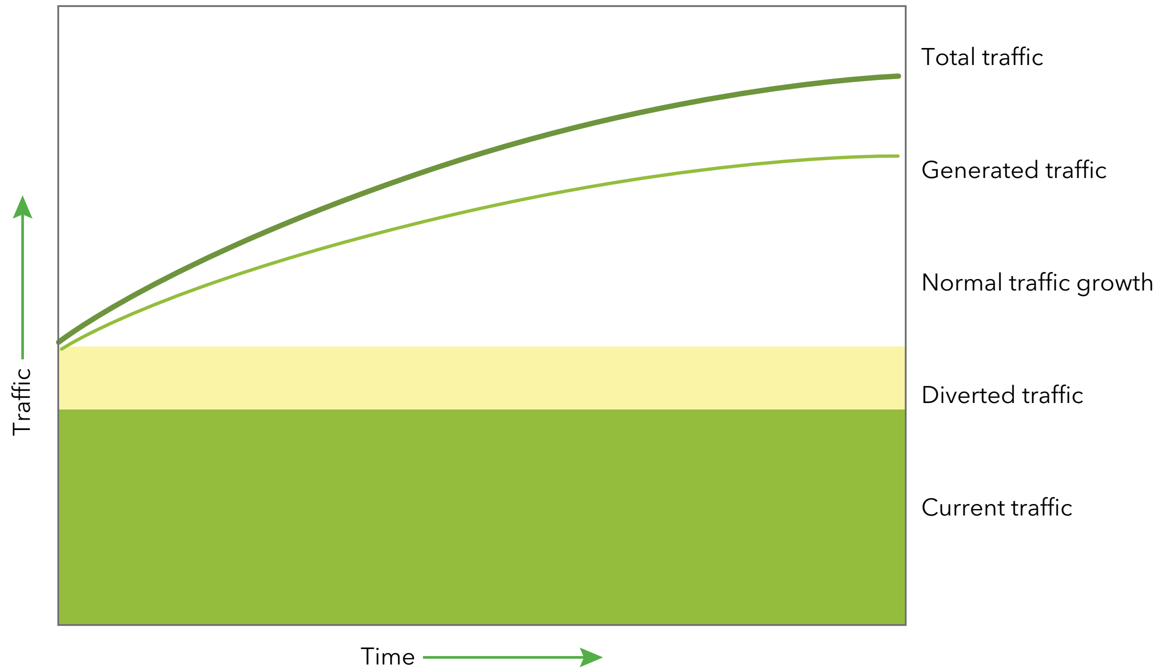
A design life of 10 years has been recommended to ensure that neither the strengthening will be needed to be carried out too soon nor will the design for a very long design period be unduly expensive by way of high initial investment required. This can go a long way in connecting more of the unconnected habitations with the same investment.

3.2.3.3 Estimate Traffic Growth per Vehicle Class

Following the establishment of the baseline traffic, further analysis is required to establish the total design traffic based on forecast of traffic growth in each vehicle class. To forecast such growth, it is first necessary to sort traffic in terms of the following categories (Figure 9):

- **Current traffic:** Existing traffic on the road before its upgrading to a higher standard.
- **Diverted traffic:** This is traffic that diverts from another route to the new road, because of the better riding condition on an existing route or shorter access on a new route, but still travels between the same origin and destination points. Unless origin-destination surveys have been carried out (very unlikely) this can only be estimated based on judgement of the traffic on nearby roads that could benefit from a shorter or more comfortable route. Local historic precedent can sometimes assist in estimating this, otherwise a rule of thumb is that diverted traffic is typically 20% of the existing traffic but it can be considerably higher.

FIGURE 9: TRAFFIC DEVELOPMENT OVER TIME ON A NEW OR IMPROVED ROAD [SCHEMATIC]



- **Normal traffic growth:** Traffic that results from the normal growth in current traffic after the road is upgraded.
- **Generated traffic:** This is the additional traffic that occurs in response to the new or improved road. This traffic is essentially created traffic resulting from increased economic activity in the area. Values for this of 6 or 7% have been routinely used in India.

Diverted traffic moves to the road quickly after completion of construction, while generated traffic is in response to the creation of new economic activities and opportunities arising from the improved or new road, and builds up over time.

Estimating traffic growth over the design period is very sensitive to economic conditions and prone to error. It is therefore prudent to assume low, medium and high traffic growth rates as an input to a traffic sensitivity analysis for pavement design purposes.

The growth rate of each traffic class may differ considerably. Motor cycles and motor cycle driven vehicle modes, for example, usually grow at a much faster rate than other traffic classes. Therefore, growth rate for all traffic classes should be considered.

There are several methods for estimating the traffic growth. IRC SP 72 suggests values but each one should be assessed on its own merit.

Local historic precedent

Evidence of traffic growth on roads recently upgraded in the area is a good guide as to what to expect.

Government predictions of economic growth

Economic growth is closely related to the growth of traffic. Economic growth rates can be obtained from government plans and government estimated growth figures. The growth rate of traffic should preferably be based on regional growth estimates because there are usually large regional differences.

It should be borne in mind that both geometric design classes and structural design classes are quite wide in terms of traffic range, typically a range of 100% or more, hence broad estimates of traffic projections would be sufficient in respect of low volume roads. A common method of choosing the design traffic is simply to estimate the initial traffic, including diverted and generated traffic, and to accommodate traffic growth by choosing the next higher traffic class for both geometric and structural design.

3.2.3.4 Estimate Initial Traffic volume (AADT) per Vehicle Class

Based on the traffic surveys, the initial traffic volume for each vehicle class can be determined. For structural design purposes, it is only the commercial vehicles (traffic classes 5 to 7 in Table 1) that will make any significant contribution to the total number of equivalent standard axles. In contrast, for geometric design purposes it is necessary to count non-motorized and intermediate means of transport including pedestrians, bicycles, motorcycles, tractors and trailers and, possibly, animal transport. Even for such cases, single lane carriageway would suffice for LVRs.

Taking account of the seasonal variations in rural traffic (Figure 7), the total number of repetitions (N) of a given vehicle type during a year, and the Average Annual Daily Traffic (AADT) are determined as follows [6]:

$N = T \times 365 + 2nT\,[0.6t]$

$AADT = T + 1.2nTt/365$

Where: T = Average number of commercial vehicles of a given traffic category per day during the lean season

t = duration of harvesting season

n = number of years

3.2.3.5 Estimate Mean ESA per Vehicle Class

For purposes of pavement design, only commercial vehicles with a gross laden weight of 3 tonnes or more along with their axle loading are considered. These may include inter alia the following:

- Trucks (heavy, medium)
- Buses
- Tractor-Trailers

The traffic parameter is generally evaluated in terms of a Standard Axle Load of 80 kN and the cumulative repetitions of the Equivalent Standard Axle Load (ESAL) are calculated over the design life.

Vehicles with single axle loads different from 80 kN, and tandem axles different from 148 kN can be converted into standard axles using the Axle Equivalence Factor, as follows:

Axle Equivalency Factor = $(W/W_s)^n$

Where W = Axle load in kN of the vehicle in question

W_s = Standard axle load of 80 kN for a single axle and 148 kN for a tandem axle

n = power exponent (lies between 2.5 and 4.5. A value of 4 is recommended for LVRs).

The likely average Equivalency Factors for converting the Standard Axle Load of 80 kN and the Tandem Axle Load of 148 kN are given in Table 2.

3.2.3.6 Estimate Vehicle Damage Factor

The Vehicle Damage Factor (VDF) is a multiplier for converting the number of commercial vehicles of different axle loads to the number of standard axle load repetitions. It is defined as the "equivalent number of standard axles per commercial vehicle". Whilst

TABLE 2: EQUIVALENCY FACTORS FOR DIFFERENT AXLE LOADS

Axle Load		Load Equivalency Factors	
(Tonnes)	kN	Single Axle	Tandem Axle Group
3.0	29.4	0.02	0.01
4.0	39.2	0.06	0.01
5.0	49.1	0.14	0.02
6.0	58.8	0.29	0.03
7.0	68.7	0.54	0.05
8.0	78.5	0.92	0.08
8.16	80.0	1.00	0.09
9.0	88.3	1.48	0.13
10.0	98.1	2.25	0.20
10.2	100.0	2.46	0.21
11.0	107.9	3.30	0.29
12.0	117.7	4.70	0.40
13.0	127.5	6.40	0.56
14.0	137.3	8.66	0.75
15.0	147.1	11.42	1.05
16.0	157.0	-	1.27
17.0	166.8	-	1.62
18.0	176.6	-	2.03
19.0	186.4	-	2.52
20.0	196.2	-	3.09

the VDF value is arrived at from axle load surveys on the existing roads, the project size and traffic volume in the case of rural roads may not warrant conducting an axle load survey. It may be adequate to adopt indicative VDF values discussed below for the pavement design.

For calculating the VDF, the following categories of vehicles may be considered:

1) **Laden Heavy Commercial vehicles (HCV)**
Fully loaded HCV (comprising heavy trucks, full-sized buses) have a Rear Axle Load of 10.2 tonnes and a Front Axle Load, about half the Rear Axle Load, i.e. 5 tonnes. The VDF works out to 2.60 (= 2.46 + 0.14).

2) **Unladen/Partially Loaded Heavy Commercial vehicles (HCV)**
Since the extent of loading of commercial vehicles is difficult to determine, a Rear Axle Load of 6 tonnes and a Front Axle Load of 3 tonnes may be assumed for an Unladen/Partially Loaded HCV. The VDF works out to 0.31 (= 0.29 + 0.02).

3) **Overloaded Heavy Commercial vehicles**
The extent of overloading may vary widely from one situation to the other. However, if an overload of 20% occurs, the VDF increases to 5.35 (= 5.06 + 0.29). However, if only 10% of the laden HCV are overloaded to the extent of 20%, the VDF works out to 2.86 (= 0.9 x 2.58 + 0.1 x 5.35).

4) **Laden Medium-heavy Commercial vehicles (MCV)**
Fully loaded MCV (mostly comprising Tractor-Trailers) have a Rear Axle Load of 6 tonnes and a Front Axle Load of 3 tonnes. The VDF works out to 0.31 (= 0.29 + 0.02).

5) **Unladen/Partially Loaded Medium-heavy Commercial Vehicles**
Since the extent of loading of commercial vehicles is difficult to determine, a rear Axle Load of 3 tonnes and a Front Axle Load of 1.5 tonnes may be assumed. The VDF works out to 0.019 (= 0.018 + 0.001).

6) **Overloaded Medium-heavy Commercial Vehicles**
The extent of overloading may vary widely from one situation to the other. However, if an overload of 20% occurs, the VDF increases to 0.65 (= 0.61 + 0.04). However, if only 10% of the laden MCV are overloaded to the extent of 20%, the VDF works out to 0.344 (= 0.1 x 0.65 + 0.9 x 0.31).

Towards the computation of ESAL applications, the indicative VDF values (i.e. Standard Axles per Commercial vehicle) are given in Table 3 below.

For pavement design purposes, the number of:

1. HCV: Laden, unladen and overloaded.
2. MCV: Laden, unladen and overloaded.

TABLE 3: INDICATIVE VDF PER VEHICLE TYPE

Vehicle Type	Laden	Unladen/Partially Laden
HCV	2.86	0.31
MCV	0.34	0.02

Must be obtained from actual traffic counts and, using appropriate VDF values, the number of Equivalent Standard Axles to be catered for over the design life can be determined. If, however, for some reason, it is not possible to carry out all the required traffic counts, recourse to local enquiries may be taken to estimate their proportions in as realistic a manner as possible.

3.2.3.7 Estimate Cumulative ESALs for all Vehicle Classes Over Design Life

The estimated ESALs for all vehicle classes over the design life of the road may be calculated as follows:

$$N = T_0 \times 365 \times \left[\frac{(1+0.01r)^n - 1}{0.01r} \right] \times L$$

$$= T_0 \times 365 \times \left[\frac{(1+0.06)^{10} - 1}{0.06} \right] \times L$$

$$= T_0 \times 4811 \times L$$

where T_0 = ESAL per day = number of commercial vehicles per day in the year of opening x VDF

L = lane distribution factor (see Step 3.2.3.8)

3.2.3.8 Establish Traffic Lane Distribution

The actual ESALs for all vehicle classes over the design life of the road need to be corrected for the distribution of heavy vehicles between the lanes in accordance with Table 4.

TABLE 4: LANE WIDTH ADJUSTMENT FACTORS FOR DESIGN TRAFFIC LOADING [13]

Cross Section	Paved width	Corrected design traffic loading (ESA)	Explanatory notes
Single lane carriageway	3 m, 3.75 m	The sum of ESAs in both directions	Traffic in both directions uses the same lane
Intermediate lane carriageway	5.5 m	80% of the ESAs in both directions	To allow for overlap in the centre section of the road

3.2.3.9 Select Traffic Category

For pavement design using the DCP method, the traffic has been divided into 6 categories as shown in Table 5.

TABLE 5: TRAFFIC CATEGORIES

Traffic Category	Cumulative ESALs (x 10^6)
T_1	< 0.010
T_2	0.010 – 0.030
T_3	0.030 – 0.100
T_4	0.100 – 0.300
T_5	0.300 – 0.700
T_6	0.700 – 1.000

3.3 ROUTE SURVEY

3.3.1 General

The successful design of a road depends on ensuring that the pavement is appropriate for the characteristics of the subgrade or the embankment on which it is placed. A good subgrade is strong enough to resist shear failure and has adequate stiffness to minimize vertical deflection. The stronger the subgrade, the thinner the pavement layers above need to be and the lower the cost of the road will be. The designer usually has little choice about the subgrade except for when a raised formation or embankment is constructed. Therefore, it is essential that the characteristics of the subgrade along the alignment are carefully assessed and understood. In cases where the subgrade materials are unsuitable, cost-effective methods of improving these materials must be identified, e.g. stabilization or improving drainage (Sections 3.4 and 3.5).

Initial reconnaissance surveys consisting of a desk study and walk-over survey should always be carried out prior to the actual survey. Issues that should be noted include:

- General soil types along the route.
- Moisture and drainage conditions.
- Variations in terrain and potential for effective side and cross drainage.
- Existing embankments heights, effectiveness against flooding and necessity, if any, for raising embankments.
- Road reserve status.
- Expected relevance of a DCP survey.

When upgrading an existing track or road, it is equally important to determine the characteristics of the existing layers of material in the pavement structure because these should be utilized in the new pavement. The most cost-effective method of obtaining sub-surface information at small intervals along the entire route to a depth of approximately 800 mm is by using a Dynamic Cone Penetrometer (DCP).

3.3.2 DCP surveys

The DCP is light and portable and tests are quick and simple. The advantage of the DCP is that information can be gathered with minimal disturbance to the in-situ material. Using this test, the strength characteristics and thickness of the in situ subsurface materials at their prevailing field moisture and density conditions are obtained directly. The DCP also has the advantage over the CBR in providing a continuous strength profile over a depth of 800 mm at much smaller intervals along the road.

The required frequency of the DCP measurements depends on the variability in conditions along the route and the level of confidence required. Where obvious changes of surface conditions occur, the frequency of the tests should be modified to include the changes. Similarly, where surface conditions are uniform, the frequency of testing may be reduced. A guideline for the frequency of testing for upgrading an existing road to a paved standard is shown in Table 6.

TABLE 6: FREQUENCY OF DCP TESTING

Road condition	Frequency of testing (number/km)
Uniform, fairly flat, reasonable drainage - low risk	5
Non-uniform, rolling uneven terrain, variable drainage - medium risk	10
Distressed, uneven terrain, poor drainage – high risk	20

Several different correlations exist between the DCP penetration rate (mm/blow) and the more familiar CBR strength. These correlations are based on CBR values versus DCP penetration rates measured in different soil types and are generally material and test method dependent with correlation coefficients ranging from 0.67 – 0.79 [14]. It is thus recommended, however, that the actual DCP penetration rates (direct indications of the shear strength) are utilized in the pavement design instead of the more variable/less reliable CBR [15]. This approach is discussed further in Chapter 4 – Pavement Design.

The general procedure for undertaking the DCP survey as part of the overall design process differs between that for new roads and for existing roads, and is discussed separately for these two situations.

3.3.2.1 Existing roads

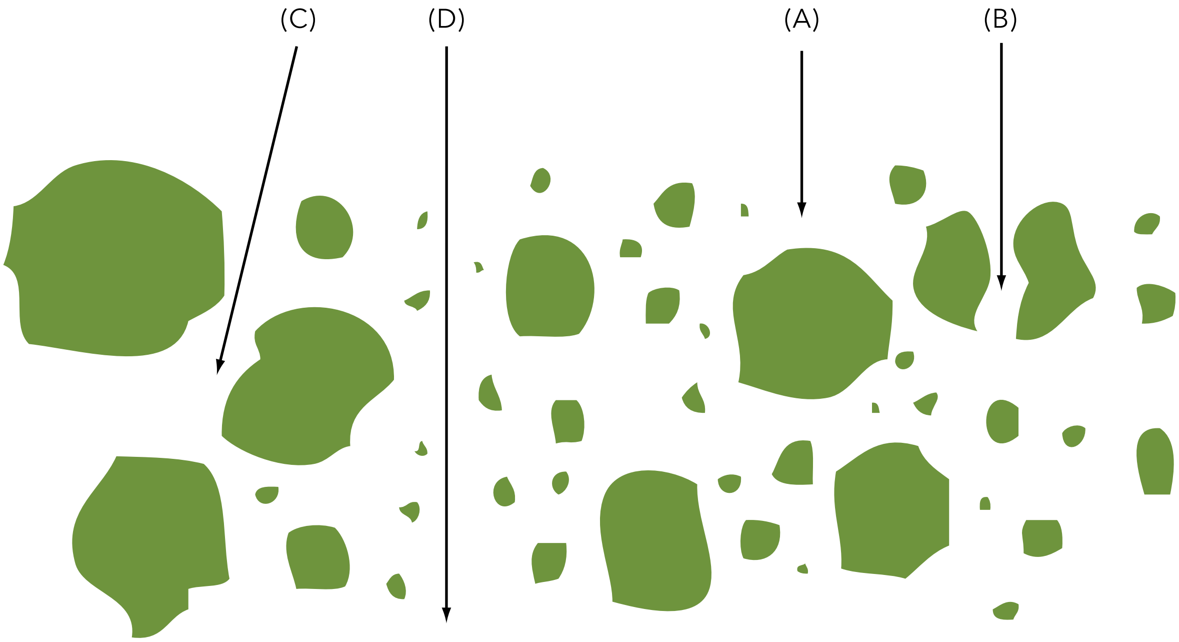
On existing roads and tracks, the new road is expected to be built directly on the material currently forming the road or track, and the DCP test will indicate the properties of the materials that will be a part of the new pavement structure. Even if the formation is raised slightly to facilitate drainage or pipe cover, the materials tested will usually influence the structural capacity of the new pavement structure.

The DCP survey must thus be carried out along the full length of the road with each measurement being taken to a depth of at least 800 mm, in order to assess the balance of the final pavement. The DCP tests should be staggered across the road at left outer wheel-track, centre line, right outer wheel-track, centre line, etc. However, the variability of the road will only be fully apparent once the tests have been carried out. To ensure statistical reliability, at least 10 DCP tests should be carried out in each uniform section (see Chapter 4), hence additional tests may be required after analysing the first set of data obtained.

Care must be exercised in carrying out the DCP survey to discard any measurements that could produce anomalous results. Such results could arise, for example, where large stones occur in the pavement layer (Figure 10). Where brick-soling or hard aggregate layers (e.g. water bound macadam) already comprise a part of the pavement, this layer should be removed and the DCP test carried out from the base of the layer, after measuring the thickness of this layer. This layer would then be included in the pavement design as an existing strong layer.

The data should be captured in the form of the number of blows of the DCP hammer and the corresponding depth of penetration and can be entered directly into a spreadsheet for use in available software. Destructive testing (sampling) is not required at each DCP test site.

FIGURE 10: DCP EFFECTS WHERE LARGE STONES ARE PRESENT



(a) Cone cannot penetrate.	(b) Cone breaks stone. DCP profile shows a plateau and subsequent readings may be low.	(c) Rod pushed aside and tilts at an angle. Excessive friction on rod gives low reading.	(d) Normal result.

3.3.2.2 New roads

The construction of new roads can result in two processes. In some cases, the new road will be at or slightly above natural ground level, in which case the DCP survey can be carried out along the alignment (as described above in Section 3.3.2.1). However, if the road is to be constructed on a new fill or embankment higher than about 500 or 600 mm, a DCP survey at the existing ground level will have little input into the pavement design or influence on the final pavement performance. The strength of the compacted embankment material needs to be determined as described above as this is what will influence the structural design of the pavement.

In some cases, the DCP can be used at natural ground level to identify potential drainage and moisture problems beneath embankments that will be up to about 1 m or more in height, which may influence the later stability or settlement of the embankment. However, DCP testing of the in situ material at the prevailing moisture and density is generally meaningless for such embankments. Instead, testing of samples of the material in the laboratory, after compaction to the expected density and at the expected moisture content, can be used to determine whether the local material can be used for construction of the embankment.

It is also possible, if the proposed embankment borrow source has been identified, to test this material in the laboratory and determine the expected embankment (subgrade) profile.

3.3.3 Test pits and Sampling

The structural assessment requires some test pits along the road to obtain samples for laboratory testing to assist with material characterization. These are also used to allow direct observations of the in-situ subgrade soil and potential fill material.

The location, frequency and depth of pits and trenches for characterizing the subgrade depend on the general characteristics of the project area (the soil type and variability). In addition, the DCP testing carried out to assist delineation of uniform sections can be used to target areas for pitting and trenching. Spacing will decrease when the subsurface soils demonstrate more variability. In these areas, pits can also be staggered left and right of the centre line to cover full width of the road formation. It is necessary to ensure that sufficient test pits are excavated in each uniform section to provide sufficient data for the entire uniform section. This needs to be carried out in conjunction with analysis of uniform sections using the DCP data (Chapter 4). Requirements are different for existing roads and new alignments.

3.3.3.1 Existing roads

Test pits should be excavated along existing roads at the required intervals to provide sufficient information per uniform section to a depth of at least the bottom of the imported selected layers in the pavement. Samples of the material in each layer should be collected for laboratory testing and an inspection and profiling of the pit carried out to identify the materials and the causes of any failures or problems.

3.3.3.2 New roads

The depth of pits and trenches is determined by the nature of the subsurface. For sampling and soil description, pits should be dug to at least 0.5 m below the expected natural subgrade level. In cut sections, the depth can be reduced to 0.3 m but in potentially problematic materials (see following section), the depth may need to be extended to at least 1.0 m below the proposed subgrade. Greater depths may also be needed for high embankment design. A limited number of deeper pits may also be needed to ascertain groundwater influence and irregular bedrock, if data in this regard is unknown, although it can often be obtained from other sources.

The location of each test pit should be determined precisely on the route alignment to cover the full range of in situ conditions and all layers, including topsoil, should be accurately described and their thicknesses measured. All horizons, below the topsoil should be sampled. This will promote a proper assessment of any materials excavated in cuts that could be used in embankments. The samples should be taken over the full depth of the layer by taking vertical slices of materials.

3.3.4 Moisture and Drainage

3.3.4.1 Assessment of moisture conditions along alignment

It is essential that an estimate of the in situ moisture condition is made at the time of the DCP survey for comparison with the expected moisture regime in service. This moisture regime affecting the in situ materials must be accurately assessed in terms of whether the road will operate at that condition or in a wetter or drier state than at the time of assessment during its service life. This will be used later to statistically determine the percentile of the DCP penetration rates for pavement design purposes.

In addition, at least 2 samples should be collected per kilometre of the proposed subgrade materials for moisture content and Optimum Moisture Content (OMC) determination from the outer wheel tracks of the road at depths of 0-150, 150-300 and 300-450 mm. This is best done during the test pitting and synchronisation of the identification of uniform sections and test pitting needs to be done as soon as possible after the DCP survey to ensure that samples are representative of all the uniform sections.

3.3.4.2 Local drainage problems and requirements

During the survey, any drainage problems or constraints that would affect drainage locally (streams, marshy areas, flat poorly drained areas, etc.) need to be identified so that areas requiring specific side or cross drainage can be pin-pointed for the design.

Cognisance should also be taken of the potential climate change effects in the long term, particularly in terms of drainage structures. Predictions indicate that the annual rainfall will increase over most of India in the long term, with more frequent extreme events, leading to potentially more flooding situations in certain areas. It is recommended that localised information is obtained from the relevant authorities (e.g. Indian Institute of Tropical Meteorology) regarding expected climate changes in areas being investigated. Areas that are visibly prone to possible flooding and water accumulation under extreme precipitation or flooding conditions must be noted, as soaked designs would be necessary in these areas.

