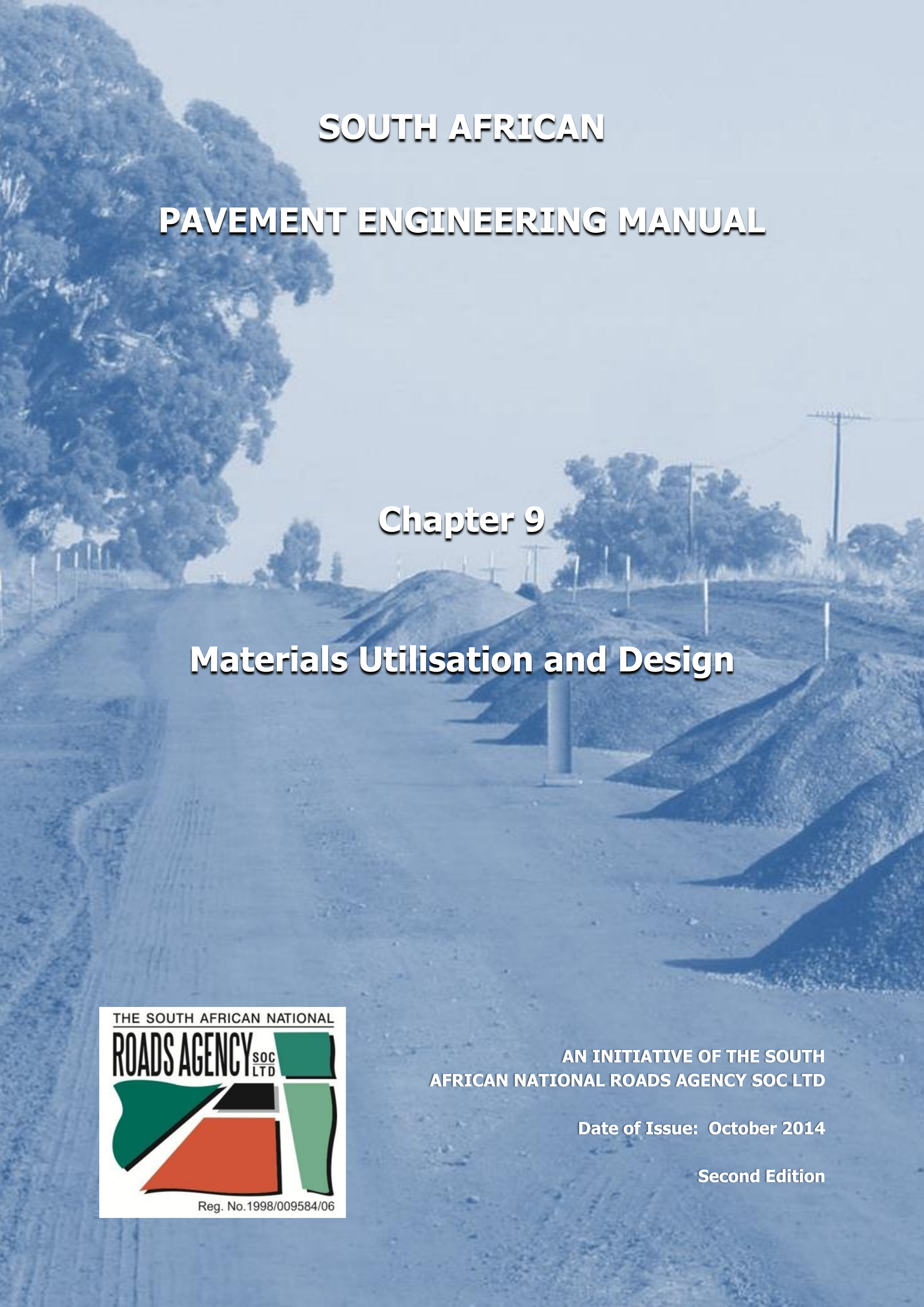

SOUTH AFRICAN

PAVEMENT ENGINEERING MANUAL

Chapter 9

Materials Utilisation and Design

THE SOUTH AFRICAN NATIONAL
ROADS AGENCY SOC LTD
Reg. No.1998/009584/06

AN INITIATIVE OF THE SOUTH AFRICAN NATIONAL ROADS AGENCY SOC LTD

Date of Issue: October 2014

Second Edition

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Chapter 9: Materials Utilisation and Design

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BACKGROUND

1. Introduction
2. Pavement Composition and Behaviour

TESTING AND LABORATORY

3. Materials Testing
4. Standards
5. Laboratory Management

INVESTIGATION

6. Road Prism and Pavement Investigations
7. Geotechnical Investigations and Design Considerations
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11. Documentation and Tendering