3.3.5 Problem subgrades

Many subgrades may be classified as problematic materials. These include a wide range of possible materials such as:

- Expansive/heaving clays (“cotton soils”)
- Wet/waterlogged areas
- Collapsible soils
- Dispersive soil
- Erodible soils
- Saline soils
- Soft clays

Each of these potentially problematic soils requires unique investigation and test protocols. During the site investigation, it is important that such problem areas are identified and suitable advice be obtained regarding the implications and treatment from geotechnical specialists where it is felt necessary. The fact that most of these are affected by moisture fluctuations is also relevant to long-term climate changes. A summary of problem soil causes, recognition and treatment follows:

3.3.5.1 Expansive Clays

Causes

Expansive clays are widespread and of major economic significance. Typical damage to roads includes longitudinal unevenness and bumpiness, differential movement near culverts and longitudinal cracking along the road. The presence of trees alongside the road often results in localized moisture extraction by their roots with the development of sporadic subsidence and arcuate cracking. Expansive clay damage to roads usually affects their serviceability more than their structural integrity, provided cracking and surface distress is timely and effectively repaired. Damage is generally restricted to areas that have significant seasonal rainfall or poor surface water drainage.

Expansive soils are those containing smectite (montmorillonite) clays, which are mostly derived from the chemical weathering of basic rock forming minerals. Probably the worst expansive clay subgrades are in areas of deeply weathered gabbros, basalts and dolerites in tropical and sub-tropical zones. Expansive clays are also commonly found in transported soils derived locally or from some distance from weathered basic igneous rocks. Smectites can also form from the alteration under alkaline conditions of other silicate minerals low in potassium, as long as calcium and magnesium are present and leaching is impeded. Although the expansive potential of a soil can be related to many factors, it is primarily controlled by the quantity and type of clay minerals (e.g. smectites).

Volume changes in expansive soils are confined to the upper few metres of a soil deposit where seasonal moisture content varies due to drying and wetting cycles. The zone within which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the in situ moisture content with depth for samples taken during the wet and dry seasons. The depth at which the moisture content shows no seasonal variation is the limit of the active zone. This is also referred to as the depth of seasonal moisture change, but could change in the long-term due to climate changes.

Recognition

The simplest way of identifying the presence of expansive soils is through field observations where the surface expression of cracking in dark grey, black or sometimes red soils is evident as shown in Figure 11. However, the presence of a thick non-expansive transported or topsoil cover can sometimes mask these cracks and excavation of a test pit, in which cracking and slickensiding of the material will be observed is necessary. The identification of smectite in subgrade soils is best done using X-ray diffraction.

By their nature, smectites will tend to be more plastic than other clay minerals and a measure of the Plasticity Index, or better still the activity (ratio of Plasticity Index to clay fraction) is a good indication of the presence of smectites. This is one of the earliest methods of indicating potentially expansive soils using Figure 12 based on the clay fraction of the soil (minus 2 μm) and the standard Plasticity Index (PI), which remains a simple but very useful means of identifying expansive soils. It should be noted that the estimates for the degree of swell using this technique do not consider the initial moisture content of the material, but assumes that they move from a state of dryness normally used in the laboratory to wet. It is known that an equilibrium moisture content develops

FIGURE 11: CRACKING ON EXPANSIVE CLAY

under a road structure and the moisture fluctuation in this zone is minimal. However, from beneath the outer wheel track of roads with unsealed shoulders to the edge of the fill, significant and variable moisture fluctuations occur. It is unlikely that the initial moisture content in these zones is, however, particularly dry.

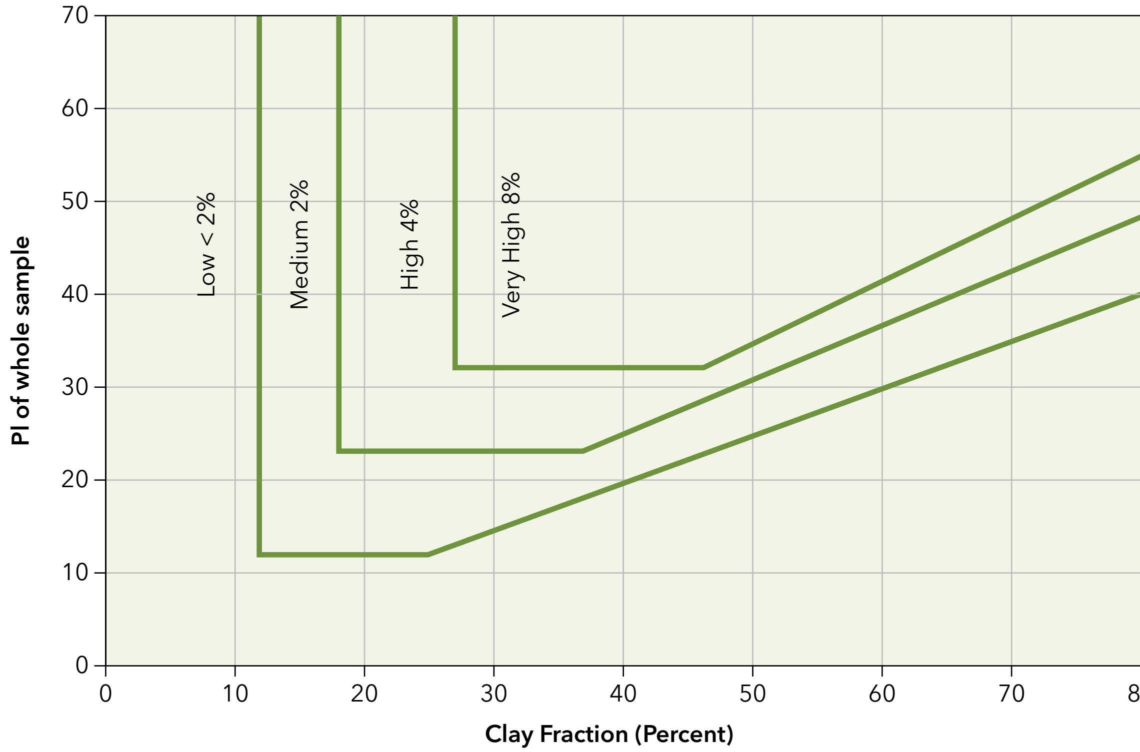
An indication of potentially expansive soils can also be obtained from land type soil maps where materials identified as "vertic" soils will always have expansive characteristics, while soils with a high base status (or eutrophic) and clay content should be investigated more thoroughly, as they have the potential to be expansive.

Countermeasures

Although the estimation of potential heave is imperative for structures on expansive clay, it is not as critical for subgrades under roads, particularly LVRs. It is more important to identify the possible existence of the problem and the potential for differential heave along the road and take the necessary precautions. These will generally be based on the expected degree of swell determined from Figure 12.

If the calculated potential heave exceeds 25 to 50 mm, countermeasures should be installed. If there is likely to be significant differential movement because of variable material properties or thicknesses, changing loading conditions or localized drainage differences, the countermeasures will need to take this into account to avoid localized sections of road with poor riding quality.

FIGURE 12: IDENTIFICATION OF EXPANSIVE CLAY SOILS AND ESTIMATE OF EXPANSION [16]



Where culverts or small bridge structures are involved, it is usually necessary to quantify the potential movement more accurately. This is best done using oedometer testing of specimens cut from block samples. Correct orientation of the block samples is imperative as expansive clays tend to be highly anisotropic with significantly lower swells in the horizontal direction. This testing needs to be carried out in conjunction with good estimates of the potential changes in *in-situ* moisture content from season to season.

Solutions that can be considered for LVRs over expansive clays include:

1. Flattening of side slopes (between 1V:4H and 1V:6H).
2. Remove expansive soil and replace with inert material (between 0.6 and 1 m depending on depth of clay).
3. Retain the road over the clay as an unpaved section.
4. Pre-wetting prior to construction of the fill or formation (to OMC).
5. Placing of un-compacted pioneer layers of sand, gravel or rockfill over the clay and wetting up, either naturally by precipitation or by irrigation (300 to 500 mm depending on clay thickness and potential swell).
6. Lime stabilization of the clay to change its properties (expensive as up to 6% lime may be required).
7. Blending of fine sand with the clay to change its activity (blend ratio to be determined by laboratory experimentation).
8. Sealing of shoulders (not less than 1 m wide).

9. Compaction of thin layers of lower plasticity clay over the expansive clay to isolate the underlying active clays from significant moisture changes.
10. Use of waterproofing membranes and/or vertical moisture barriers, which are generally geosynthetics (only limited success has been achieved using these methods.

Figure 13 provides a preliminary indication of possible counter-measure options (numbered as above) as a function of potential expansiveness. It should be noted that usually a combination of these is most effective and all should go together with careful design and construction of side-drains, which should preferably be sealed.

In many cases for roads with less than 20 vpd and swells higher than 4%, it may be better to retain the road as a gravel road over the expansive clay sections and apply the necessary maintenance.

One of the most important considerations is to try and minimize the zone of seasonal moisture movement beneath the road (Figure 14) and increase the zone of moisture equilibrium. A combination of slope flattening, material replacement, sealed shoulders and lined side drains is usually the most cost-effective means of achieving this, but the design of counter-measures needs to be specific to any situation. This is a particular problem on narrow roads (e.g. 3.75 m, where only the central 1 or 1.5 m achieves equilibrium.

Expansive clays are often thick and laterally widespread and this makes the implementation of countermeasures costly. The most successful technique for counteracting subgrades susceptible to high movement is to remove the expansive clay beneath the road structure and replace it with a raft of inert material. This would typically involve the excavation and removal of between 600 and 1 500 mm (or even

FIGURE 13: POSSIBLE SOLUTIONS FOR THE CONSTRUCTION OF ROADS ON ACTIVE CLAYS

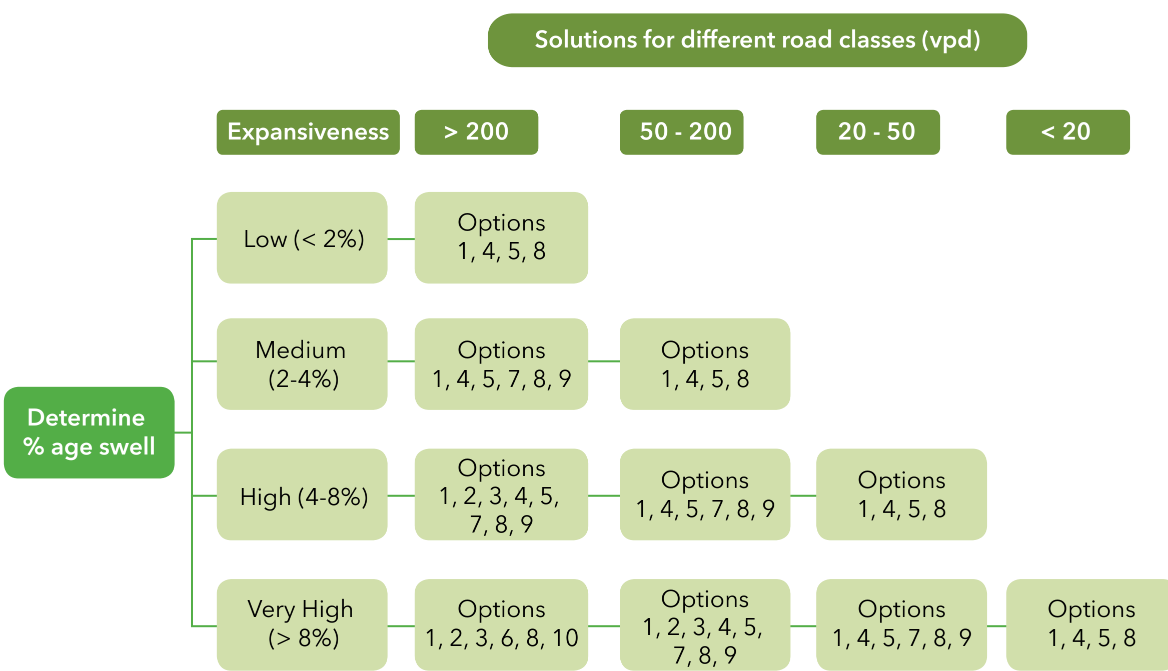
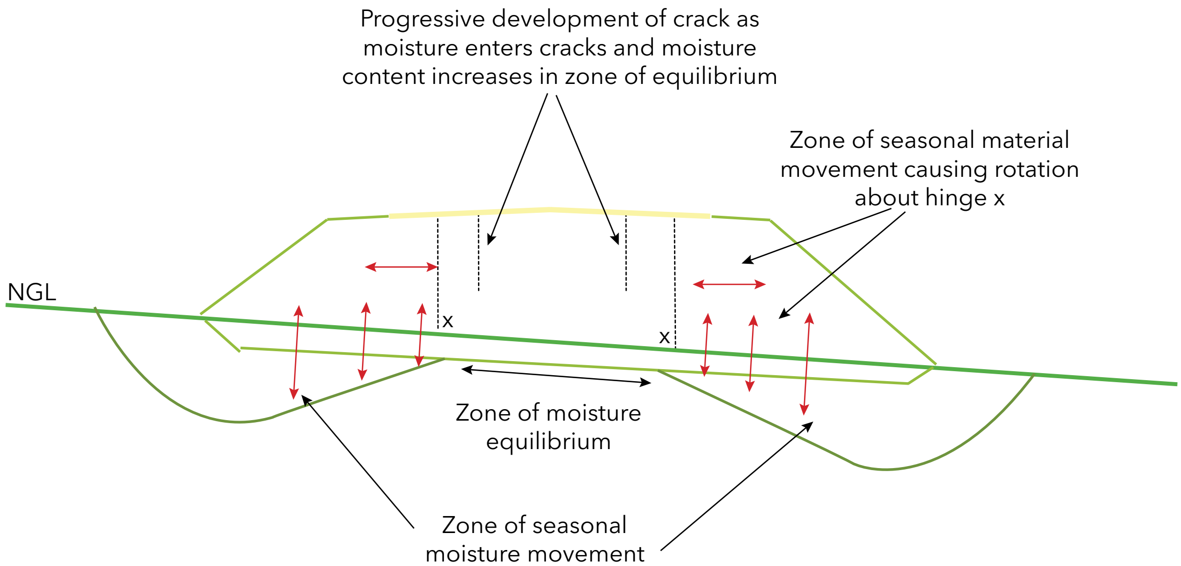


FIGURE 14: TYPICAL MOISTURE MOVEMENT REGIME UNDER ROADS ON EXPANSIVE CLAYS



deeper in some cases) of material over the entire footprint of the road prism (or at least beneath shoulders and side slopes) combined with drainage structures that remove all water from adjacent to the fill slopes and culverts. Removal of material results in the reduction of the swell potential as well as slightly increasing the load on the expansive subgrade with a usually denser, better compacted material. Unfortunately, this is often impracticable or uneconomic for low volume roads, unless the problem is localized. More frequently, expansive materials cover a wide area and the importation of substitute material involves the haulage of large quantities of inert material over long distances.

The recommended and probably most economical solution specifically for low volume roads showing high to very high potential swell is to partially remove the clay from the upper subgrade (250 mm) and replace it with a less active material, increase the fill height using inactive material to provide a greater load on the underlying clay, seal the shoulders of the road and flatten the fill slopes using the material removed from the subgrade and side drains. This has the effect of moving the zone of seasonal moisture fluctuation away from the pavement structure and inducing movements and cracking in the more flexible fill slopes rather than in the stiffer pavement structure.

Particular attention should be paid to culverts. The clay beneath them must be replaced with an inert material, all joints must be carefully sealed to avoid leakage and inlets and outlets well graded to avoid ponding of water. It is essential, however, that a proper understanding of the potential moisture movements in and around the road is obtained and this is related to the swell potentials of the various pavement materials (fill, shoulders, subgrade, etc.).

It is also good practice to remove and control the re-establishment of "water loving" trees. The roots of such trees seek water beneath the pavement and remove it from the clay, causing significant depressions in the road during the dry season, which may or may not recover in the wet season. This is usually associated with arcuate and/or longitudinal cracking.

3.3.5.2 Dispersive/erodible/slaking materials

Causes

Dispersive, erodible and slaking materials are similar in their field appearance (highly eroded, gullied and channelled exposures), but differ significantly in the mechanisms of their actions. Fortunately for road builders, only the (probably less common) dispersive soils present problems of any consequence. Figure 15 shows a typical dispersive soil in an embankment with definite evidence of piping.

FIGURE 15: DISPERSIVE SOIL SHOWING FORMATION OF "PIPES"

Dispersive soils are those soils that, when placed in water, have repulsive forces between the clay particles that exceed the attractive forces. This results in the colloidal fraction going into suspension and in still water staying in suspension (Figure 16). In moving water, the dispersed particles are carried away. This obviously has serious implications in earth dam engineering, but is of less consequence in road engineering except when used in fills. Dispersive soils often develop in low-lying areas with gently rolling topography and relatively flat slopes. Their environment of formation is also mostly characterized by an annual rainfall of less than 850 mm.

FIGURE 16: DISPERSIVE SOIL (CRUMB) TEST SHOWING SUSPENSION THAT DOES NOT SETTLE OUT

Erodible soils will not necessarily disintegrate or go into dispersion in water. They tend to lose material because of the frictional drag of water flowing over the material that exceeds the cohesive forces holding the material together.

Slaking soils disintegrate in water to silt, sand and gravel sized particles, without going into dispersion. The cause of this process is probably a combination of swelling of clay particles, generation of high pore air pressures as water is drawn into the voids in the material and softening of any incipient cementation.

Slaking and erodible soils when occurring as subgrades or even when used in fills are unlikely to cause significant problems unless rapid flows of water through the fill or subgrade occur. Problems are thus mostly associated with poor culvert and drainage design. The inclusion of dispersive soils in the subgrade or fill on the other hand has been seen to lead to significant failures through piping, tunnelling and the formation of cavities in the structure. It is therefore important to identify dispersive soils timely.

Recognition

The testing and recognition of dispersive soils requires various soil engineering and pedological laboratory tests. These include:

- Determination of the Exchangeable Sodium Percentage (ESP)
- Pinhole test
- Cation Exchange Capacity (CEC)
- Crumb test
- Double hydrometer test
- Sodium Absorption Ratio (SAR) and the pH.

The crumb test on undisturbed lumps of material is usually the best first indication, but is not always fool proof. Dispersive soils tend to produce a colloidal suspension or cloudiness over the crumb/lump during the test, without the material necessarily disintegrating fully. Disintegration of the crumb in slaking soils is very rapid and forms a heap of silt, sand and gravel. Erodible soils do not necessarily always disintegrate in the crumb test as they require a frictional force of moving water to loosen the surface material, without any of the loose material remaining in suspension.

It is not very important (or even possible) to quantify the actual potential loss of dispersive material from subgrades and fills as the process is time related and given enough time, all the colloidal material could theoretically be dispersed and removed, leading to piping, internal erosion and eventually loss of material on a large scale. It is, however, important to identify the presence of dispersive soils, and their differentiation from erodible and slaking materials, so that the necessary precautions can be taken if they affect the constructed pavement.

Countermeasures

The countermeasures for avoiding dispersive soil damage in the road environment are relatively simple:

- Avoid its use in fills as far as possible.
- Remove and replace it in the subgrade.
- Manage water flows and drainage in the area well.

As the presence of sodium as an exchange cation in the clays is the major problem, treatment with lime or gypsum will allow the calcium ions to replace the sodium ions and reduce the problem. The use of gypsum is recommended over lime as lime may lead to soil stabilization with its associated cracking, allowing water to move through the cracks.

It is also important that the material is compacted at 2 to 3% above optimum moisture content to as high a density as possible.

To avoid problems with slaking and erodible soils, the drainage must be well controlled. Covering of the soils with non-erodible materials and careful bio-engineering, assisted by geosynthetics where necessary, is usually effective. Once erosion has occurred, the channels and gullies should be back-filled with less erodible material and the water flows redirected.

3.3.5.3 Saline Soils

Causes

Unlike dispersive soils that are affected by the presence of excessive cations of sodium attached to clays, saline materials are affected by the combination of specific cations and anions in the form of soluble salts, independent of clays. These can be a major problem on road projects where migration of soluble salts to beneath bituminous surfacings (Figure 17) leads to weakening of the upper base and blistering and disintegration of the surfacings. Soluble salts, particularly sulphates, and their acids can also have a serious detrimental effect on the stability/durability of chemically stabilized materials and concrete.

Soluble salt damage to roads has been reported primarily from arid, semi-arid and warm dry areas. Salts can originate from the in situ natural soils beneath the structures as well

FIGURE 17: MECHANISM AND MANIFESTATION OF SOLUBLE SALT DAMAGE TO BITUMINOUS SURFACINGS

as from imported material for the pavement layers or from saline construction water. Only the presence of soluble salts in subgrade materials is considered in this report as the materials for other layers can be controlled provided the problem is identified timely.

Subgrade materials in areas where the land surface shows some depression resulting in seasonal accumulation of water are particularly prone to the accumulation of salts leached from the surrounding areas. In other flat areas, capillary rise of groundwater and precipitation in saline soils can result in the upward migration of salts to or near the soil surface.

Recognition

In some cases, the visible presence of crystallized salt deposits at the soil surface is a certain indication of the need for additional investigation for possible salt problems. This is often associated with the presence of animals licking the soil surface. In most other cases, the presence of salt is best confirmed by using laboratory test methods.

In the conventional road engineering context, the identification of possible soluble salt problems is based on the pH and conductivity of the materials. Most roads departments do not differentiate between the subgrade materials and the imported layer materials.

It should be noted that the results of the electrical conductivity and pH tests can vary significantly depending on the pre-treatment, the moisture content at which the measurements are made and particularly on the material size fraction tested.

Limits for the use of saline materials are generally based on work in specific countries and their applicability to other areas is unknown. In general, an electrical conductivity on the fraction passing 6.7 mm of more than 0.15 Sm^{-1} (or an electrical resistance of less than 200 Ω on the minus 2 mm fraction) should raise concern and indicate the need for further investigation. Similarly, soluble salt contents higher than 0.5% should be a cause for possible concern and need for additional investigations.

Countermeasures

The following measures should be considered:

- As soluble salt problems arise from the accumulation and crystallization of the salts under the road surfacing and in the upper base layer, minimization of salts in the pavement layers and subgrade should be attempted.
- If the surfacing is sufficiently impermeable (coefficient of permeability, k in nanometre/second)/surfacing thickness, T in mm or k/T < 30 (μsec)-1) to avoid water vapour passing through it, crystallization will not occur beneath the surfacing.
- Construction should proceed as fast as possible to minimize the migration of salts through the layers. Only impermeable primes should be used, e.g. bitumen emulsions.
- The addition of lime to increase the pH to more than 10.0 will also suppress the solubility of the more soluble salts.

Even for the lowest classes of road (< 50 vpd), the effects of excessively saline materials can lead to a rapid and total loss of the bituminous seal and precautions should thus be taken for all road classes. The use of non-bituminous surfacings should be considered over saline materials.

3.3.5.4 Soft Clays

Causes

Widespread problems, mostly in estuarine (lagoon) and marshy areas result from the presence of very soft alluvial clays in these areas. Deep soft clays in estuarine areas are formed mostly by periodic fluctuations in sea level. Inland soft clays tend to be much shallower having been deposited in marshy areas. Soft clays are generally, but not necessarily saturated and normally consolidated to lightly over-consolidated (because of fluctuating water tables). The materials thus have low shear strengths, are highly compressible and their low permeabilities result in time-related settlement problems. In addition, the frequent occurrence of organic material in the clays affects their behaviour and the determination of their properties.

These materials are present predominantly in the coastal areas although they can also be associated with large mature river systems. The shear strength of these clays would normally be between 10 and 40 kPa, making them difficult to walk on. Soft clays are seldom uniform with depth and are usually interlayered with silts and sands, which provide more permeable drainage paths than would be determined from oedometer testing of undisturbed clay samples. However, the depths and strengths of the materials are such that inspection of the materials in test pits or auger holes is not recommended.

Recognition

The in situ condition of these materials is one of their most important properties that need to be considered – testing of disturbed samples will usually provide results that are meaningless. It is thus better to use in situ test methods such as Standard Penetration Testing (SPT), vane shear or Cone Penetration Testing (CPT) to determine the depths, presence of silt or sand layers, strengths and if possible, permeabilities. If these can be identified to a reasonable degree of confidence, estimates of the quantity and rate of settlement and the potential stability of embankments over the materials can be made.

Countermeasures

Road embankments built on soft clays thus need careful control during their construction to avoid stability failures as pore water pressures increase under the applied loads. It is recommended that embankments in these areas are constructed slowly, layer by layer, while monitoring pore water pressures and additional layers are only added once the pore water pressures have dissipated adequately. Despite even these measures, long-term settlement continues and problems are often encountered with large differential settlements between the approach fills founded on the clays and bridges founded on piles. These long-term differential settlements require ongoing maintenance to provide an adequate performance of the road.

The use of the wide range of geosynthetic products as separation layers and to facilitate and accelerate drainage has contributed to improved construction over such areas in the past decade or two, and specialist advice in this respect should be obtained.

3.3.5.5 Wet and Flooded Areas/High Water Tables

Causes

It is possible that some non-clayey areas have a water table (permanent or temporary) close to the natural ground surface, which makes the placement of road structures difficult and can affect their structural integrity. Unlike the clay areas, the problem is not the low strength or settlement potential, but the effect of the water (and high pore-water pressures under traffic loading) on the pavement structure. Flooding has a similar effect when the road is open to traffic while still wet.

High water tables result in a steady, high in situ moisture but it is also possible that fluctuating high moisture content conditions within the pavement sub-structure may occur because of seasonal precipitation. A good understanding of the moisture conditions and environment needs to be defined during any investigation involving subgrade materials.

Various moisture indices such as Thornthwaite's Moisture Index or water surplus maps can provide very useful information on potential problems in this regard. Many of the problems encountered in roads are common to specific moisture zones, and these have been highlighted under their respective headings in this document.

Recognition

It is usually easy to recognize potential wet conditions, which are characterized by areas of standing water, specific types of vegetation (reeds, papyrus grasses, etc.), localized muddy conditions and often the presence of crabs and frogs. Reports of seasonal flooding conditions will normally be available from local communities.

Countermeasures

The treatment of wet areas for roads can be costly if the aim is to reduce the water tables using sub-surface drainage systems. These would seldom be warranted for low volume roads, because of the cost and the ongoing need to maintain them diligently. However, in cases where they are essential, they should be designed by a drainage/ground-water specialist.

The only cost-effective measures for low volume roads are to raise the level of the road to at least 750 mm above the natural ground or expected flood level, with a permeable gravel or rock fill layer (at least 100 to 150 mm thick) on the natural formation (after removal of the topsoil and vegetation). Properly designed and graded side drains should also be constructed to avoid the presence of standing water adjacent to the road.

3.3.5.6 Collapsible Soils

Collapsible soils result from a unique condition in which "bridges" of fine materials (usually clays or iron oxides) within a framework of coarser and harder particles (mostly quartz) become weak when wet and collapse under load. The important condition is that the material must be in a partially saturated condition and then wetted up and loaded simultaneously, which is a common situation beneath road structures.

Collapsible materials can occur on both residual and transported materials. Many granites and feldspathic sandstones when weathered result in the feldspar altering to kaolinite with the quartz particles staying intact. This forms a honeycomb type of structure, which, when wetted up and loaded, results in shearing or "collapse" of the clay bridges and a settlement or reduction in volume of the material. Certain basalts and dolerites with dry densities of 1200 to 1300 kg/m^3 have also shown collapse potential.

Indications of the possibility of collapsible materials are:

- A very low density, because of the large number of voids separating the quartz framework.
- Densities of less than about 1600 kg/m^3 (mostly in the range 1000 to 1585 kg/m^3).
- The presence of "pinholing" or voiding observed during the soil profiling.
- Usually more than 60% of the mass of the material lies in the 0.075 to 2 mm range and less than 20% is finer than 0.075 mm.
- When the material excavated from a pit is insufficient to fill the pit again (the collapse structure will be disturbed and the material will decrease in volume).

If potentially collapsible soils are identified, specialist assistance should be used for roads carrying more than 200 vpd to avoid excessive rutting. The deformation that is likely to affect lower classes of roads will seldom have a major impact on their performance. The result of collapse of the subgrade is mostly manifested by the development of a deeply rutted and often uneven road surface and significant deterioration of the riding quality of the road (Figure 18).

FIGURE 18: TYPICAL MANIFESTATION OF COLLAPSIBLE SUBGRADE

3.4 CONSTRUCTION MATERIALS SURVEY

3.4.1 General

Part of the initial survey programme is also to identify potential sources of construction materials. Although the DCP design method attempts to optimise the use of in situ materials and minimise additional material usage, in many cases, the local materials may not be of suitable quality and other materials may be required.

The availability of construction materials is becoming increasingly constrained as suitable materials are rapidly being depleted and environmental pressures limit possible material exploitation. This is leading to increased costs in obtaining and hauling material, long haulage distances that cause damage to the existing road infrastructure with associated increased vehicle emissions and possible construction delays, while alternative materials are sourced. The increasing need for environmental preservation and "Green" issues is also changing the road construction milieu.

To minimize the cost of road projects, particularly those carrying light traffic, materials used in their structural layers should be sourced as close to the project as possible (haulage costs are frequently the highest component of material provision) and should be of the most appropriate standard for the respective layer. This means that the material should:

- provide the necessary strength and stiffness for the proposed layer in the road, without having excessively good properties.
- be able to retain those properties over the design life, and preferably longer, under the impacts of traffic and climate.

It is essential that appropriate specification requirements are introduced for different categories of LVR, considering the main properties required, as well as the appropriate test methods (Section 3.5). This is particularly important for lightly trafficked rural access roads, where the traffic and loads imposed on most roads are minimal, and significant economies can be made by using material appropriate for the specific pavement characteristics. The design philosophy should be changed from "finding materials to suit the proposed pavement design" to "designing the pavement to suit the available materials".

The main requirements of materials in the structural layers of roads are to provide an adequate strength and stiffness to avoid shear failure or traffic induced compaction in the subgrade and to retain these properties over the expected life of the road, i.e. to be durable. It should be noted that traffic induced compaction (or rutting) can be almost eliminated by ensuring that all materials in the pavement are compacted to as high a density as possible (exclusion of as many voids as possible).

Recent developments and research have indicated that the plasticity is in fact inherent in the strength of a material, together with other properties and in most cases, provided that the strength requirements are met, the plasticity is almost immaterial. It does, however, provide an indication of the potential moisture susceptibility of the material. One of the problems of prescribing various "interrelated" properties, in terms of specifications, is that if a material fails to satisfy one of these specifications, it will be rejected for use. In this way, materials that easily satisfy the "most important" strength criterion, but are marginally deficient on the plasticity or grading are often rejected for use.

Despite the innumerable differences that exist among local materials, there are some dominant characteristics that affect pavement performance which should be appreciated to design and construct LVRs using such materials with confidence. These characteristics depend on whether the materials are used in an unbound or bound state, which affects the way they derive their strength in terms of the following intrinsic properties:

- Inter-particle friction.
- Cohesive effects from fine particles.
- Soil suction forces.
- Physico-chemical (stabilization) forces.

The relative dependence of a material, and the influence of moisture, on each of the above components of shear strength will significantly influence the way they can be incorporated within a pavement. In this regard, Table 7 summarizes the typical relative characteristics of unbound and bound materials that critically affect the way in which they

TABLE 7: PAVEMENT MATERIAL TYPES AND CHARACTERISTICS

Parameter	Pavement Type			
	Unbound			Bound
	Unprocessed	Moderately Processed	Highly processed	Very highly processed
Material types	Category 1 As-dug gravel	Category 2 Screened gravel	Category 3 Crushed rock	Category 4 Stabilized gravel
Variability	High	Decreases		Low
Plastic Modulus	High	Decreases		Low
Development of shear strength	Cohesion, suction and particle friction/interlock	Cohesion, suction & some particle interlock	Particle interlock.	Particle interlock & chemical bonding
Susceptibility to moisture	High	Decreases		Low
Design philosophy	Material strength maintained only in a dry state	Selection criteria reduces volume of moisture sensitive, soft and poorly graded gravels		Material strength maintained even in wetter state
Appropriate use	Low traffic loading in very dry environment	Traffic loading increases, environment becomes wetter		High traffic loading in wetter environments
Cost	Low	Increases	High	High
Maintenance requirement	High	Decreases		Low
Of particular significance to LVRs				

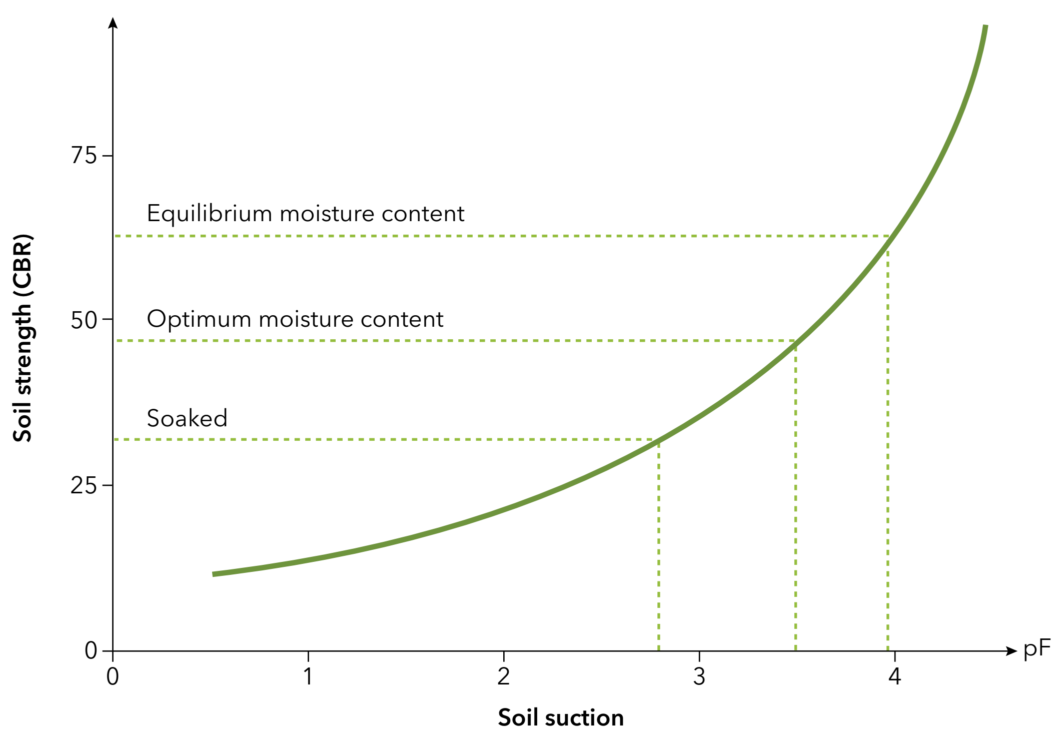
can be incorporated into a pavement in relation to their properties and the prevailing conditions of traffic, climate, economics and risk.

Unprocessed materials (Category 1), such as laterite, are highly dependent on suction and cohesion forces (gravel components will also add some interlock and friction, provided that the particles are strong enough to avoid excessive break-down) for development of shear resistance which will only be generated at relatively low moisture contents. Consequently, special measures must be taken to ensure that moisture ingress into the pavement is prevented, otherwise suction forces and shear strength will be reduced which could result in failures (Figure 19).

Since most LVRs are constructed from unbound materials, a good knowledge of the performance characteristics of such materials is necessary for their successful use as discussed below:

- **Category 1 materials:** are highly dependent on soil suction and cohesive forces for development of shear resistance. The typical deficiency in hard, durable particles prevents reliance on inter-particle friction. Thus, even modest levels of moisture, typically approaching 60% saturation, may be enough to reduce confining forces sufficiently to cause distress and failure.
- **Category 2 materials:** have a moderate dependency on all forms of shear resistance – friction, suction forces and cohesion. Because these materials have rather limited strength potential, concentrations of moisture, typically 60-80% saturation may be enough to reduce the strength contribution from suction or cohesion sufficiently to cause distress and failure. This would occur at moisture contents lower than those necessary to generate pore pressures.

FIGURE 19: ILLUSTRATIVE SOIL STRENGTH/SUCTION RELATIONSHIP



- **Category 3 materials:** have only minor dependency on suction and cohesion forces but have a much greater reliance on internal friction which is maximized when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100% will be necessary to cause distress and this will usually result from pore pressure effects.
- **Category 4 materials:** rely principally on physio-chemical forces which are not directly affected by water. However, the presence of water can lead to distress under repetitive load conditions through layer separation, erosion, pumping and breakdown.

The management of moisture during the construction and operational phases of a pavement affects its performance, especially when unbound, unprocessed, generally relatively plastic materials are used. Emphasis should be placed on minimizing the entry of moisture into an LVR pavement to ensure that it operates as much as possible at an unsaturated moisture content. The beneficial effect of so doing is illustrated in Table 8, which shows the variation of a material strength (CBR) with moisture content.

TABLE 8: VARIATION OF CBR WITH MOISTURE CONTENT

Laboratory Soaked CBR (%)	Approximate Laboratory Unsoaked CBR (%) at varying FMC/OMC Ratios		
	1.0	0.75	0.50
80	105	150	200
65	95	135	185
45	80	115	165
30	65	95	140
15	45	70	110
10	35	60	100
7	30	50	85

The FMC/OMC ratio is a significant contributory factor related to the performance of an LVR. If, through effective drainage, the materials in the road pavement can be maintained at a moisture content that does not rise above OMC in the rainy season, then more extensive use can be made of local, relatively plastic materials that might otherwise not be suitable if they were to become soaked in service.

3.4.2 In situ materials

The in situ materials are those beneath the proposed road or in the immediate surroundings that may be utilised in the construction. They consist of those materials that are naturally in their current form (state) and position because of geological and geomorphological processes.

There is no control over the quality of these materials and decisions must be made as to whether they are to be utilised or spoiled. These decisions are typically based on the results of the test pitting and testing.

3.4.3 Borrow pit materials

On most projects, it will be necessary to make use of some imported materials. These would be natural gravels consisting of transported or residual materials that occur relatively close to the project, to minimise haul distances.

During the initial surveys, it is important to identify existing borrow pits or use local indicators (excavations, plants, erosion channels, etc.) to identify other potential sources of construction materials. In most cases, these would be for use in base course construction.

Materials of particular interest for construction of layer works include local moorums, kankar and laterite and particular note should be made of the presence of these.

3.4.4 Aggregate sources

Fresh or unweathered rock is ultimately the source of all the above material groups and consists of igneous, sedimentary or metamorphic material that has undergone no (or very little) decomposition or alteration. It usually requires blasting, crushing and screening before its use in roads, but is often necessary for concrete aggregate for structures along the road as well as aggregate for surfacing seals and asphalt.

These materials, depending on the rock type and origin, generally provide high quality, durable aggregates after excavation, crushing and screening. However, they are costly materials, require the development of a quarry and are difficult to locate in or close to urban areas where environmental and mining issues often result in severe technical constraints and cost implications.

The use of fresh rock in low volume and rural access road structures should thus be limited to high quality surfacing aggregates where appropriate and concrete applications within the overall road system. There is seldom a need to use such high quality and expensive materials for pavement layers (or even many bituminous surfacings, such as sand and Otta seals) in LVRs and such materials should be conserved for use in higher standard roads in future.

To promote "greenness" in road construction, the use of non-natural materials should be increased as far as possible. These include by-products from most industrial enterprises (e.g. mining, manufacturing, industrial, power generation, etc.) as well as wastes produced in the urban environment (small industrial, construction and demolition wastes and even domestic wastes).

These materials have mostly been through a severe comminution and often heating process and generally incorporate a significant quantity of "embodied energy" through their primary processing as opposed to using a lot of additional energy in their production for use in roads. Their use also emits considerably less emissions than the processing of an equivalent quantity of natural material.

There is a wide range of such materials available, many of which can be (and have been) used in structural layers for roads, while others can be used in lower support layers or as modifiers or stabilizers for improving materials. Common waste (or preferably termed "by-product" materials include, but are not limited to:

- Reclaimed bituminous material (RA)
- Crushed concrete
- Pulverized fuel ash (PFA or flyash)

- Blast-furnace slag
- Steel slag
- Other metallurgical slags (e.g. chrome, manganese, copper, zinc)
- Aluminium industry wastes
- Colliery spoil
- Construction and demolition waste
- Mine and stone processing waste (Marble, slate, granite, etc.)
- Phosphogypsum
- Tires
- China clay sand
- Foundry sand
- Crushed waste brick
- Used rail ballast
- Furnace bottom ash
- Cement kiln dust
- Glass
- Spent oil shale

Some of the issues with the utilization of by-product materials are the variability often encountered in existing "waste" dumps due to uncontrolled disposal as well as the unique properties of each type of by-product material, which need research into their individual characteristics and usage requirements. Another issue is that many of these products are, usually incorrectly, classified as hazardous or toxic wastes by environmental authorities and require special permits for their use. It is thus important that all potential waste materials be tested for toxins and environmental acceptability before substantial work is carried out on them. Table 9 summarizes some of the uses and potential problems associate with by-product materials.

3.4.5 Typical Local Materials and Industry By-Products

Typical local materials and Industry waste available in some states, which could be used in road construction for LVRs are presented below. It is recommended that other materials and industry waste, not mentioned here, should be investigated for use in road construction.

3.4.5.1 Kota Limestone

There is considerable by-product material produced by the mining and production of limestone wall and floor tiles in the Kota area of Rajasthan that

Kota Stone

TABLE 9: CLASSIFICATION AND SUMMARY OF USES AND PROPERTIES OF BY-PRODUCT MATERIALS

Classification of by-product material	Possible uses	Potential problems	Special requirements
URBAN BY-PRODUCTS			
Crushed concrete	Base Subbase Concrete aggregate	Can lead to over-stabilization	Requires careful sorting at collection point Requires crushing and screening
Aluminium industry wastes	Below subbase Stabilizer	Very fine material Often mixed with other wastes	Unknown at this stage
Construction and demolition waste	Base Subbase Concrete aggregate	Requires careful processing/sorting May contain a lot of deleterious materials	None – should behave as a normal aggregate
Tires	Below subbase Bitumen modifier Concrete aggregate	Variability of rubber types in tires (natural and synthetic)	Requires innovative research (e.g. highly flexible pavements) Collection, sorting and processing must be formalized
Foundry sand	Subbase Below subbase Mechanical stabilizer	Fine grained. May contain unusual chemicals (release agents)	Unknown at this stage
Cement kiln dust	Stabilizer	Very fine dusty material Pozzolanicity lost with storage	More rapid construction (quicker reaction rates due to fineness). Difficult to place and mix
Glass	Surfacing aggregate Mechanical stabilizer	Difficult to work with Fractures into flaky particles	Special processing/ crushing technique needs to be developed
RURAL BY-PRODUCTS			
Pulverized fuel ash (PFA or flyash)	Cemented layer Stabilizer	Very fine/dusty. Monopolies often control supply	New innovative uses need to be assessed
Blast-furnace slag	Surfacing aggregate Base Subbase Concrete aggregate	Contains unhydrated oxides. Can be variable Possible deleterious components Variable density (voids)	Must be conditioned before use to eliminate oxide
Steel slag	Surfacing aggregate Base Subbase Concrete aggregate	Contains unhydrated oxides Can be variable Possible deleterious components Variable density (voids)	Must be conditioned before use to eliminate oxides. Otherwise similar to a conventional aggregate

Other metallurgical slags	Base Subbase	Contains unhydrated oxides. Can be variable Possible deleterious components Variable density (voids)	Must be conditioned before use to eliminate oxides
Colliery spoil	Subbase Below subbase	Highly variable materials May contain sulfates Abrasive on crushing plant	Needs good sorting and processing. Otherwise similar to a conventional aggregate
Mine and stone processing waste	Surfacing aggregate; Base; Subbase; Concrete aggregate	May contain deleterious materials	Need to be assessed individually
Crushed waste brick	Subbase Mechanical stabilizer	Variable physical properties Small individual deposits	Needs investigation for each use
Furnace bottom ash	Base Subbase Below subbase	Soft particles Contains unburnt coal May contain sulfates	Needs careful processing
Spent oil shale	Subbase Below subbase	Variable hardness and composition	Little known
BY-PRODUCTS FROM BOTH RURAL AND URBAN ENVIRONMENTS			
Reclaimed bituminous material (RA)	Asphalt Base Subbase	Variable materials	Each source needs evaluation and investigation for each project
Phosphogypsum	Cemented layer	Very fine powder Changes moisture condition readily	Needs additional research for bulk use
China clay sand	Mechanical stabilizer Below subbase	Fine grained	Little research done
Used rail ballast	Base Subbase Concrete aggregate	Collection of sufficient quantities	New disposal strategy from railway authority required

could be used in LVR construction. Quarries producing Kota limestone generate large quantities of solid waste.

Every year nearly 23 million tonnes of solid waste is added to the existing quantities in the Kota area. This waste is in the form of overlying burden, inter bedded burden and production waste generation during cutting, sizing and splitting at quarry floor. The waste limestone has a high compressive strength of nearly 150 MPa. It can be used as a subbase and base material in road construction. Even though a large quantity of this material is available, its economic haulage will need to be ascertained. However, from an environmental perspective, it would be desirable to use this by-product material.

3.4.5.2 Jarosite

Jarosite stockpile

Jarosite is a by-product material produced during the extraction of zinc ore concentrate in the hydro-metallurgy operation. Technical specifications have been developed for the utilization of Jarosite material in the construction of embankment and subgrade layers of road. The findings indicate that Jarosite (100%), Jarosite-soil mixes (50-75%) and Jarosite-bottom ash mixes (50-75%) have the potential for the construction of road embankments while the Jarosite-soil and Jarosite-bottom ash mixes (50-75%) may be used for construction of subgrade layers of pavements [17].

The economic viability for use in roads by mixing with local soils and comparative increase in strength and its use in road subbase should be assessed. The high addition of lime and cement, however, may make it uneconomical except in certain areas.

3.4.5.3 Marble Dust

Marble dust

The wastes generated from the marble industry have the potential to be utilised in various applications including road subbase material. A study in Turkey [18] indicated that fly ash, marble dust and waste sand are potentially useful additive materials in road subbases as they improve the CBR and reduce swelling in clayey soils.

3.4.5.4 Kankar

Kankar

Kankar is available at the road-side in open borrow-pits in many areas. There may be several locations where similar material may be available for direct use in the roads especially for very low volume roads.

3.4.5.5 Copper Slag

Copper slag is available as a waste product in Rajasthan and other states in India. It is a “waste” product from the copper smelting process. It is estimated that in the production of 1 tonne of blister copper, 2.2 tonnes of slag are generated. Copper slag can be classified as a non-hazardous material. However, it needs to be tested to verify that it meets the regulatory requirements. Granulated copper slag is more porous and, therefore, has similar particle sizes to coarse sand.

Copper slag

Trials and past research [19] suggest that:

- Copper slag can be used as a construction material in combination with cement and fly ash to improve the properties of expansive clay soils.
- From 30% to 50% copper slag can be mixed with soils to improve their characteristics.
- Fine sand with up to 40% copper slag can be used as fine aggregate in pavement quality concrete as well as in dry lean concrete.
- CBR values of cement mixed with soil and copper slag are 3 to 7 times higher than that of the soil with copper slag waste without the presence of cement.
- Expansive soils can be improved by utilizing 40% of the copper slag along with 2% Portland cement.
- The soaked CBR value of a 30% fly ash / 70% copper slag blend was reported to be 78% after 28 days of curing.
- To get good soil stabilization, combinations of 70% clay/30% copper slag ranging to 30% clay/70% copper slag were the most satisfactory combinations.
- In Texas, copper slag used as a coarse aggregate in a concrete pavement has performed well after over 15 years in service.

3.4.6 Material improvement

There are many instances when the local materials, even if highly compacted and retained in a dry state do not meet the strength requirements for a specific layer. In these cases, the use of some form of mechanical or cementitious improvement may be necessary. There are several options available, many of them, however, being expensive. A laboratory investigation to assess the improvement in material properties with each of the techniques is essential, to optimize the cost and the material properties. This usually requires innovative thinking and appropriate laboratory testing on part of the design engineer.

In this process, the following questions should be answered:

- What are the pavement requirements?
- What materials are available for investigation?
- How best are each of these materials treated?
- What laboratory test procedure is necessary to prove their "fitness for purpose"?

3.4.6.1 Mechanical stabilization

This involves the improvement of a material by mechanical means. Various techniques are available for this and must be seriously considered as they are far more cost-effective than chemical or other types of stabilization.

Compaction

Compaction is by far the most economic and simple, way of improving a material. All the properties of a material are enhanced by achieving as high a degree of compaction as possible. These include:

- Higher shear strength and stiffness,
- Lower permeability
- Less rutting potential
- A better support (anvil) for the compaction of overlying layers

All attempts to improve the compaction of layers and achieve the highest possible compaction should be made. This requires good compaction equipment with operating vibration capacity, a uniform distribution of moisture (at close to or just below optimum moisture content for modified compaction) through the material and the necessary number of passes. Compaction is one of the cheapest construction activities and additional compaction generally involves only a small additional fuel cost – the equipment and operators are on site anyway, even when the compaction process is not taking place, and thus involve no additional costs.

No compromise on compaction should be allowed: the use of well-compacted marginal materials and subgrades not only improves the pavement performance but also allows thinner pavement layers to be used, with significant material savings and the associated sustainability benefits. Compaction to refusal, which normally entails a maximum of 4 or 5 additional roller passes until no additional densification occurs, but avoiding breakdown of particles and de-densification, is generally considered a highly beneficial operation.

Removal of oversize

The removal of oversize material enhances the workability of the material as well as improving its properties. Material with excessive oversize material often acts as "plums in a pudding" resulting in no strength being contributed by the aggregate (no inter-particle friction/interlock), with the strength of the finer matrix determining the overall material properties. In addition, the impact of the oversize material on the properties of the bulk material cannot be determined as these aggregate particles are excluded from most testing.

There are many ways of removing oversize, but this is done most economically at source where unnecessary haulage to the roads and usually back to the borrow pit can be avoided. Screening and/or crushing in the borrow pit (using small or mobile crushers) should be carried out when excessive oversize material is present, this usually being classified as material with more than about 5 or 10% larger than 37.5 mm.

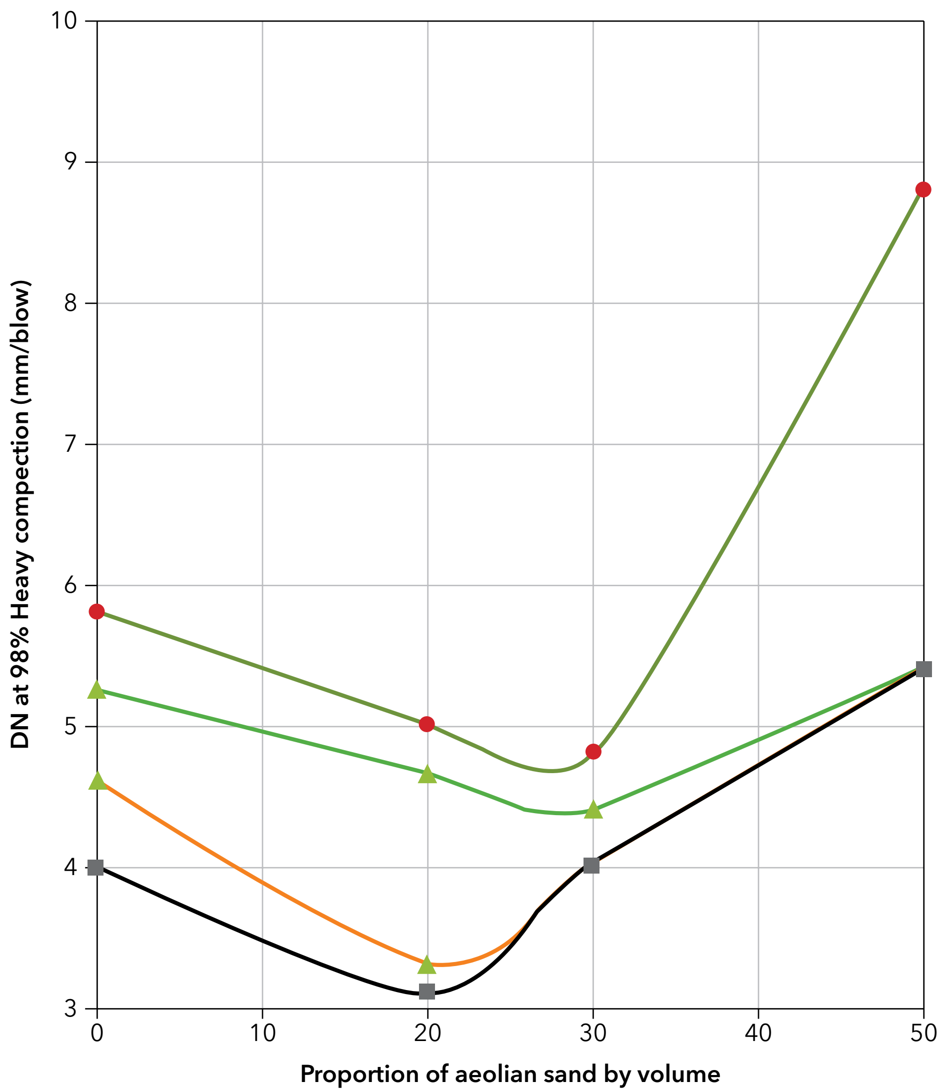
The presence of oversize materials results in a poor finish of the compacted layer surface and makes final cutting of levels difficult. This is not critical for subgrade and subbase layers (in fact a rough surface may enhance the bond between the layers) but is not permissible for base courses where a thin bituminous surfacing is to be applied. Large aggregate particles also interfere with the uniform compaction of the materials (material adjacent to large particles is not effectively compacted).

Blending

Blending of different materials can be used to improve the characteristics of many poor materials. Mixing of materials from different sources, usually one coarse and one finer can reduce (or increase when required) plasticity and improve the shear strength and performance of many materials. The optimum blend ratio is usually determined by laboratory testing, but can also be estimated mathematically using various processes.

In India, the increased use of blending is perceived to be a major advantage in future construction of LVRs. Many of the natural subgrades are very weak fine materials (e.g. Ganga alluvial silts and clays) with no gravelly alternatives and their strengths can be significantly

FIGURE 20:CHANGE IN STRENGTH (DN IN MM/BLOW) AS A RESULT OF BLENDING



improved by the controlled addition of coarser materials. These coarse materials can often be obtained from the by-product materials discussed previously (a typical example being waste bricks), preferably materials in the range 5 to 25 mm in diameter. However, to optimize each possible blend, laboratory tests with different proportions of the two materials need to be carried out to identify the optimum blend ratio.

The addition of a coarse fraction will usually raise the quality of the silty/clayey material to at least selected subgrade and often subbase quality. In such cases, laboratory investigations are necessary using various proportions of the two materials to determine the optimum blend ratio.

This type of investigation is particularly relevant in parts of India, where there are often, for instance, large sources of reject bricks (brick-bat) that could be broken/crushed and added to the local silty materials to provide a better material. Representative samples of the reject bricks (even including the residual ash and soft bricks) should be crushed to a maximum size of about 20 or 25 mm and added in various proportions to local soils as indicated for the sands above, to determine whether the strength (in terms of DN value) can be improved to that required for the proposed layer.

Each source of local material and by-product material needs to be tested individually to check for compatibility and effectiveness. This is usually best done by initially blending various ratios of the dominant material with smaller quantities of the "improving" material (e.g. 90:10, 80:20, 70:30, etc.) to identify the optimum blend ratio.

3.4.6.2 Chemical stabilization

Chemical stabilization involves the addition of a chemical agent to the materials that affects the chemical properties of the material. This could be either chemical modification, in which only the actual minerals are affected without any chemical cementation (i.e. no tensile strength is developed) or cementation in which new cementitious bonds (hydrated calcium silicate minerals) are developed, imparting a significant tensile strength to the material.

The quantity of stabilizer required to achieve the desired properties may be so high as to make the process totally non-cost-effective. For this reason, each material must be assessed using the most suitable stabilizers and following a standard test protocol. This protocol will vary depending on the types of stabilizer investigated.

Conventional chemical stabilization makes use of cement, lime, lime/flyash blends and various other mostly by-product materials (ground granulated blast furnace slag (GGBS), cement kiln dust, burnt rice husk ash, etc.). Cement is typically used for materials with a Plasticity Index (PI) below about 10%, lime for materials with a PI greater than 15% and combinations of the two with intermediate PI values. It should also be noted that cement can have a range of compositions, depending on the type and quantity of extender (i.e. PFA, GGBS, pozzolan, ground limestone, etc.) added and the rate of reaction depends significantly on the fineness to which the cement is ground.

Flyash or GGBS alone cannot be used to stabilise soils. They must be activated by the addition of lime or cement (preferably lime) to allow their pozzolanic or latent hydraulic reactions to occur. Thus, it is always necessary to carry out laboratory testing with various combinations of the flyash or GGBS with and lime or cement for each soil type being investigated.

Once the best stabilizer type has been identified (in terms of both availability and reactivity, based on initial testing with a nominal stabilise content), it is necessary to determine the optimum stabilizer content. This requires that the material is tested with various contents of the selected stabilizer to determine the optimum content at which the design strength (Unconfined (UCS) and tensile (ITS)) is obtained. It should be noted that, to ensure long-term durability of the cemented layer, this percentage of stabilizer should exceed the Initial Stabilizer Demand (ISD) by at least 1%. It is also often useful to treat blended materials (coarse and fine blends (see 3.4.5.1)) with a chemical stabiliser to improve the material even further. This is done on the pre-blended material as described above.

Chemical stabilization must not be a purely mechanical process in which a fixed quantity of stabilizer is added to any material, as every material has unique mineralogy and properties resulting in different stabilizer requirements – both in terms of quantity and type of stabilizer. Testing of the ISD is essential to determine which stabilizer is best and what quantity should be added to produce the necessary properties. Suitable guidelines in this respect do not currently exist in India and need to be developed as a separate Guidance Note.

Chemical stabilization is an art and champions in this art need to be developed in India. Many problems with chemical stabilization occur during construction and it is essential that a pool of experts be developed in India, who can respond to problems and quickly identify the mechanisms for avoiding them during projects. It is also important that the behaviour and performance characteristics of stabilised materials are fully understood by users.

3.4.6.3 Proprietary soil stabilizers

Internationally, the road construction market is currently flooded with various proprietary chemical soil stabilizers and improvers. These chemicals should be used with caution, only after rigorous laboratory testing, careful economic analysis and preferably carrying out some properly controlled and monitored field trials. The use of chemically treated layers near the top of the pavement structure, as proposed by many of the chemical vendors to use local materials, should ensure that the pavement balance is not disturbed too much. Road performance is seldom satisfactory when a strong treated layer is constructed on a poor support.

Proprietary soil stabilizers make use of various physical and chemical additives, each with its own mechanisms, actions and results. Many of these are not communicated fully by the suppliers and it is often difficult to carry out tests to assess their performance, without knowing the fundamental principles related to the products.

3.4.7 Construction water

Road construction requires significant sources of water to ensure proper compaction, stabilisation and finishing of the structural layers. During the initial investigations and surveys, it is thus also important to identify potential sources of construction water. Normal river water is usually adequate for construction, but groundwater resources may be slow and expensive to extract and may also be excessively saline for road construction in many cases. Typically, water that is potable to humans should be used for construction, to minimise potential soluble salt problems.

3.5 MATERIALS TESTING

3.5.1 General

The samples collected during the route survey must be tested in the laboratory to assess their properties and suitability for possible use in the pavement. All testing should comply with local standards and the methods must be meticulously followed, using the correct and recently calibrated equipment.

These standard material testing techniques (and their accompanying material specifications) have been developed to assess the quality and durability of road construction materials over the past century or so. Unfortunately, many of these have not been reviewed for the past 70 or 80 years and are still applied in the standard manner, despite numerous changes to road design philosophies over this period. An example of this is the almost international use of a minimum soaked California Bearing Ratio (CBR) of 80% and a maximum Plasticity Index of 6% for all base course materials irrespective of traffic and environment. It is well known that both tests have different protocols in different countries, and thus produce significantly different results, and yet the same limits are used in various countries. It should also be noted that a CBR of 80% for base course materials is used in many design guides, irrespective of the need of the road, i.e., the specification is applied to roads carrying 1,000,000 standard axles as well as those designed to carry only 10,000 standard axles. This results in high construction costs as well as a wastage of material that should be conserved for roads with higher volume of traffic.

3.5.2 Test methods

Samples of the local in situ materials as well as the possible construction materials collected during the soil survey should be tested in an approved laboratory for routine properties that allow the material to be classified. Classification testing will include as particle size distribution, plasticity and compaction characteristics (maximum dry density and optimum moisture content) and material strength.

Two important deviations from conventional Indian practice are, however, required:

1. Compaction testing must include the use of IS 2720 Part 8 – (heavy compaction) as the use of higher compactions alone can result in significant improvements in material properties in the field as well as leading to savings in material quantity necessary for any single traffic class.
2. Instead of conventional CBR testing, the materials are compacted into the CBR moulds (150 mm diameter) and testing using a DCP directly in the mould (Figure 21).

This testing should be done at various moisture and density combinations for the typical soils to get a good understanding of the moisture/density/strength relationships of the materials as shown in Figure 22.

When blending or improving materials, the standard test protocols should be followed on the individual materials as well as the different blends. If blending alone is insufficient to improve the materials to the required quality, controlled addition of different stabilisation agents, (e.g. cement, cement/flyash mixtures, lime, etc.) to the blended materials should be attempted. It is unlikely that such mixtures, without prohibitively high cement contents, will lead to any significant cementation development, and conventional CBR testing (at heavy compaction effort) should be carried out on the blends.

The data obtained through this testing will be applied directly to the design of the pavement structure using the DCP method (Chapter 4). In addition, the results of the laboratory testing at the specified density and compaction moisture content can be used later for quality control purposes during construction.

FIGURE 21: DETERMINATION OF LABORATORY DN VALUE

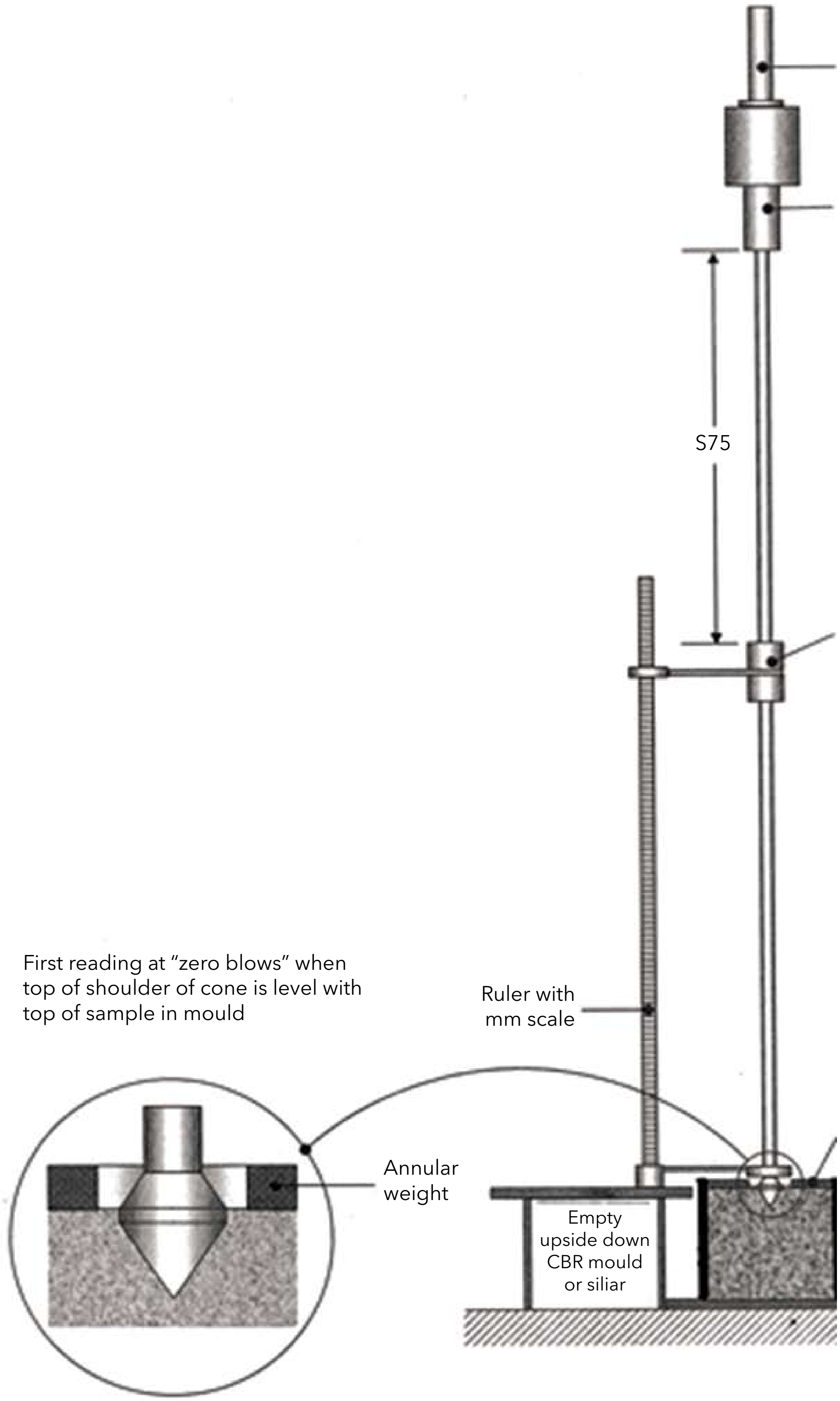
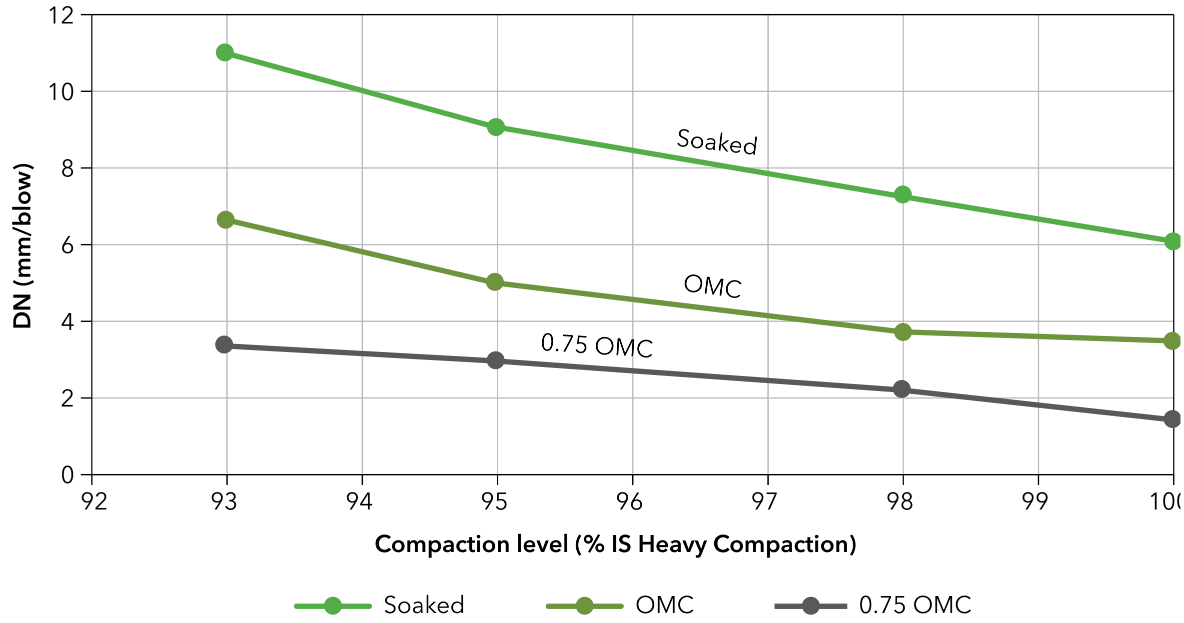


FIGURE 22: DN/DENSITY/MOISTURE RELATIONSHIP



4. PAVEMENT DESIGN

4.1 INTRODUCTION

4.1.1 Background

The objective of pavement design is to produce an economic, well balanced pavement structure, in terms of material types and thicknesses, that can withstand the expected traffic loading over a specified period (the chosen design life of the pavement), without deteriorating below a pre-determined level of service. To achieve this goal, sufficient knowledge of the subgrade strength, pavement materials, traffic loading, local environment factors (particularly climate and drainage) and their interactions is required to be able to predict reasonably the performance of any pavement configuration. In addition, there should be a clear view as to the level of performance and pavement condition that is considered appropriate in the circumstances for which the pavement structure is being designed.

Pavement design for LVRs presents a particular challenge to designers. This is largely because, until relatively recently, such roads were not specifically catered for and the step from a gravel road to a paved road one was a large one. However, considerable research has been carried out internationally that has led to the development of simplified pavement design methods that enable unpaved roads to be upgraded economically to a paved standard by making optimal use of local materials that do not meet the standard specifications found in most design manuals. It is these design methods that are described in this chapter.

One of the biggest changes in the design process discussed in this chapter is that it is based on extending design procedures for unpaved roads upwards for low volume paved roads. The more conventional practice of trying to reduce conventional paved road designs for low volume roads has been found to have numerous pitfalls. Probably the most significant of these is the use of the normal assumptions of isotropic, elastic, uniform material concepts, that are less appropriate for natural gravels and local materials used in LVRs.

4.1.2 Purpose and Scope

The purpose of this chapter is to provide details of the manner of determining the structural requirements of an LVR pavement in terms of the required layer thicknesses and material quality for different traffic categories. The design method is based entirely on the use of the DCP in contrast to the more traditional methods that are based on the California Bearing Ratio (CBR).

The scope of the chapter is to provide an overview of the DCP design method and includes a step-by-step procedure to be followed in producing a pavement design for an LVR based on the key characteristics of subgrade strength, the expected traffic loading and the properties of the available materials for use as the pavement layers.

4.2 DESIGN PRINCIPLES

4.2.1 Approach to design

The general approach to design of LVRs differs in several respects from that for 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 pavement costs for LVRs can be effected by reducing the number of pavement layers and/or thickness, by using local materials and by using lower cost, more appropriate surfacing options.

An important aspect of the design of HVRs is the minimization of pavement deflections. However, many of the lighter LVR pavement structures can tolerate relatively higher deflections (more than 1.0 mm). This is not necessarily a problem, but the choice of surfacing would certainly be influenced, with more flexible types of bituminous seal being necessary.

Ultimately, the challenge of good pavement design for LVRs is to provide a pavement that is appropriate to the road environment in which it operates and fulfils its function at minimum life cycle cost at an optimal level of service. However, positive action in the form of timely and appropriate maintenance, as well as adequate control of vehicle overloading will be necessary to ensure that the assumptions of the design phase hold true over the design life of the road.

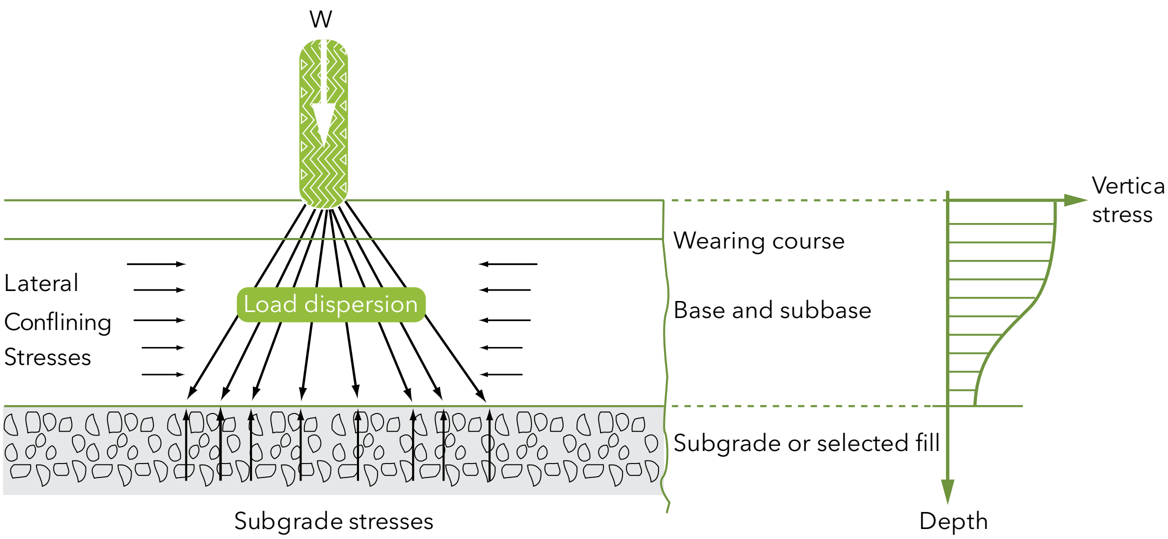
4.2.2 Pavement structure and function

When the natural subgrade of a road is not strong enough to support the repeated application of axle loads without deforming, it will be necessary to protect it from overstressing by traffic loads. This can be achieved by introducing stronger materials above the subgrade (the pavement layers) to provide a chosen level of service as cost effectively as possible. The materials comprising these pavement layers must possess the following attributes if the pavement is to perform satisfactorily within the dictates of the prevailing road environment:

1. Sufficient stiffness (load-spreading ability) which is achieved essentially through inter-particle friction and shear strength (as measured with the DCP), which depend on the presence of horizontal confining stresses.
2. Sufficient bearing capacity which is the ability to withstand repeated cycles of vertical stress without excessive deformation.

Figure 23 illustrates conceptually the way in which a pavement functions under loading. In essence, the wheel load, W, is transmitted to the pavement surface through the tire. The pavement then spreads the wheel load to the subgrade so that the maximum pressure

FIGURE 23: DISPERSION OF SURFACE LOAD THROUGH A GRANULAR PAVEMENT STRUCTURE



on the subgrade is reduced sufficiently to avoid overstressing it to an unacceptable level. This can be achieved by proper selection of pavement materials of appropriate thickness and quality.

4.3 DESIGN METHODS

4.3.1 General

4.3.1.1 Design methods

There are a number of methods that may be used for the design of LVRs which may be categorised as follows:

1. CBR cover method
2. The AASHTO design method
3. Catalogue/chart methods: Examples
   a. UK Overseas Road Note 31
   b. South African TRH4
   c. Austroads
   d. SADC/TRL
   e. IRC : SP : 72

The above methods all require four main design activities namely:

- Assessing the strength of the subgrade or of the layers of an existing old road prior to improvement or upgrading.
- Assessing the design traffic loading.
- Selecting materials for the pavement layers.
- Determining the thicknesses of the pavement layers.

4.3.1.2 CBR cover method

This is one of the original design methods that was developed in the 1950s from empirical data. The method is based on the approach of protecting the subgrade by providing enough cover of sufficient strength to protect the subgrade from the traffic loading. Various design charts have been prepared from which the depth of construction required to protect a subgrade of any defined strength (in terms of CBR) is defined for various traffic categories and equivalent wheel loads.

Due to its empirical nature, and the fact that it was developed under locally prevailing conditions in the USA, the CBR Cover Curve Method should be evaluated critically before it is applied to local environmental and traffic conditions. This method is currently not used extensively in many countries due largely to the development of a number of "Catalogue" design methods based on mechanistic-empirical and empirical design, as discussed below.

4.3.1.3 AASHTO design method

The AASHTO design method, which was developed from the results of the AASHO Road Test that was conducted in Chicago, United States, during 1959 and 1960, is still used widely in many tropical countries. However, it suffers from a number of drawbacks which cast serious doubts on its applicability to the design of LVRs in tropical countries, including:

- The Road Test was located in an area where winter temperatures cause up to 1.5 m of frost penetration for three months of the year.
- Almost all the pavement deterioration took place during the spring thaw when the pavements were saturated with water.
- It is very difficult to estimate how the roads would have performed in the absence of the freeze/thaw cycle. However, it is very likely that the long term deterioration would not have been the same as was observed during the later months of summer at the Road test.
- The Road Test was built on one type of subgrade of very low strength. Estimating the performance of roads built on other subgrades from the results of the Road test is very difficult.
- The Road Test was an accelerated test. Environmental effects that play an important role in the deterioration of roads in tropical countries was not evaluated.
- Wheel load and contact pressures remained constant. If mixed traffic had been used, the conclusions regarding the relative damage effect of each wheel load would have been somewhat different.
- The axle loads used in the Road Test were 13 tonnes on a single axle and 21.5 tonnes on a tandem axle. Such loads are not the norm on LVRs.

4.3.1.4 Design catalogues

Design catalogues/charts are the easiest design process to use as all the practical and theoretical work has been carried out and different structures are presented in catalogue form for various combinations of traffic, environmental effects, pavement materials and design options. Three main types of catalogues have been developed as follows:

1. Those based on accelerated testing, theoretical analyses, in situ testing and evaluation of pavements in service. Examples include:
    a. South African TRH4 design catalogues
    b. DCP design catalogues
2. Those based on the results of full-scale experiments where all factors affecting performance have been accurately measured and the variability quantified. Examples include:
    a. UK Overseas Road Note 31 design catalogues
3. Those based on back-analysis of existing pavements involving the monitoring, collection and analysis of performance data. Examples include:
    a. The SADC/TRL design catalogues
    b. Austroads design charts

The design approach that has been adopted in these Guidelines is based on the DCP-DN method which offers the following advantages:

- The fundamental principle of the method is based on moving technologies up from unpaved roads instead of attempting to downscale from conventional roads based on several non-valid assumptions.
- It moves away from the more traditional, empirically developed, CBR design approach, which provides an indirect measure of the strength of a material, to a more direct method of measuring in situ shear strength based on the use of the Dynamic Cone Penetrometer (DCP).
- It focuses on the use of the DCP for evaluating in situ road conditions and, by integrating the design strength profile optimally with the in situ strength profile, for designing LVR pavement structures in a highly cost-effective manner that minimizes the use of imported materials.
- It facilitates the greater use of local, more abundant, and therefore less expensive, locally available and by-product materials in the road pavement by a variety of techniques for improving these materials and for evaluating their properties in the laboratory using the DCP.
- Under circumstances where unsoaked conditions can reliably be assumed based on good drainage conditions and preferably sealed shoulders (ref. Section 4.4.4.3) and adequate maintenance.

The DCP method of design is described below and an example is provided in Annex II.

4.3.2 DCP Design Method

4.3.2.1 General

The philosophy behind the DCP design approach is to achieve a balanced pavement design whilst also optimizing the utilization of the in situ material strength at the expected in-service moisture content as far as possible. This is achieved by:

- Determining the design strength profile needed, which is related to the design traffic loading.
- Integrating the design strength profile with the in situ strength profile.

To utilize the existing gravel road strength, the materials in the pavement structure need to be tested for their actual in situ strength, using a DCP. This instrument has been designed to provide a rapid, low-cost, non-destructive method of estimating the in situ strength of fine-grained and granular subgrades, granular base and sub-base materials and weakly cemented materials.

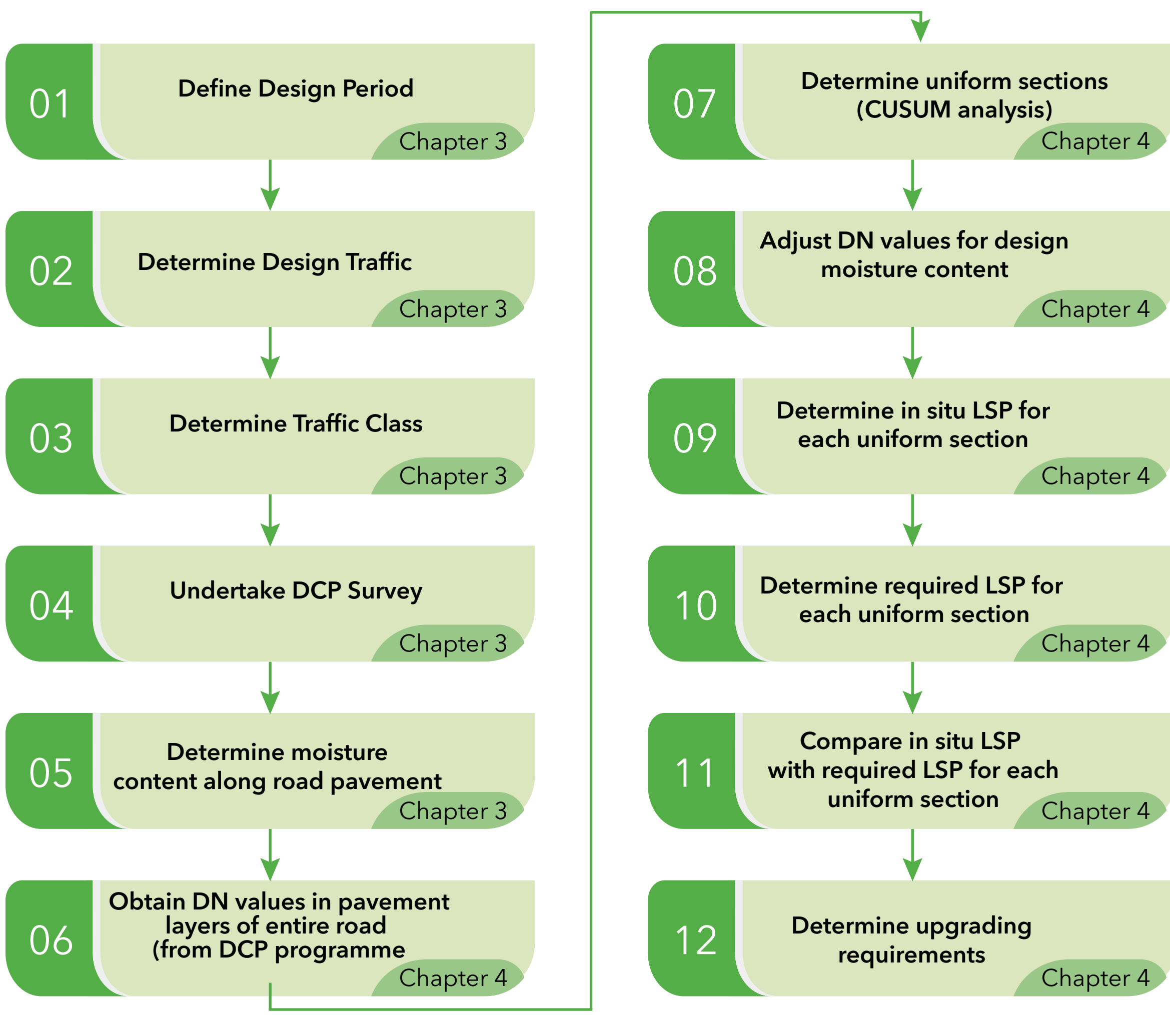
All of the previous Indian design manuals (IRC: SP20-2002; IRC: SP 72-2007 and IRC: SP72-2015) permit the use of unsoaked CBR strengths in selected locations: although this is one of the fundamentals of the DCP design concept, it thus does not differ fundamentally from the understanding in India over the past 15 years.

4.3.2.2 DCP Design Procedure

Existing tracks and roads

A flow diagram of the DCP design process is shown in Figure 24. The process entails carrying out a series of activities that are aimed ultimately at determining a suitable pavement structure from a design catalogue (this is related to the design traffic loading and corresponding traffic design category) and comparing that with the existing pavement structure determined from the DCP survey. Steps 1–5 are addressed in Chapter 3. The remaining design steps, Steps 6-12 are described in this chapter.

FIGURE 24: FLOW DIAGRAM OF DCP DESIGN PROCEDURE [16]



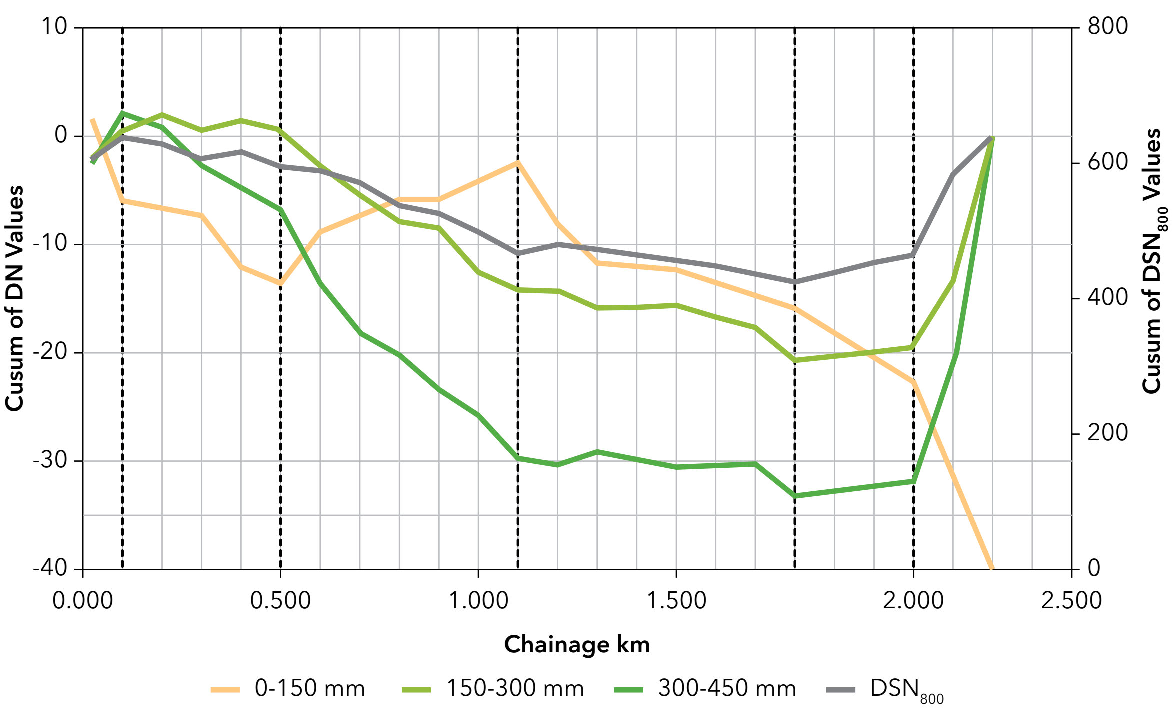
Step 6: Obtain DCP penetration rates in pavement layers and input into a DCP analysis programme: The DCP results obtained at each measurement point can be analysed manually using conventional spread-sheets but can also be analysed using software such as the AfCAP DCP program (http://www.research4cap.org/SitePages/LVRDCPSoftware.aspx) and the data processed to obtain weighted average DN values (penetration rate in mm/blow), for each test point and 150 mm layer of the pavement structure and the DSN_{800} value (total number of blows required to reach a depth of 800 mm.

For many existing pavements, the underlying structure may be adequate for the design traffic (already constructed and compacted by traffic) and only the upper layers may need reconstruction or addition. This will be confirmed by the DCP profile.

Step 7: Determine uniform sections: The DN values for each 150 mm layer as well as the DSN_{800} should be plotted against the chainage of the road, using a simple cumulative sum (CUSUM) technique to identify uniform sections along the road (described in Annex II). This will typically identify changes in underlying material types, transitions from cut to fill or variable soil moisture conditions. The manner of determining uniform sections, based on consideration of the upper three,150 mm layers, as well as the DSN_{800} of an existing gravel road is illustrated in Figure 25. This method has the advantage of looking at the entire pavement structure including material types, moisture conditions, compaction, etc. and not just the limited areas from where CBR samples were collected.

When defining uniform sections, short sections (less than about 500-1000 m) should be included in longer sections, after considering their conditions. This may entail using a slightly heavier design for the entire section to avoid localized weak spots or may entail the strengthening of one or more layers over these sections to bring them up to the

FIGURE 25: EXAMPLE OF UNIFORM SECTIONS OBTAINED FROM CUSUM ANALYSIS OF DCP RESULTS



same standard as the rest of the section. Short sections should be combined as far as possible to minimize the number of different pavement structures along the road. In areas where submergence is likely, soaked values should be used and it is suggested that the DCP testing is carried out immediately as the monsoon retreats and while the pavement is still soaked.

Step 8: Adjust DN results for design moisture: The DCP results must be adjusted for moisture conditions. Based on the estimate of the in situ moisture condition at the time of the DCP testing (Step 4) adjust the DCP results in accordance with the percentile values shown in Table 10. The DCP data collected during the dry season will be stronger (lower DN) than that collected during the wet season. The use of the respective 80th and 20th percentiles (for design traffic < 0.5 MESA) or the 90th and 30th percentiles (for design traffic 0.5–1.0 MESA) effectively results in an estimate of the expected in service conditions. The manner of determining percentiles for adjusting DN results for the chosen design moisture content is described in Annex III.

TABLE 10: RECOMMENDED PERCENTILES OF MINIMUM IN SITU STRENGTH PROFILE TO BE USED

Anticipated long-term in-service moisture content in pavement	Percentile of minimum strength profile (maximum penetration rate – DN mm/blow)	
	Design traffic < 0.5 MESA	Design traffic 0.5–1.0 MESA
Drier than at time of DCP survey	20	30
Same as at time of DCP survey	50	65
Wetter than at time of DCP survey	80	90

The in situ moisture content tends to be a function of the height of the pavement layer(s) above natural ground level, adjacent cuttings, material properties and the depth of the water table below natural ground level. Given a conducive moisture regime, after surfacing the moisture content in the base tends to stabilize at typically 70–90% of OMC. Nonetheless, since the performance of the road will be sensitive to the in-service moisture content of the pavement layers, it is of paramount importance that the long-term moisture condition is realistically assessed and, in cases of uncertainty regarding the adequacy of the drainage system, it would be prudent to assume a worst case, soaked condition.

Step 9: Determine in situ layer strength profile for each uniform section: By analyzing each uniform section on its own the spread of the data, and hence the risk level, is reduced. As illustrated in Figure 26, the in situ layer strength profile is determined by an average analysis of the DN values within each uniform section by using the DCP programme and applying the correct percentile based on the moisture condition in the pavement at the time of the DCP survey.

Step 10: Determine required Layer Strength Profile (LSP) for each uniform section: For a particular design traffic class, the required layer strength profile for each uniform section is determined from the DCP design catalogue (Table 11) for different traffic classes. Approximate CBR values are included in this design catalogue for comparison purposes with other catalogues. The design catalogue is based on the anticipated, long

FIGURE 26: AVERAGE MAXIMUM AND MINIMUM STRENGTH PROFILES FOR A UNIFORM SECTION

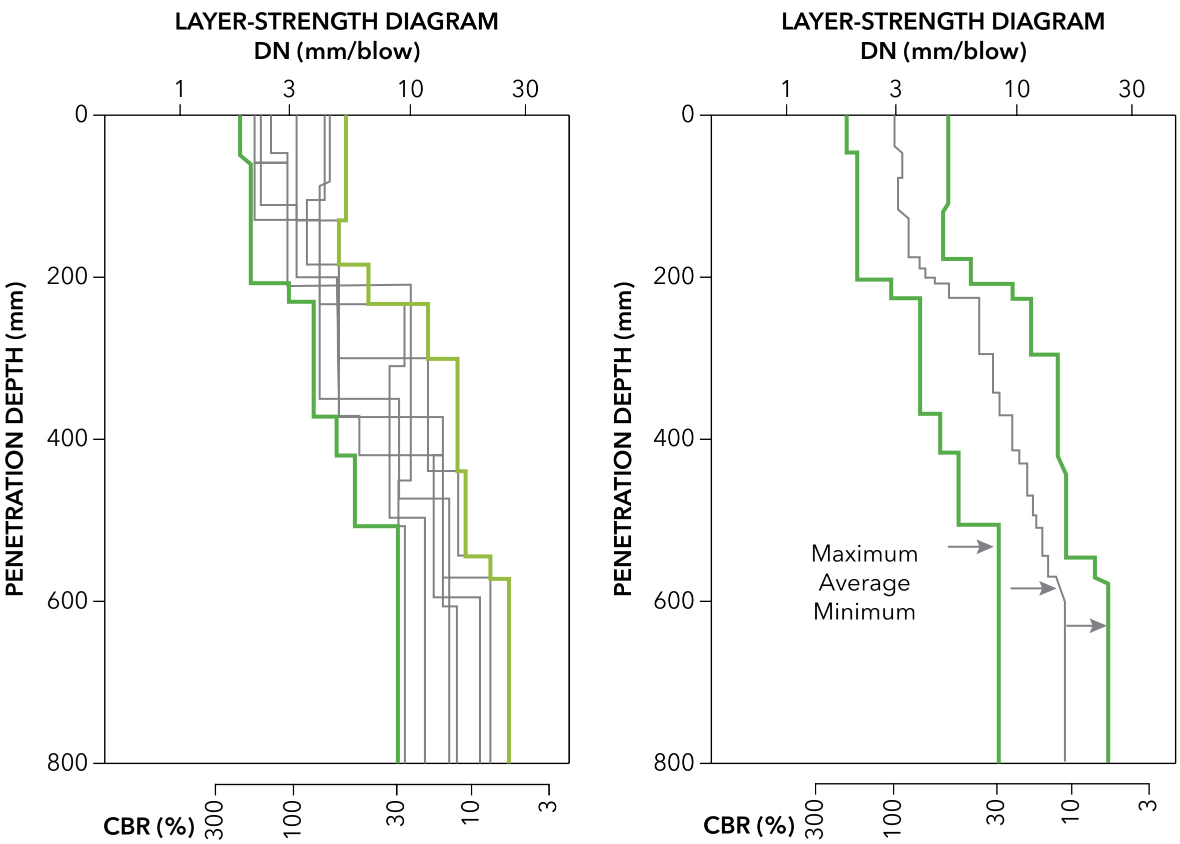


TABLE 11: DCP DESIGN CATALOGUE FOR DIFFERENT TRAFFIC CLASSES

Traffic Class	DN in mm/blow (approx. CBR%) for Traffic Class (MESA)					
	0.003-0.01	0.01-0.03	0.03-0.10	0.10-0.30	0.30-0.70	0.70-1.00
0 - 150 mm Base ≥ 98% Mod. AASHTO	DN ≤ 8 (30)	DN ≤ 5.9 (45)	DN ≤ 4 (70)	DN ≤ 3.2 (90)	DN ≤ 2.6 (120)	DN ≤ 2.5 (130)
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 19 (10)	DN ≤ 14 (15)	DN ≤ 9 (25)	DN ≤ 6 (45)	DN ≤ 4.6 (60)	DN ≤ 4.0 (70)
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 33 (5)	DN ≤ 25 (7)	DN ≤ 19 (10)	DN ≤ 12 (17)	DN ≤ 8 (20)	DN ≤ 6 (45)
450-600 mm in situ material	DN ≤ 40 (4)	DN ≤ 33 (5)	DN ≤ 25 (7)	DN ≤ 19 (10)	DN ≤ 14 (15)	DN ≤ 14 (15)
600-800 mm in situ material	DN ≤ 50 (3)	DN ≤ 40 (4)	DN ≤ 40 (4)	DN ≤ 25 (7)	DN ≤ 25 (7)	DN ≤ 25 (7)
DSN 800	≥ 39	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

(Figures in Brackets indicate approx CBR %)

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 condition. Figure 27 shows the same catalogue in a "Layer strength diagram" format as used for analysis.

FIGURE 27: DCP LAYER-STRENGTH DIAGRAM FOR DIFFERENT TRAFFIC CLASSES

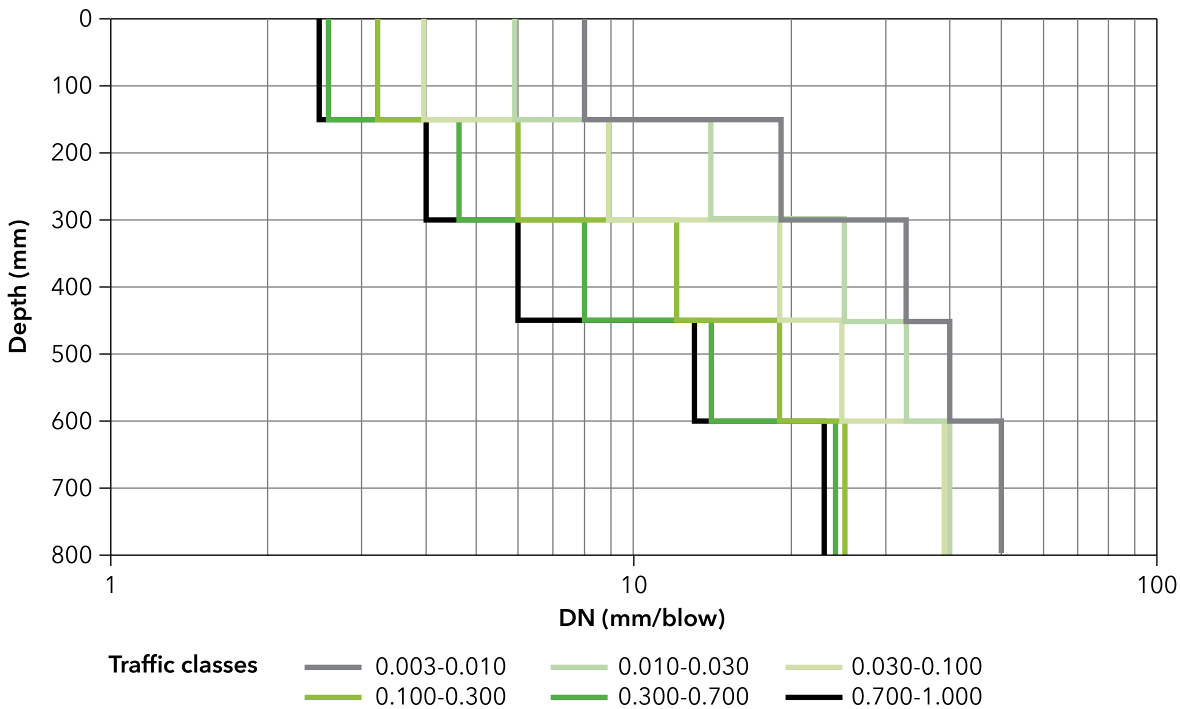
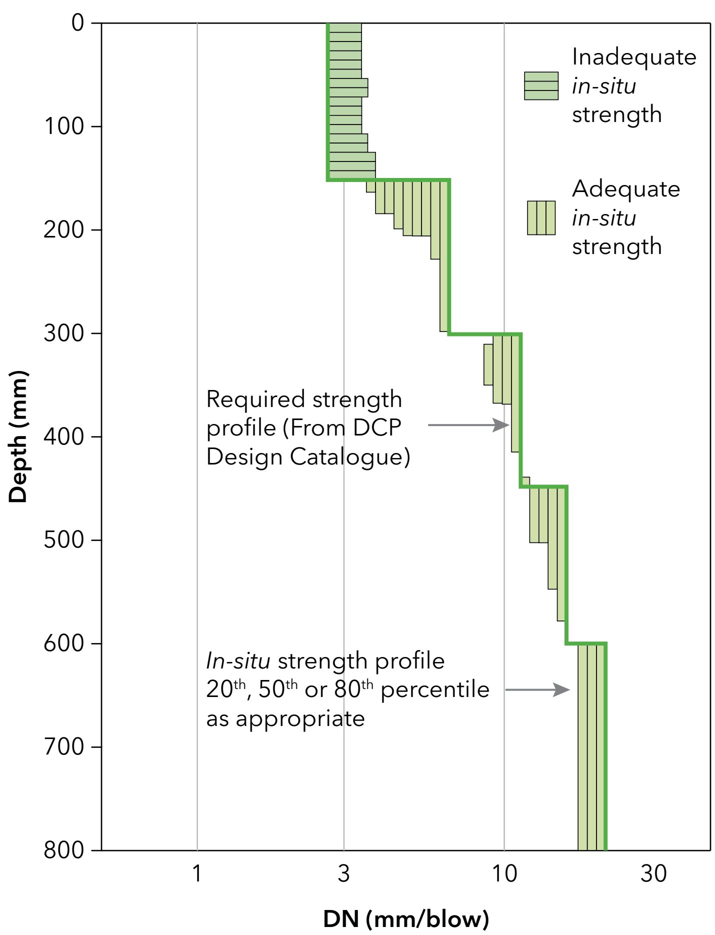


FIGURE 28: COMPARISON OF DCP DESIGN AND IN SITU STRENGTH PROFILES



In the AfCAP software the above DCP catalogue is illustrated as shown in Figure 27.

Step 11: Compare in situ Layer Strength Profile (LSP) with required LSP for each uniform section: The required strength profile is plotted on the same layer-strength diagram on which the uniform section layer strength profiles were plotted (Figure 28). The comparison between the in situ strength profile (blue and brown lines) and the required design strength profile (red broken line) allows the adequacy of the various pavement layers with depth to be assessed for carrying the expected future traffic loading.

Step 12: Determine upgrading requirements for each uniform section: The options for upgrading the pavement must then 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 drainage requirements are adequate).

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, 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 stabilization (Section 3.4.6.2).

In cases where the existing road consists of a water-bound macadam or brick soling, this can be reworked, blended with additional material or broken down (e.g. by heavy vibrating or impact rolling) and used as a new base course where it is suitable, or where underlying layers do not need to be improved.

New roads

The approach to be adopted in the DCP design of new roads is similar, in principle, to that adopted for existing roads. However, two scenarios need to be considered, viz.:

a. When the road pavement is to be founded at, or just above existing ground level.

b. When the road pavement is to be founded on a new fill or embankment, higher than about 500 or 600 mm above existing ground level.

In the case of Option (a) the DCP survey can be undertaken along the alignment (as described in Section 3.3.3.1) as the outcome would provide a meaningful input into the design process. In addition, the density and moisture content are not likely to be equivalent to that of the material in the designed and compacted embankment. However, in the case of Option (b) undertaking a DCP survey at existing ground level would be pointless as, at that depth, it would have little input into the pavement design process. As a result, the strength of the compacted embankment material would need to be determined in the laboratory as this is what will influence the structural design of the pavement.

4.3.3 CBR Design Method

For designing LVRs using the CBR, rather than DCP, method of pavement design, two options are available:

1. Base the design on CBR values obtained from the DCP/CBR correlation using the Kleyn equation [20]. The CBR values so derived (Table 11) are less conservative than those presented in the IRC: SP: 72–2015.
2. CBR values can be determined directly and the pavement designed using IRC:SP:72- 2015 (Guidelines for the Design of Flexible Pavements for Low Volume Rural Roads).

4.3.4 Comparison of DCP-DN and IRC DCP Catalogues

A comparison of the structural capacities provided by the DCP-DN and IRC: SP: 72–2015 CBR catalogues is presented in Annex IV from which it can be seen that they are almost identical in terms of traffic carrying capacity. However, if the strength distribution of the two structures are plotted with depth and compared with the theoretical balance curves, the DCP DN design shows a well-balanced structure compared with the relatively shallow pavement structure exhibited by the IRC design which makes the latter very sensitive to overloading with a number of stress concentrations in the upper 100 mm.

Although the standard catalogue for the DCP design method uses layers of 150 mm (primarily to overcome problems often seen on site with layer thickness control), the method can be adapted for any selected layer thicknesses or materials available, as illustrated in Annex IV.

4.4 PRACTICAL CONSIDERATIONS

4.4.1 General

There are some practical considerations associated with the performance of a road that are of particular significance to LVRs. These include:

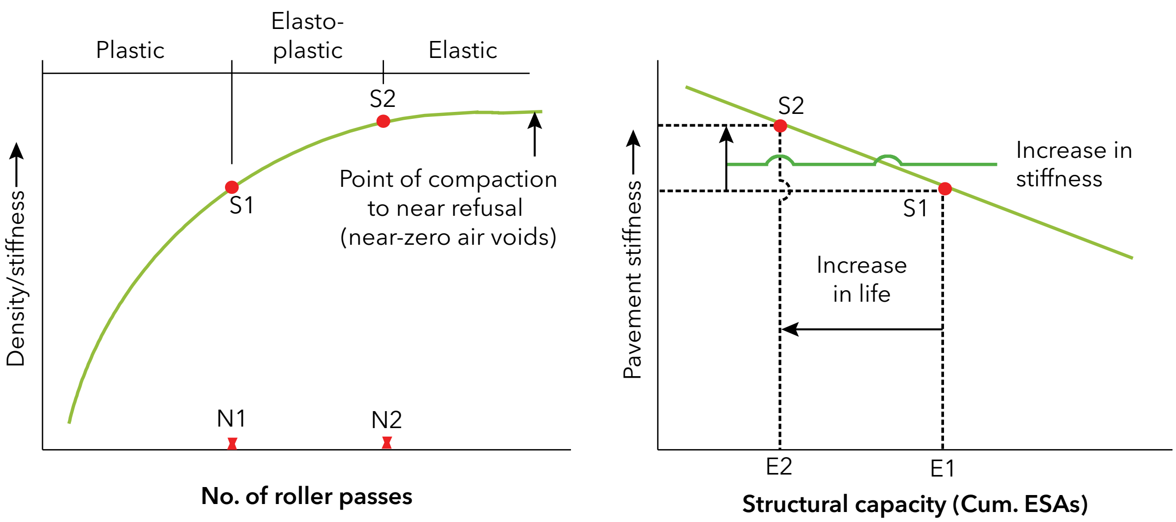
- Compaction
- Drainage
- Shoulders and cross section

4.4.2 Compaction

Effective compaction of the existing running surface of the gravel road which is to be upgraded is one of the most cost-effective means of improving the structural capacity of the LVR pavement. A well compacted running surface possesses enhanced strength, stiffness and bearing capacity, is more resistant to moisture penetration and less susceptible to differential settlement. The higher the density, the stronger the layer support, the less the required thickness of the overlying pavement layers and the more economical the pavement structure. Thus, there is every benefit to achieving as high a density and related strength as economically possible in the subgrade and pavement layers, and by assessing the density values obtained in relation to a heavy (e.g. Modified Proctor) rather than light (e.g. Standard Proctor) compaction standard.

Maximizing the strength potential of a subgrade soil can be achieved, not necessarily by compacting to a pre-determined relative compaction level, as is traditionally done but, rather, **by compacting with the heaviest equipment available to attain the highest uniform level of density possible ("compaction to near refusal") without significant strength degradation of the particles.** In so doing, there is a significant reduction in permeability as well as a beneficial gain in density, strength and stiffness, with the latter correlating directly with longer pavement life, as illustrated in Figure 29. For these compelling reasons, where the higher densities can be realistically attained in the field from field measurements on similar materials or other established information, they should be specified in the tender documents.

FIGURE 29: BENEFITS OF COMPACTION TO REFUSAL

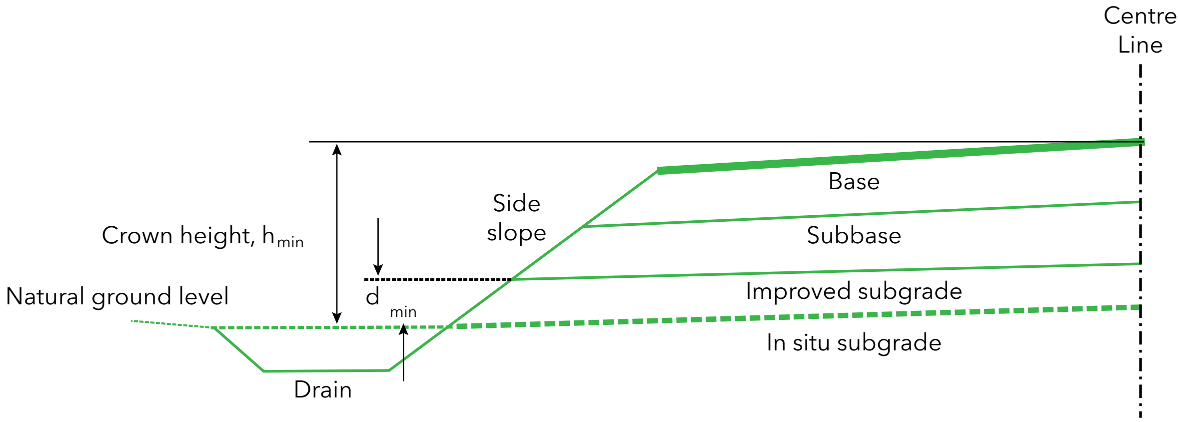


4.4.3 Drainage

Moisture is the single most important factor affecting pavement road performance and long-term maintenance costs. This factor is even more important for LVRs which are generally constructed from natural, often unprocessed, materials which tend to be more moisture sensitive than traditional materials. It is therefore necessary to provide a pavement structure in which the weakening effects of moisture are contained to acceptable limits in relation to the traffic loading, nature of the materials being used, construction/maintenance provisions and degree of acceptable risk.

The crown height of an LVR, i.e. the vertical distance from the bottom of the side drain to the finished road level at the center line (h_{min}), is a critical parameter that correlates well with the in-service performance of pavements constructed from naturally occurring materials[1]. This height must be sufficiently great to prevent moisture ingress into the potentially vulnerable outer wheel track of the carriageway for which a minimum value of 0.75 m is recommended (Figure 30).

FIGURE 30: MINIMUM DRAINAGE REQUIREMENTS



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 village areas, are shown in Table 12.

TABLE 12: RECOMMENDED CROWN HEIGHT IN RELATION TO DRAIN TYPE AND LONGITUDINAL GRADIENT

Crown height, h_{min} (meters)			
Unlined drains		Lined drains	
g < 1%	g > 1%	g < 1%	g > 1%
0.75	0.65	0.65	0.50

In water logged areas, where the subgrade is within the zone of capillary saturation, consideration should be given to the installation of suitable capillary cut-off as per IRC: 34 at an appropriate level underneath the pavement.

The height between the natural ground level and the bottom of the subbase (d min) shall be not less than 150 mm (Figure 30).

4.4.4 Shoulders

4.4.4.1 General

Shoulders fulfil several important functions which are enhanced when they are sealed – a feature which is strongly recommended, where possible. The advantages of sealing shoulders include:

- Provision of better support and moisture protection for the pavement layers and also reduces erosion of the shoulders (especially on steep gradients).
- Improved pavement performance by ensuring that the zone of seasonal moisture variation does not penetrate to under the outer wheel track.
- Reduced maintenance costs by avoiding the need for regravelling at regular intervals.

- Reduced risk of road accidents, especially where the edge-drop between the shoulder and the pavement is significant or the shoulders are relatively soft.

4.4.4.2 Width of Shoulder Sealing

This should be such as to ensure that any moisture variation is contained within the shoulders and does not penetrate the outer wheel track of the pavement. The recommended minimum paved shoulder width will depend on the extent of lateral infiltration into the shoulder at the wettest time of the year. The recommended minimum widths for a 2-lane road would be of the order of 1.0 m. However, narrower widths would also be advantageous on single-lane carriageways where vehicles tend to use the central portion of the road.

4.4.4.3 Type of Shoulder Surfacing

Ideally, the same type of surfacing used on the carriageway should be extended over the shoulder as well, particularly where it is likely to be heavily trafficked as in towns and built-up areas. Where a two-layer surface treatment is used on the carriageway, at least the first layer should be extended to the shoulder with an appropriate second cover seal which may be of a different type to that used on the carriageway. The objective is to achieve a durable, close-textured surfacing on the shoulder. This will generally require an increased binder application rate to cater for the lesser trafficking of shoulder compared with the carriageway.

5. SURFACINGS

5.1 INTRODUCTION

5.1.1 Background

The surfacing of any road plays a critical role in its long-term performance. It:

- Prevents gravel loss
- Eliminates dust from unpaved roads
- Improves skid resistance, and
- Reduces water ingress into the pavement once surfaced.

The latter attribute is especially important for LVRs where moisture sensitive materials are often used.

There are many 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, and the environment. Careful consideration should therefore be given to all these factors in order to make a judicious choice of the most suitable type of surfacing to provide satisfactory performance and minimize life cycle costs.

5.1.2 Purpose and Scope

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

- The various types of bituminous and non-bituminous surfacings available for use on LVRs.
- The factors that affect the selection of these surfacings.

5.2 BITUMINOUS SURFACINGS

5.2.1 General

Bituminous surfacings for purposes of these Guidelines are divided into surface treatments (often referred to as surface dressings) and asphalt surfacings, as shown in Figure 31.

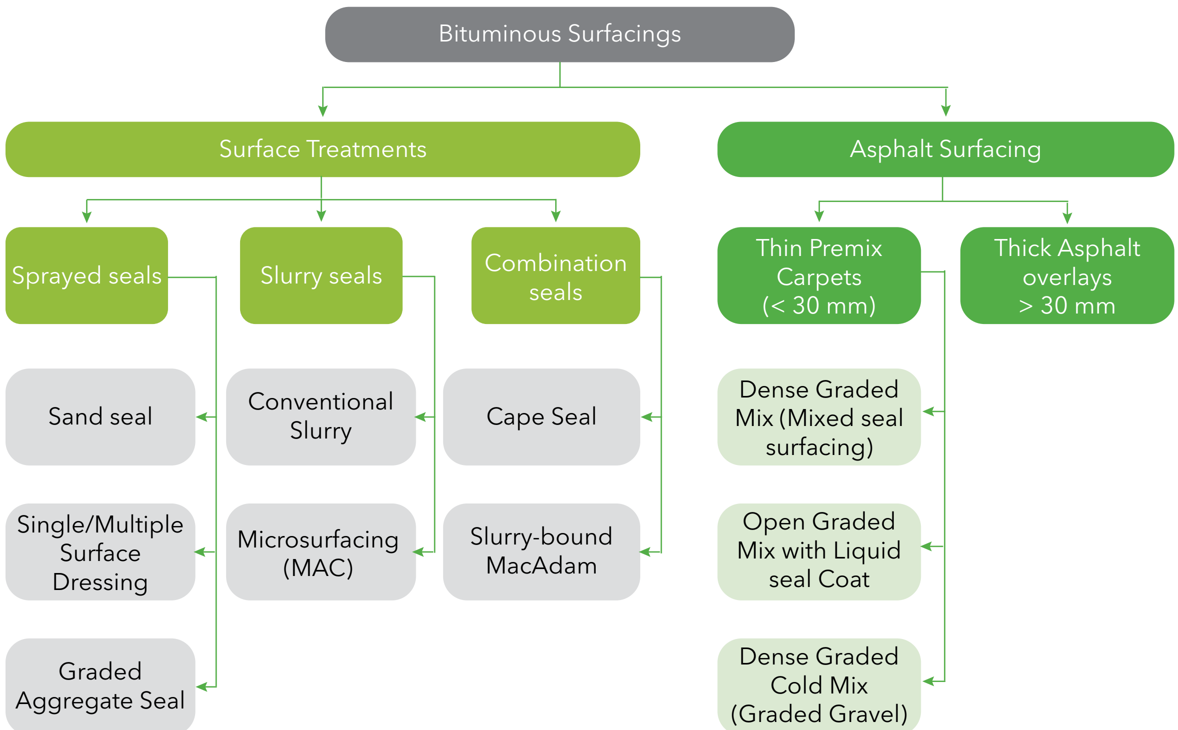
5.2.2 Surfacing Types

As shown in Figure 31, various options for bituminous surfacings are available for low volume roads. These can basically be classified as follows with various configurations within each of these groups.

- Asphalt surfacings
- Surface treatments
  - Sprayed Seals
  - Slurry (conventional slurries, microsurfacings)
  - Combination seals and are mainly used on low volume roads.

The design of surface treatments (surface dressings) is addressed in a companion IRC document on the Design, Construction and Maintenance of Surface treatments.

FIGURE 31: BITUMINOUS SURFACINGS FOR LVR

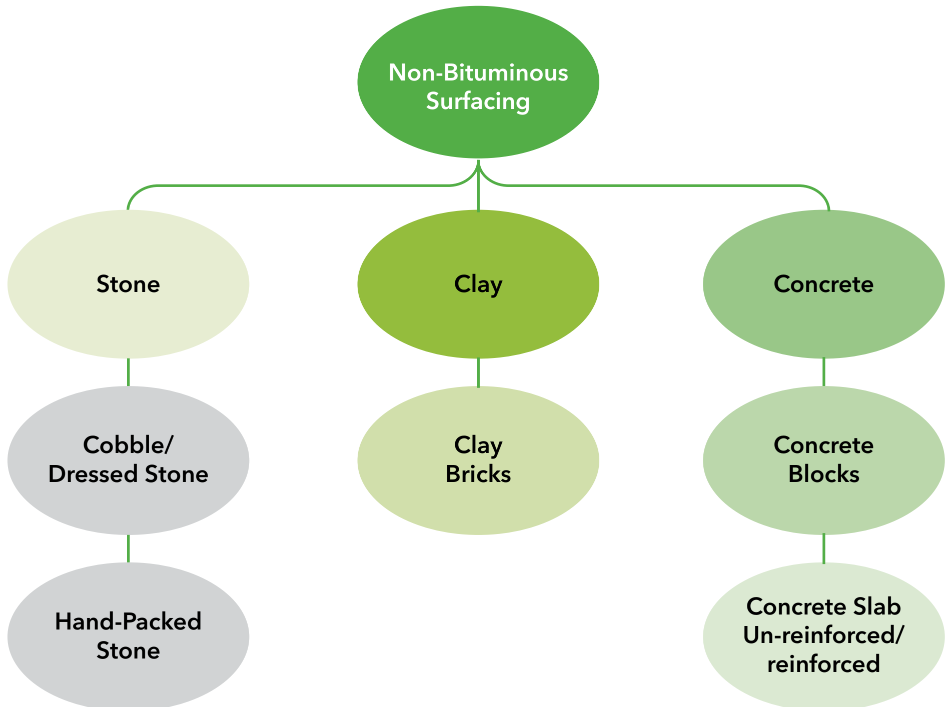


5.3 NON-BITUMINOUS SURFACINGS

5.3.1 General

Surfacings constructed from materials such as stone, clay and concrete can also be considered for use on LVRs instead of conventional bituminous surfacings. The current practice of utilizing concrete pavements through villages, at very steep grades or where there is risk of water overtopping the road is an example of an environmentally judicious choice of surfacing in circumstances where bituminous surfacings often do not perform well.

FIGURE 32: EXAMPLES OF TYPICAL NON-BITUMINOUS SURFACINGS FOR LVRs



5.3.2 Surfacing Types

The main types of non-bituminous surfacings can be divided into three distinct groups as shown in Figure 32.

The non-bituminous surfacings listed 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, 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.

5.4 SURFACING TYPE SELECTION

5.4.1 Selection criteria

The selection of the best option depends on a wide range of factors with the main influencing factors affecting the choice as follows:

- Material quality and availability.
- Traffic volume and vehicle type distribution.
- Cost effectiveness (cost and expected service life).

- Purpose and life-cycle strategy.
- Safety and contractual requirements.
- Social and environmental requirements and impacts.
- Institutional capacity and maintenance capability.
- Urban or rural drainage systems.
- Construction method and risks.
- Road gradients (see Table 13).
- Turning actions/external stresses (see Table 14).
- Base or existing substrate quality/macro texture (see Table 15).

The key physical factors that exert a strong influence on the choice of appropriate surfacing for an LVR are discussed briefly below.

5.4.2 Gradient

Table 13 indicates the categories of gradient that affect the choice of surfacing, as illustrated in Table 16.

TABLE 13: CONSTRUCTION GRADIENT

Very steep	>10%
Steep	6–10%
Flat -Gentle	0–6%

5.4.3 Turning actions/external stresses

Table 14 indicates the severity of turning actions/external stresses that affect the choice of surfacing, as illustrated in Table 16.

TABLE 14: EXTERNAL STRESSES

Very high	High risk of water overflowing, high risk of farm or industrial equipment damage, high risk of landslides and subsequent material removal
High	High occurrence of heavy vehicles turning/braking, high probability of loose material on the road surface and slight probability of the risks mentioned under "Very high"
Medium	Occasional turning/braking of heavy vehicles, possibility of loose material and very low probability of risks mentioned under "Very high"
Low	Very low risk of damage due to external stresses mentioned above

5.4.4 Macrotexture

The macrotexture of a pavement refers to the visible roughness of the pavement surface and is defined as texture ("bumps and dips") in a pavement with a wavelength (distance from "bump" to "bump") ranging from 0.5 mm to 50 mm.

Single and double surface dressings are particularly sensitive to coarse textures due to the applied binder running into the voids of the existing surface and not properly adhering to the surfacing aggregate. Fine slurry is often applied as a pre-treatment (Void fill) to obtain a uniform fine texture before the final surface treatment is applied.

Table 15 categorises macrotexture in terms of texture depth as a basis for selecting an appropriate type of surfacing, as illustrated in Table 16.

TABLE 15: TEXTURE DEPTH BY TYPE OF MACROTEXTURE

Macrotexture	Texture Depth (mm)	Typical Base Type
Very coarse	> 5	WBM or excessively brushed crushed stone
Coarse	2-5	Well brushed crushed stone
Medium-Fine	< 2	Well brushed natural gravel

5.4.5 Preliminary selection guideline

Given the current practice in India, the drive towards surface treatments (surface dressings), the scarcity and high transport cost of crushed aggregate in some states and availability of clay, natural gravels and sand, Table 16 has been developed as a guide for the selection of appropriate initial construction surfacings.

The selection of potential surfacing types is based on:

- Availability of materials\
- Gradients at which construction will take place (Table 13)
- External stresses expected on the road (Table14)
- The macro texture of the base before surfacing (Table 15)

Based on risks of poor performance, only the following bituminous surfacings are considered for adoption as initial construction seals on LVRs:

- Asphalt premix
  - 20 mm Premix Carpets
  - 30–40 mm Continuous Graded Asphalt (Cold or hot mix)
  - 25–40 mm Graded Gravel Mix (Cold mix)
- Surface treatments
  - Cape seals
  - Otta seals (Graded aggregate seals)
  - Slurry-bound Macadam (A 20 – 30 mm layer of 13.2 mm aggregate with a fine slurry vibrated into the voids and covered with a final fine slurry layer)
  - Low-risk double seals or Stone plus sand seals (where suitable coarse sand is available). A low-risk double seal is defined as a double seal, where the second layer aggregate is a third or less the size of the first aggregate layer size
  - Thick slurry seals or Microsurfacing
  - Double "graded sand" seals

The choice of one or combination of these seal types is mainly dependent on the existing macro texture of the base and availability of suitable aggregate.

If the base is coarse or very coarse, only double Otta seals, thick coarse slurry/ Microsurfacing or a void filling slurry seal in combination with a Cape seal or double seal should be considered.

The type and grade of coarse slurry (Slurry Type II or Type III) appropriate for use as a void filling seal depends on the depth and volume of voids to fill.

The direct application of a Cape seal or double seal should only be considered if the base macro texture could be described as medium to fine (Volumetric texture depth < 2 mm).

TABLE 16: GUIDE FOR SELECTION OF LVR BITUMINOUS SURFACINGS

External Stresses	Gradient	Macro Texture	Appropriate Bituminous Surfacings											Min. Thickness
Very High	→	→	No Conventional Bituminous Surfacings recommended											
High	Very Steep			No Conventional Bituminous Surfacings < 40 mm recommended										40 mm
	Steep	→	AC											
	Flat-Gentle	→	AC											
Medium	Very Steep	Very Coarse	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	30 mm
		Coarse	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
	Steep	Very Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	20 mm
		Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	20PMC	Otta			SL/M	SBMac	CS	DS	DGSS	
	Flat-Gentle	Very Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	20PMC	Otta			SL/M	SBMac	CS	DS	DGSS	
Low	Very Steep	Very Coarse	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	30 mm
		Coarse	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	30PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
	Steep	Very Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	20 mm
		Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	20PMC	Otta			SL/M	SBMac	CS	DS	DGSS	
	Flat-Gentle	Very Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	15 mm
		Coarse	AC	CMGG	20PMC	Otta	VF+CS	VF+DS	SL/M	SBMac	CS	DS	DGSS	
		Medium-Fine	AC	CMGG	20PMC	Otta			SL/M	SBMac	CS	DS	DGSS	

AC = 30–40 mm Asphalt (Premix) Continuous Graded
CMGG = 25–30 mm Cold Mix Graded Gravel
PMC = 20–30 mm Premix Carpet
Otta = Double Otta seal

VF = Void filling slurry seal
CS = Cape Seal
SL/M = Thick slurry or Microsurfacing

SBMac = Slurry-bound Macadam
DS = Double seal
DGSS = Double graded sand seal

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ANNEX-I: Example of Design Traffic Determination

BASIC DATA

1. Design life = 10 years
2. Annual growth rate of traffic for all categories of commercial traffic = 6%
3. Two harvesting seasons in the area
4. Traffic data collected 1 year before opening the road to traffic
5. Traffic counting conducted over a period of 3 days during the lean season
6. Duration of each harvesting season = 75 days
7. Results of traffic survey data presented in Table 1.

TABLE 1: RESULTS OF TRAFFIC SURVEY (Refer Figure 7 in Chapter 3)

(a) Average Daily Traffic as per last count				**No.**
(Lean Harvesting Season)				
(i)	Cars, Jeeps, Vans, Three Wheelers		=	29
(ii)	Motorised Two Wheelers		=	25
(iii)	Light Commercial Vehicles		=	29
(iv)	Heavy Commercial Vehicles (HCV)			
	(A)	Trucks		
		Loaded	=	2
		Unloaded	=	2
		Overloaded	=	0
	(B)	Buses		
		Loaded	=	2
		Unloaded	=	2
		Overloaded	=	0
(v)	Agricultural Tractor Trailers (MCV)			
	Loaded		=	14
	Unloaded		=	14
	Overloaded		=	0

(vi)	Cycles	=	0
(vii)	Cycle Rickshaws	=	0
(viii)	Animal Drawn Vehicles	=	5
		Total	T=124

b. Computation of the Design Traffic

(i) Harvesting Seasons in the area = 2 No.

(ii) Duration of each Harvesting Season (t) = 75 Days

(iii) Average Daily Traffic during the Lean Season (T) = 124 No.

(iv) Annual Average Daily Traffic (AADT)

AADT = T + (1.2nTt)/365

= 124 + (1.2 x 1 x 124 x 75)/365

= 124 + (11160)/365

= 124 + 30.6

= 154.6 say 155

(v) AADT before opening to traffic = $155 \times (1.06)^1$

= 164.3 say 164

(vi) Proportion of HCV

(a) Laden = (164 x 4)/124 = 5.30 say 6

(b) Unladen = (164 x 4)/124 = 5.30 say 6

(c) Overloaded = 0

(vii) Proportion of MCV

(a) Laden = (164 x 14)/124 = 18.52 say 19

(b) Unladen = (164 x 14)/124 = 18.52 say 19

(c) Overloaded = 0

(viii) Animal drawn carts have not been considered for design purposes

(ix) T_0 for HCV = 6 x 2.60 + 6 x 0.31

= 17.46 say 18

(x) T_0 for MCV = 19 x 0.31 + 19 x 0.02

= 6.27 say 7

(xi) Total for HCV + MCV = 25

(xii) Cumulative ESAL applications (N) over the design life (10 years) @ 6% growth rate

N = T_0 x 4811 x L

= 25 x 4811 x 2

= 240,550 ESA

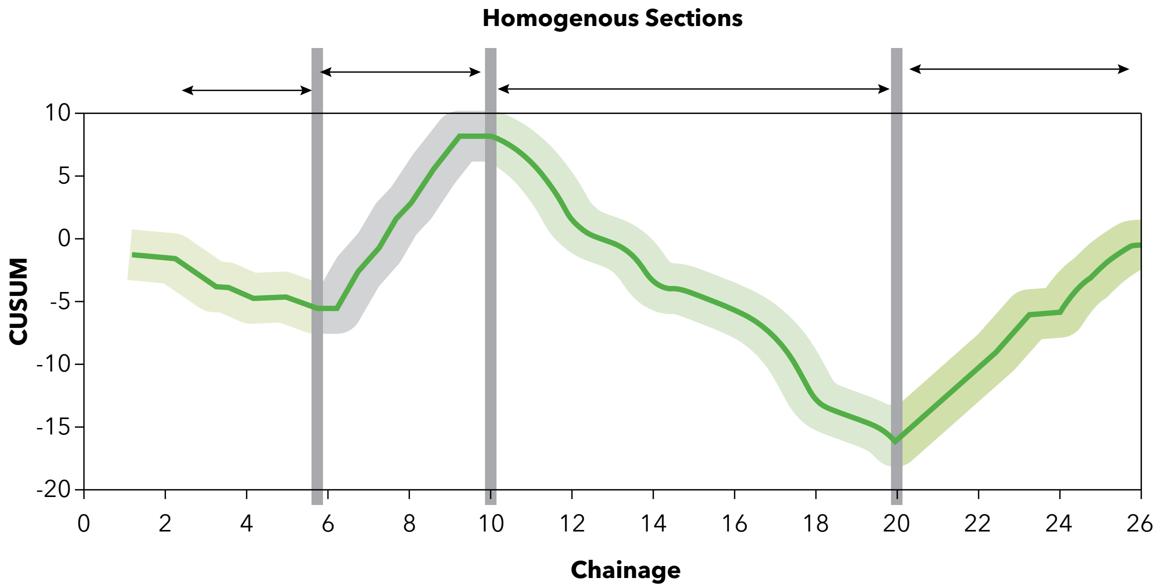
= 0.241 MESA

(xiii) Traffic category = T_4 (0.100 – 0.300) MESA

ANNEX-II: Determination of Uniform Sections from Cusum Analysis

1. CUSUM ANALYSIS

Chainage (km)	B Measured DCP (DN Value-mm/blow)	C Difference from average (A-B)	CUSUM (Accumulated Values of C)
1	14	-1.2	**-1.2**
2	13	-0.2	**-1.4**
3	15	-2.2	**-3.6**
4	14	-1.2	**-4.8**
5	13	-0.2	**-5.0**
6	14	-1.2	**-6.2**
7	7	5.8	**-0.2**
8	9	3.8	**3.4**
9	8	4.8	**8.2**
10	13	-0.2	**8.0**
11	15	-2.2	**5.8**
12	18	-5.2	**0.6**
13	14	-1.2	**-0.6**
14	16	-3.2	**-3.8**
15	14	-1.2	**-5.0**
16	14	-1.2	**-6.2**
17	15	-2.2	**-8.4**
18	18	-5.2	**-13.6**
19	14	-1.2	**-14.8**
20	15	-2.2	**-17.0**
21	9	3.8	**-13.2**
22	10	2.8	**-10.4**
23	9	3.8	**-6.6**
24	12	0.8	**-5.8**
25	9	3.8	**-2.0**
26	11	1.8	**-0.2**
Average: A = 12.8			


Homogenous Sections
10
5
0
-5
-10
-15
-20
CUSUM
0
2
4
6
8
10
12
14
16
18
20
22
24
26
Chainage

ANNEX-III: Determination and Choice of DN Percentile Values

1. **DCP Survey results–Uniform section derived from CUSUM analysis of DN 0-150 (Base)**
(N.B. DN 0-150 = DN in first 150 mm of pavement.

<table>
<tr><th rowspan="2">Chainage (km)</th><th rowspan="2">Point No</th><th rowspan="2">DN 0-150 (Base)</th><th colspan="3">Percentile of minimum strength Profile (max. penetration rate – DN)</th></tr>
<tr><th>20th</th><th>50th (Mean)</th><th>80th</th></tr>
<tr><td>0.00</td><td>1</td><td>2.29</td><td rowspan="10">3.46**</td><td rowspan="10">5.24</td><td rowspan="10">8.19</td></tr>
<tr><td>0.25</td><td>2</td><td>4.44</td></tr>
<tr><td>0.50</td><td>3</td><td>2.00</td></tr>
<tr><td>0.75</td><td>4</td><td>8.67</td></tr>
<tr><td>1.00</td><td>5</td><td>3.75</td></tr>
<tr><td>1.25</td><td>6</td><td>8.07</td></tr>
<tr><td>1.50</td><td>7</td><td>5.11</td></tr>
<tr><td>1.75</td><td>8</td><td>5.37</td></tr>
<tr><td>2.00</td><td>9</td><td>6.60</td></tr>
<tr><td>2.25</td><td>10</td><td>10.12</td></tr>
<tr><td colspan="3">Anticipated long-term in-service moisture content in pavement*</td><td colspan="3"></td></tr>
<tr><td colspan="3">Drier than at time of DCP survey</td><td>3.46</td><td>N/A</td><td>N/A</td></tr>
<tr><td colspan="3">Same as at time of DCP survey</td><td>N/A</td><td>5.24</td><td>N/A</td></tr>
<tr><td colspan="3">Wetter than at time of DCP survey</td><td>N/A</td><td>N/A</td><td>8.19</td></tr>
</table>

* This is one of the most carefully considered decisions the designer will have to make to ensure that a reliable DCP design is achieved. See Section 2.5.5 for the basis on which the decision should be made.

** The percentile value in an Excel spreadsheet may be obtained from the expression: = PERCENTILE (N$1:N$10,0.2) where N is the column containing the DN values.

2. Definition of Percentile

A percentile of a range of values is the point in the range at or below which a given percentage of values is found. For example, the 80th percentile of the distribution of DN values given in the above example is the point at or below which 80% of the values fall, i.e. 8.19, as illustrated below.

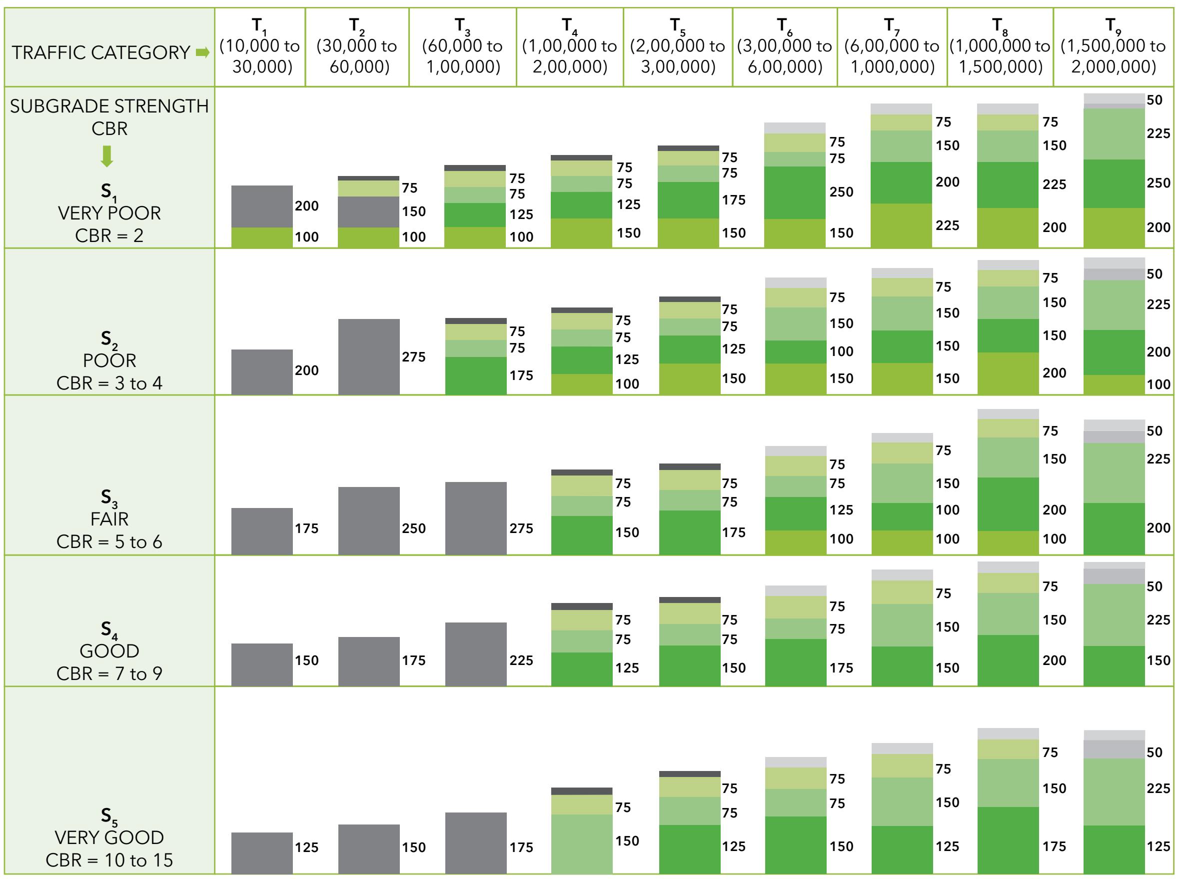
DN value	% ile*	DN %ile	Anticipated long-term in-service moisture content in pavement
2.00			
2.29			
3.75	20^{th}	3.36	Drier than at time of DCP survey
4.44			
5.11			
5.37	50^{th}	5.24	Same as at time of DCP survey
6.60			
8.07			
8.67	80^{th}	8.19	Wetter than at time of DCP survey
10.12			

* Percentile value may need to be adjusted for moisture sensitivity of material.

ANNEX-IV: Comparison of DCP-DN and IRC CBR Catalogues

It is informative to compare the structural capacities, materials requirements and pavement balance of some of the existing Indian catalogue designs and the DCP-DN designs.

The current catalogue in IRC SP72: 2015 is shown below.



- Modified Soil/Improved Subgrade (CBR not < 10)
- Granular Subbase (CBR not < 20) in exceptional care can be 15
- Gravel Base (CBR not < 80). in Lower base course shall not be less than 50 Clause 2.3.5 (in exceptional case may be relaxed suitably)
- Base of Gravel/CRMB/WBM (CBR not < 100 Where 100mm thickness is recommended it be modified to 75 mm for WBM with corresp. Increase of 25 mm in Subbase
- WBM Grade-3
- Bituminous Macadam
- Surface Dressing
- OGPC

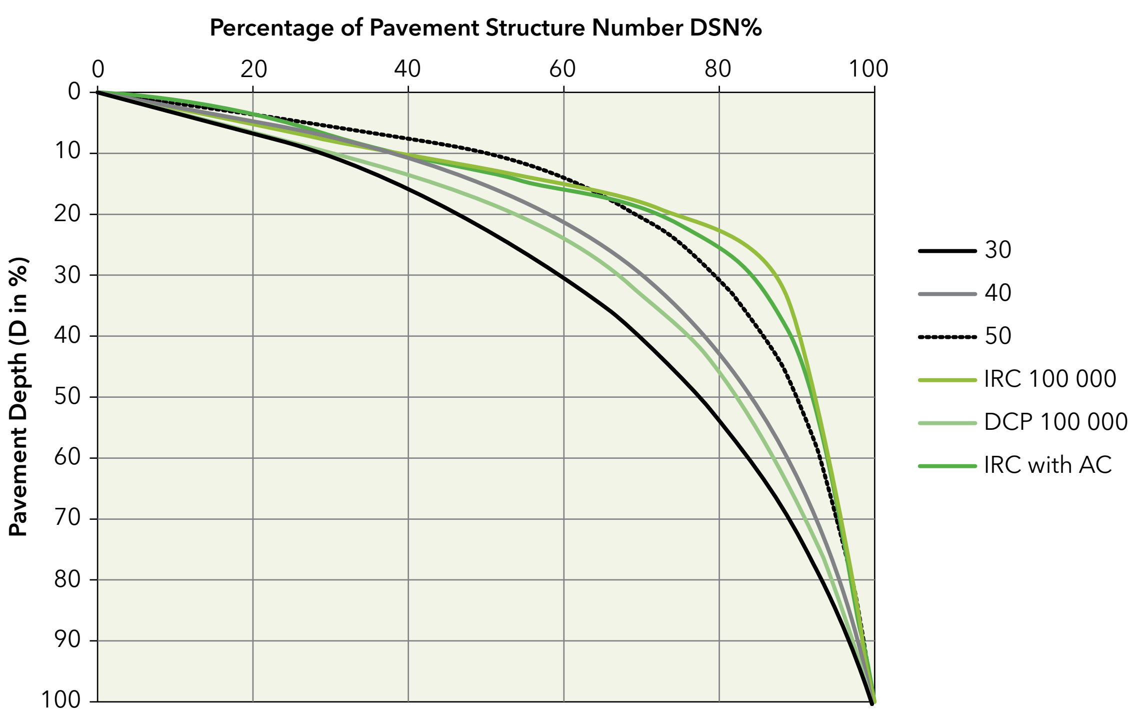
For less than 1,00,000 standard axles

There are only two designs for paved roads carrying up to 1,00,000 standard axles and they are for weak subgrades of 2-4% soaked CBR.

In the example below, the IRC SP72 catalogue for 1,00,000 axles and a subgrade CBR of 3% is compared with the DCP-DN design for similar traffic. The subgrade CBR is automatically included in the DCP design profile. CBR values according to the IRC catalogue have been allocated and converted to DCP DN values. These have then been distributed through the pavement profile according to the layer thicknesses as shown in the table below.

Thickness (mm)	CBR (%)	DN (mm/bl)	Blows/layer	Blows/layer (no AC)	Blows/layer
IRC SP72 Catalogue for < 100 000 esa and SG CBR = 3%					DCP DN catalogue < 100, 000 esa
20	200	1.5	13		
75	100	3	25	25	37.5
75	100	3	25	25	16.5
175	20	14	12.5	12.5	8
455	3	60	7.6	7.6	6
					5
DSN_{800}			**83**	**70**	**73**

The DSN800 of the IRC design is 83 (i.e. 83 DCP blows would be required to penetrate 800 mm into the pavement) including the 20 mm AC layer. However, this thin layer is used primarily to protect the pavement structure and generally contributes little to the structural capacity of the road. If the theoretical strength contribution of the AC is ignored, the DSN800 then becomes 70 which is very close to the 73 of the DCP design method. In other words, the two structures are almost identical in terms of theoretical structural capacity (the IRC design is very slightly weaker).



However, if the strength distribution of the two structures are plotted with depth and compared with the theoretical balance curves as shown below, the DCP DN design shows a well-balanced structure with the ideal balance number for a granular pavement of between 30 and 40. The IRC design shows a shallow pavement with a balance number of higher than 50 (i.e. a pavement structure that is extremely sensitive to overloading) with a number of stress concentrations in the upper 100 mm. The structural effect of the asphalt can also be seen to be negligible, slightly worsening the balance.

Example of pavement design for different layer thickness

Although the standard catalogue for the DCP design method uses layers of 150 mm (primarily to overcome problems often seen on site with layer thickness control), the method can be adapted for any selected layer thicknesses or materials available. Essentially, the DSN_{800} values for the different road categories relate the required structural capacity of the road to the design traffic. This DSN_{800} is distributed through the layer thicknesses selected and the materials available as shown below. One of the disadvantages of moving away from the standard designs, however, is that the pavement balance could move away from the optimum.

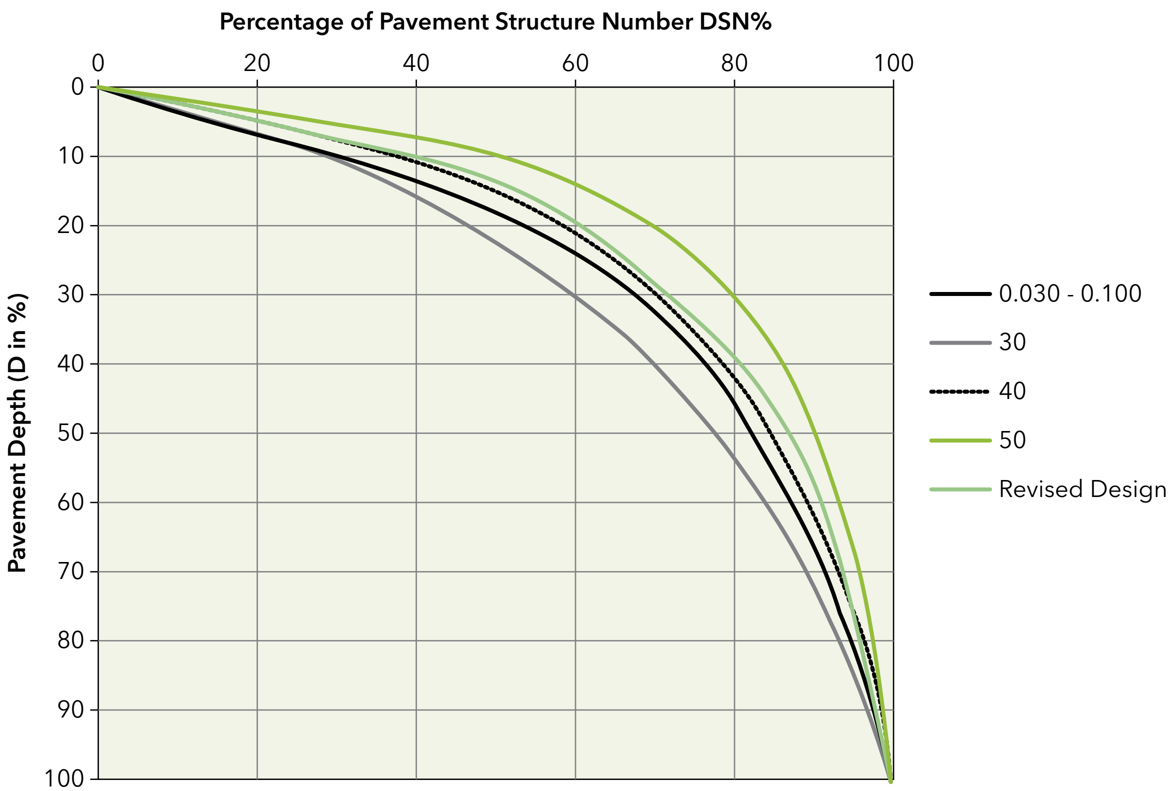
Theoretical Example

Granular base material with an unsoaked CBR value of 95% (soaked CBR of 45%) is available for a road being designed to carry 300 000 standard axles. However, no suitable sub-base material is available and it is planned to use the same base material with a reduced thickness for the subbase. What thickness is appropriate using an environmentally optimised design where the base and subbase material will operate at optimum moisture content and what will happen to the pavement balance? The existing layers in the road conform to the lower layer requirements for this design, but the capping layer has a CBR of 25%. The pavement structure and DSN_{800} for a 3,00,000 esa road are shown below.

Layer	Required DN value (mm/blow)	Revised design
Base – 150 mm	3.2	3.1
Subbase – 150 mm	6	3.1 (thickness reduced)
Selected/capping layer – 150 mm	12	9
Subgrade – 150 mm	19	19
Subgrade – 200 mm	25	25

Layer	Thickness (mm)	DN (mm/bl)	Blows/layer
Base	150	3.1	48
Subbase	?	?	(19.4)
Capping	150	9	16.6
Subgrade 1	150	19	8
Subgrade 1	200	25	8
DSN_{800}			100

By dividing the layer thicknesses by the DN values the number of blows per layer can be determined. The sum of these should be 100 for the required design (i.e. the DSN_{800}) but without the subbase make up only 80.2% of the requirement. The subbase thus needs to make up the difference, which is the 9.4 deficit multiplied by the DN value (3.1 In this case) giving a thickness of 60 mm. As this layer is too thin to work effectively, the road should be constructed with two 105 mm layers making up the base and subbase. The impact of this design on the pavement balance is shown below:



It is evident that by increasing the strength of the upper portion, the balance number increases slightly but the pavement is still a well-balanced relatively deep structure.

The main advantages of the DCP DN design method illustrated above are that:

- The pavement balance is significantly improved and less overloading sensitive.
- Much weaker materials can be used for the upper layers, i.e. natural gravels instead of crushed aggregate (water-bound or wet-mix macadam).
- It is much easier to design for unsoaked conditions, but soaked conditions can be equally simply handled.
- Offers improved design reliability due to the much larger data set of DCP-DN measurements for statistical analysis and pavement design based on discrete uniform sections rather than general blanket designs.

ANNEX-V: DCP Design Example

1. Project Details

- Project name: Alpha-Beta Road
- Road length: 4.4 km
- DCP survey carried out in dry season

2. Design Procedure

- As per section 4.3.2.1

3. Step 1 – Design Period

- Design period = 15 years (only for illustrative purposes)

4. Step 2 – Design Traffic

- 0.241 MESA (Annex-I – Determination of Design Traffic Loading)

5. Step 3 – Design Traffic

- Traffic Class = T_4 (0.10 – 0.30 MESA) (see Annex I)

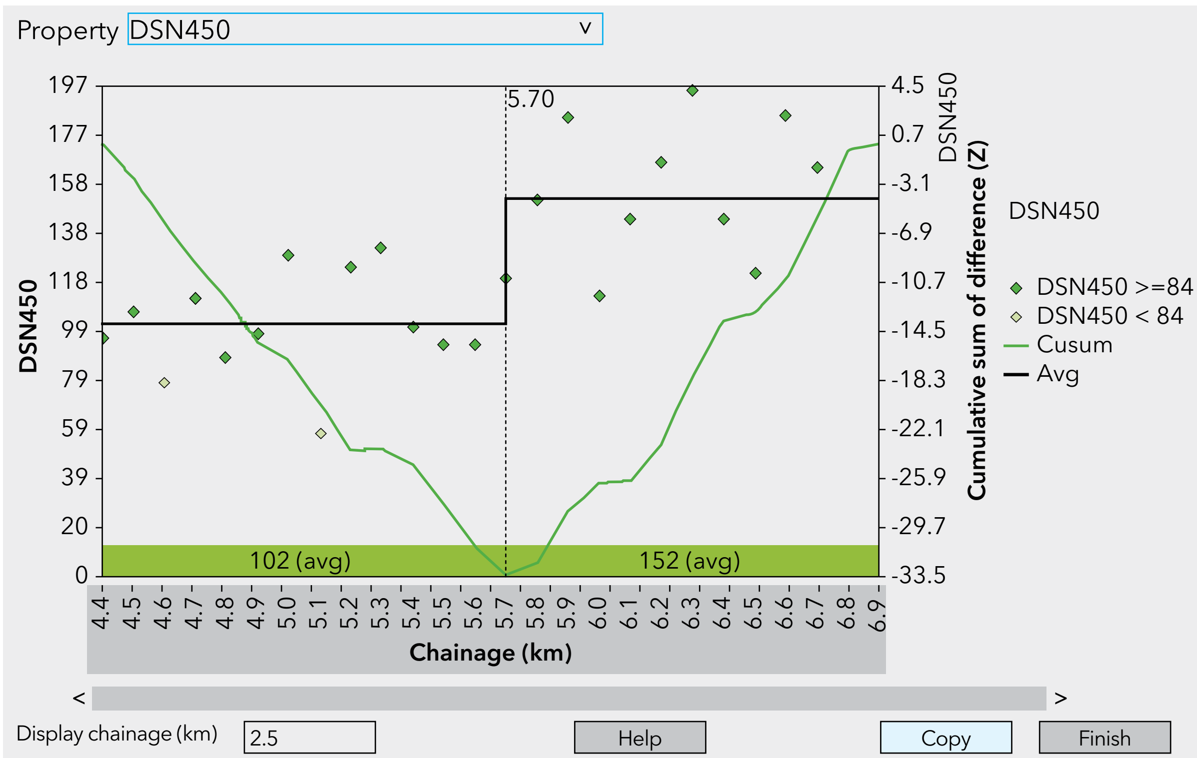
After entering all the DCP data, the AfCAP LVR DCP programme calculates all the DSN and DN values for each test point as shown below. DSN450/800 is the number of blows to penetrate to a depth of 450mm and 800mm respectively. The DN values are the weighted averages (or 20th/80th percentiles) of the penetration/blow through each 150mm layer.

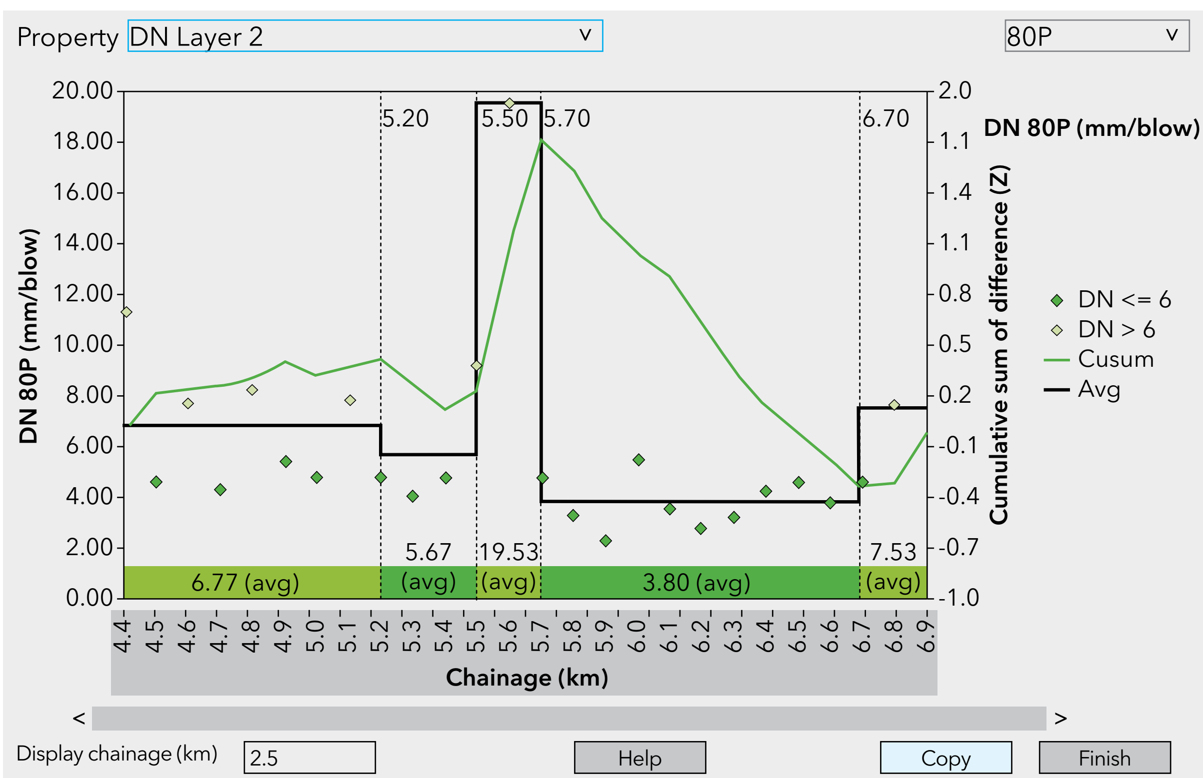
Averages	Weighted Average
Percentiles	Normal Distribution

DCP Test Point nr	DCP Test Point Name	Survey date	Chain-age (km)	Road Side	Distance from CL (m)	DSN450 (Blows)	DSN800 (Blows)	Radio (%)	Weighted Average DCP Number, DN in mm/blow														
									0-150 mm			151-300 mm			301-450 mm			451-600 mm			601-800 mm		
									20P	Mean	80P	20P	Mean	80P	20P	Mean	80P	20P	Mean	80P	20P	Mean	80P
1	Test 1	11/07/2016	4,4	LHS	0,00	95	116	82	1,540	2,690	3,830	7,460	9,370	11,290	16,600	17,260	17,920	18,570	20,350	22,130	12,440	16,390	20,330
2	Test 2	11/07/2016	4,5	RHS	2,00	106	142	75	2,760	4,420	6,080	3,170	3,880	4,590	5,580	6,140	6,690	8,190	8,850	9,510	10,230	10,790	11,360
3	Test 3	11/07/2016	4,6	CL	2,00	78	124	63	4,970	6,170	7,380	4,530	4,110	7,690	5,250	5,990	6,740	6,720	6,850	6,980	7,410	8,220	9,020
4	Test 4	11/07/2016	4,7	LHS	0,00	111	172	65	3,380	3,810	4,250	3,900	4,410	4,290	3,980	4,250	4,520	4,890	5,230	5,560	5,530	6,480	7,430
5	Test 5	11/07/2016	4,8	RHS	2,00	87	137	64	2,680	3,390	5,160	5,340	6,810	8,270	5,660	6,640	7,620	6,090	6,920	7,740	5,540	7.940	10,330
6	Test 6	11/07/2016	4,9	CL	2,00	97	143	68	2,880	4,110	5,350	4,050	4,750	5,460	5,760	5,930	6,100	6,660	7,520	8,370	6,720	8,190	9,670
7	Test 7	11/07/2016	5	LHS	0,00	128	183	70	2,090	2,840	3,590	2,820	3,780	4,740	5,060	5,180	5,290	4,880	5,570	6,260	6,450	7,620	8,800
8	Test 8	11/07/2016	5,1	RHS	3,00	57	97	59	6,860	7,830	8,800	6,680	7,250	7,810	8,280	8,800	9,320	8,180	8,970	9,950	9,750	8,870	10,180
9	Test 9	11/07/2016	5,2	CL	2,00	124	248	50	2,390	3,100	3,810	2,870	3,800	4,740	5,000	5,540	6,080	3,940	4,860	5,770	1,900	2,220	2,550
10	Test 10	11/07/2016	5,3	LHS	0,00	131	180	73	2,070	2,870	3,660	2,690	3,350	4,010	5,170	5,830	6,480	6,120	6,530	6,950	-0,0540	11,730	24,000
11	Test 11	11/07/2016	5,4	RHS	2,00	100	155	65	3,920	4,610	5,290	3,720	4,250	4,780	4,600	4,910	5,230	5,860	6,090	6,320	6,050	6,670	7,280
12	Test 12	11/07/2016	5,5	CL	2,00	92	138	67	2,850	3,780	4,720	5,150	7,150	7,150	9,150	5,480	5,890	6,300	6,060	6,820	7,590	8,720	9,820
13	Test 13	11/07/2016	5,6	LHS	0,00	92	170	54	-0,400	8,550	17,510	2,910	11,220	19,530	4,140	4,500	4,860	4,520	4,660	4,790	4,060	4,450	4,840
14	Test 14	11/07/2016	5,7	RHS	2,00	119	171	70	2,590	3,620	4,650	3,050	3,910	4,770	4,280	4,860	5,450	5,730	6,100	6,460	6,430	7,720	9,010
15	Test 15	11/07/2016	5,8	CL	2,00	151	209	72	2,070	2,420	2,760	2,440	2,870	3,310	4,140	4,620	5,100	5,200	5,370	5,550	6,090	6,590	7,090
16	Test 16	11/07/2016	5,9	LHS	0,00	183	229	80	1,890	3,080	4,270	1,670	1,980	2,3000	2,650	3,380	4,110	8,610	13,000	17,390	6,010	6,570	7,140
17	Test 17	11/07/2016	6	CL	2,00	112	151	74	1,950	2,510	3,070	3,620	4,560	5,510	7,600	15,630	23,670	8,270	9,050	9,830	8,400	9,190	9,970
18	Test 18	11/07/2016	6,1	RHS	2,00	143	194	74	2,000	2,680	3,370	2,580	3,050	3,520	4,390	4,900	5,410	4,560	6,770	8,970	7,030	8,410	9,790
19	Test 19	11/07/2016	6,2	CL	2,00	166	213	78	2,070	3,030	3,990	1,840	2,290	2,750	3,540	3,830	4,130	5,100	6,040	6,970	7,870	10,320	12,770
20	Test 20	01/07/2016	6,3	LHS	0,00	195	258	75	1,160	1,850	2,540	2,240	2,710	3,170	4,080	4,450	4,810	4,850	5,220	5,600	5,460	5,830	6,200
21	Test 21	18/07/2016	6,4	CL	2,00	143	187	76	1,630	2,180	2,730	2,960	3,580	4,210	5,250	5,760	6,270	6,860	7,170	7,480	8,110	8,660	9,210
22	Test 22	18/07/2016	6,5	LHS	0,00	122	168	72	2,110	3,320	4,520	3,000	3,790	4,590	5,490	6,550	7,610	6,320	7,560	8,800	6,580	8,190	9,800
23	Test 23	18/07/2016	6,6	RHS	2,00	184	221	84	1,240	1,680	2,120	2,690	3,260	3,820	5,020	5,820	6,610	6,440	7,830	9,230	11,480	12,640	13,800
24	Test 24	11/07/2016	6,7	CL	2,00	163	195	84	1,290	1,940	2,590	2,150	3,360	4,580	6,190	7,420	8,650	7,940	9,340	10,740	11,760	14,740	17,730
25	Test 25	11/07/2016	6,8	LHS	0,00	196	263	75	1,390	2,080	2,760	2,030	4,830	7,630	1,960	2,550	3,140	3.390	3,750	4,110	5,920	8,490	11,060
26	Test 26	11/07/2016	6,9	RHS	3,00	67	83	80	3,520	3,940	4,360	7,330	8,860	10,390	13,620	14,210	14,800	17,480	19,770	22,060	21,120	23,190	25,260

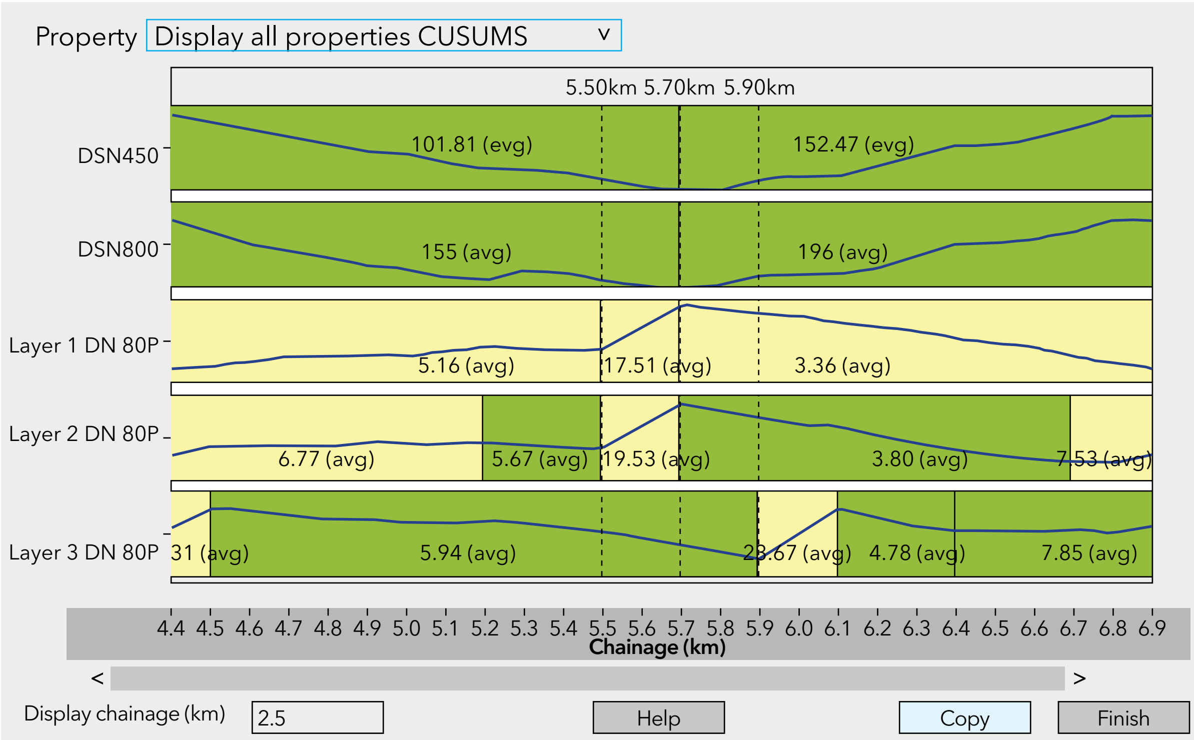
In this example, the DCP tests were carried out in the dry season, hence 80th percentile DN values were used for the pavement design assuming that the existing pavement may be wetter (i.e. weaker) in service than when the DCP tests were carried out.

As shown in the following screenshots, the software then allows the user to determine uniform sections based on a Cumulative Sum analysis of the data, both for the DSN values and each layer separately.

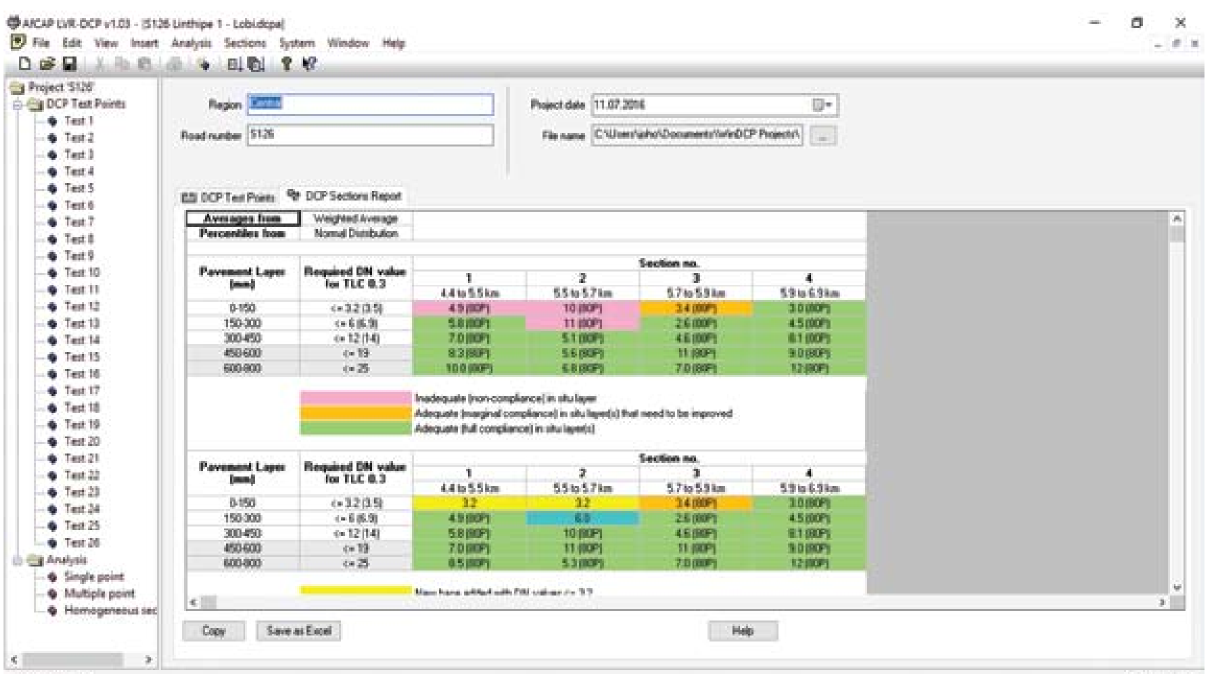




Following the separate analysis of the DSN and DN values, the user can then determine uniform sections on an overall assessment of all properties as shown in the next screenshot. In this case four sections have been determined.



On this basis, the AfCAP LVR DCP software produces the design report as shown below. The design report can be exported to Excel for incorporation in reports.



Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1: 4.4 to 5.5 km	2: 5.5 to 5.7 km	3: 5.7 to 5.9 km	4: 5.9 to 6.9 km
0-150	<= 3.2 (3.5)	4.9 (80P)	10 (80P)	3.4 (80P)	3.0 (80P)
150-300	<= 6 (6.9)	5.8 (80P)	11 (80P)	2.6 (80P)	4.5 (80P)
300-450	<= 12 (14)	7.0 (80P)	5.1 (80P)	4.6 (80P)	6.1 (80P)
450-600	<= 19	8.3 (80P)	5.6 (80P)	11 (80P)	9.0 (80P)
600-800	<= 25	10.0 (80P)	6.8 (80P)	7.0 (80P)	12 (80P)

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1: 4.4 to 5.5 km	2: 5.5 to 5.7 km	3: 5.7 to 5.9 km	4: 5.9 to 6.9 km
0-150	<= 3.2 (3.5)	3.2	3.2	3.4 (80P)	3.0 (80P)
150-300	<= 6 (6.9)	4.9 (80P)	6.0	2.6 (80P)	4.5 (80P)
300-450	<= 12 (14)	5.8 (80P)	10 (80P)	4.6 (80P)	6.1 (80P)
450-600	<= 19	7.0 (80P)	11 (80P)	11 (80P)	9.0 (80P)
600-800	<= 25	8.5 (80P)	5.3 (80P)	7.0 (80P)	12 (80P)

Averages from	Weighted Average
Percentiles from	Normal Distribution

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no.			
		1	2	3	4
		4.4 to 5.5 km	5.5 to 5.7 km	5.7 to 5.9 km	5.9 to 6.9 km
0-150	<=3.2 (3.5)	4.9 (80P)	10 (80P)	3.4 (80P)	3.0 (80P)
150-300	<=6 (6.9)	5.8 (80P)	11 (80P)	2.6 (80P)	4.5 (80P)
300-450	<=12 (14)	7.0 (80P)	5.1 (80P)	4.6 (80P)	8.1 (80P)
450-600	<=19	8.3 (80P)	5.6 (80P)	11 (80P)	9.0 (80P)
600-800	<=25	10.0 (80P)	6.8 (80P)	7.0 (80P)	12 (80P)

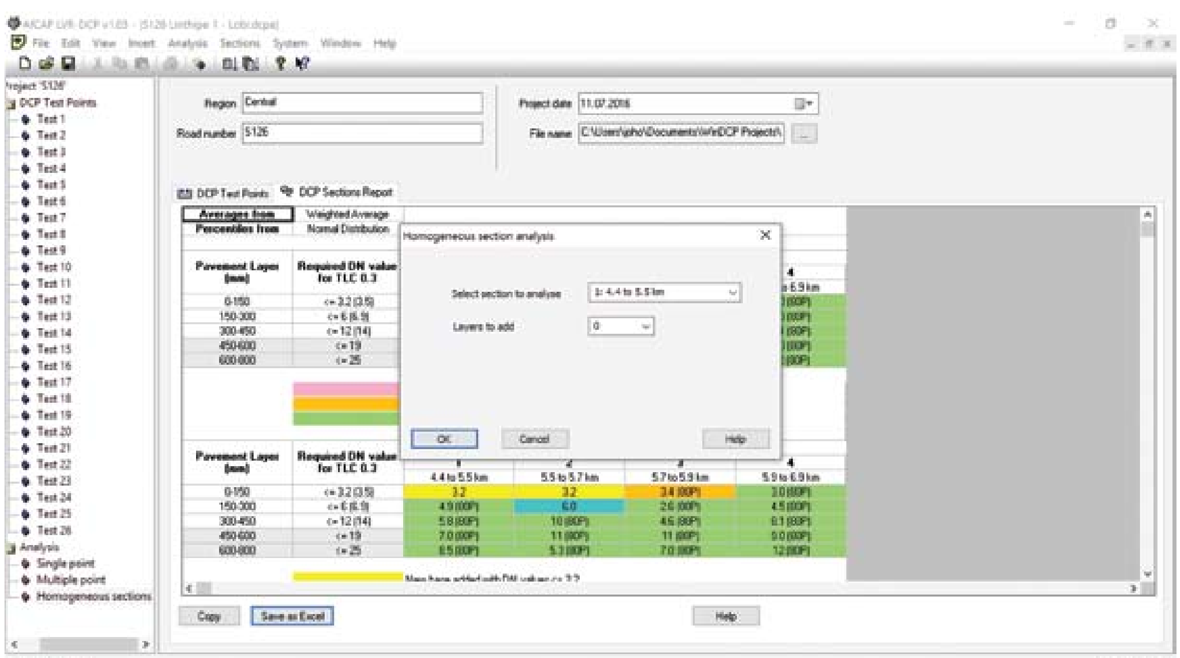
Inadequate (non-compliance) in situ layer
Adequate (margin al compliance) in situ layer(s) that need to be impro
Adequate (full compliance) in situ layer(s)

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no.			
		1	2	3	4
		4.4 to 5.5 km	5.5 to 5.7 km	5.7 to 5.9 km	5.9 to 6.9 km
0-150	<=3.2 (3.5)	3.2	3.2	3.4 (80P)	3.0 (80P)
150-300	<=6 (6.9)	4.9 (80P)	6.0	2.6 (80P)	4.5 (80P)
300-450	<=12 (14)	5.8 (80P)	10 (80P)	4.6 (80P)	8.1 (80P)
450-600	<=19	7.0 (80P)	11 (80P)	11 (80P)	9.0 (80P)
600-800	<=25	8.5 (80P)	5.3 (80P)	7.0 (80P)	12 (80P)

New base added with DN values <=3.2
New sub base added with DN values <=6
Inadequate (non-compliance) in situ layer
Adequate (margin al compliance) in situ layer(s) that need to be improve
Adequate (full compliance) in situ layer(s)

The Design Report shows four uniform sections identified within the 2.5km, each analysed separately for the upgrading requirements.

The top part of the Design Report shows the situation for each uniform section before upgrading where each layer has been assessed against the specifications to the left for the appropriate Traffic Load Class, in this case TLC 0.3 for traffic loading between 100,000 and 300,000 Equivalent Standard Axles over the design life of the road.

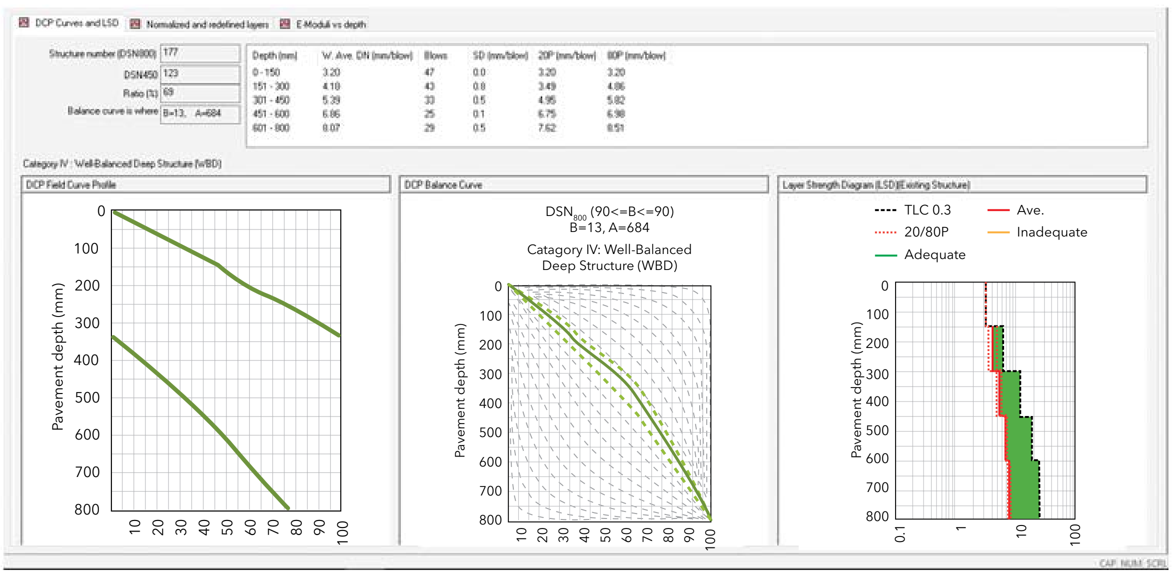


It is shown that Sections 1 and 2 needs a new base layer with a DN value ≤ 3.2 mm (from the DCP-DN catalogue). Section 2 also needs a new subbase with a DN ≤ 6 mm. In Section 3 it is assumed that the existing base layer can probably be improved by scarifying, reshaping and compaction to refusal, hence the amber colour. In Section 4 the existing base is already strong enough and needs only reshaping before surfacing.

The bottom part of the Design Report shows the situation after upgrading.

Each section can then be analysed separately using the "DCP Section Analysis per Section" feature as shown below.

Below is shown the analysis for Section 1 after upgrading by adding a new 150 mm base layer. Section 1 will then have a "Well balanced deep structure", which one should always try to achieve (but which is not always possible). It can also be seen that the DSN 800 value is above the minimum requirement of 100 blow for this design class.



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