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QUALITY MANAGEMENT

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SCOPE

The South African Pavement Engineering Manual (SAPEM) is a reference manual for all aspects of pavement engineering. SAPEM is a best practice guide. There are many relevant manuals and guidelines available for pavement engineering, which SAPEM does not replace. Rather, SAPEM provides details on these references, and where necessary, provides guidelines on their appropriate use. Where a topic is adequately covered in another guideline, the reference is provided. SAPEM strives to provide explanations of the basic concepts and terminology used in pavement engineering, and provides background information to the concepts and theories commonly used. SAPEM is appropriate for use at National, Provincial and Municipal level, as well as in the Metros. SAPEM is a valuable education and training tool, and is recommended reading for all entry level engineers, technologists and technicians involved in the pavement engineering industry. SAPEM is also useful for practising engineers who would like to access the latest appropriate reference guideline.

SAPEM consists of 14 chapters covering all aspects of pavement engineering. A brief description of each chapter is given below to provide the context for this chapter, Chapter 9.

Chapter 1: **Introduction** discusses the application of this SAPEM manual, and the institutional responsibilities, statutory requirements, basic principles of roads, the road design life cycle, and planning and time scheduling for pavement engineering projects. A glossary of terms and abbreviations used in all the SAPEM chapters is included in Appendix A. A list of the major references and guidelines for pavement engineering is given in Appendix B.

Chapter 2: Pavement Composition and Behaviour includes typical pavement structures, material characteristics and pavement types, including both flexible and rigid pavements, and surfacings. Typical materials and pavement behaviour are explained. The development of pavement distress, and the functional performance of pavements are discussed. As an introduction, and background for reference with other chapters, the basic principles of mechanics of materials and material science are outlined.

Chapter 3: Materials Testing presents the tests used for all material types used in pavement structures. The tests are briefly described, and reference is made to the test number and where to obtain the full test method. Where possible and applicable, interesting observations or experiences with the tests are mentioned. Chapters 3 and 4 are complementary.

Chapter 4: Standards follows the same format as Chapter 3, but discusses the standards used for the various tests. This includes applicable limits (minimum and maximum values) for test results. Material classification systems are given, as are guidelines on mix and materials composition.

Chapter 5: Laboratory Management covers laboratory quality management, testing personnel, test methods, and the testing environment and equipment. Quality assurance issues, and health, safety and the environment are also discussed.

Chapter 6: Road Prism and Pavement Investigation discusses all aspects of the road prism and pavement investigations, including legal and environmental requirements, materials testing, and reporting on the investigations. The road pavement investigations include discussions on the investigation stages, and field testing and sampling (both intrusively and non-intrusively), and the interpretation of the pavement investigations. Chapters 6 and 7 are complementary.

Chapter 7: Geotechnical Investigations and Design Considerations covers the investigations into fills, cuts, structures and tunnels, and includes discussion on geophysical methods, drilling and probing, and stability assessments. Guidelines for the reporting of the investigations are provided.

Chapter 8: Material Sources provides information for sourcing materials from project quarries and borrow pits, commercial materials sources and alternative sources. The legal and environmental requirements for sourcing materials are given. Alternative sources of potential pavement materials are discussed, including recycled pavement materials, construction and demolition waste, slag, fly ash and mine waste.

Chapter 9: Materials Utilisation and Design discusses materials in the roadbed, earthworks (including cuts and fills) and all the pavement layers, including soils and gravels, crushed stones, cementitious materials, primes, stone precoating fluids and tack coats, bituminous binders, bitumen stabilized materials, asphalt, spray seals and micro surfacings, concrete, proprietary and certified products and block paving. The engineering properties of the materials are discussed. Guidelines for all aspects of the application and use of these materials and their mix designs are given. The materials classification system for design is also covered.

Chapter 10: Pavement Design presents the philosophy of pavement design, methods of estimating design traffic and the pavement design process. Methods of structural capacity estimation for flexible, rigid and concrete block pavements are discussed.

Chapter 11: Documentation and Tendering covers the different forms of contracts typical for road pavement projects; the design, contract and tender documentation; the tender process; and the contract documentation from the tender award to the close-out of the Works.

Chapter 12: Construction Equipment and Method Guidelines presents the nature and requirements of construction equipment and different methods of construction. The construction of trial sections is also discussed. Chapters 12 and 13 are complementary, with Chapter 12 covering the proactive components of road construction, i.e., the method of construction. Chapter 13 covers the reactive components, i.e., checking the construction is done correctly.

Chapter 13: Quality Management includes acceptance control processes, and quality plans. All the pavement layers and the road prism are discussed. The documentation involved in quality management is also discussed, and where applicable, provided.

Chapter 14: Post-Construction incorporates the monitoring of pavements during the service life, the causes and mechanisms of distress, and the concepts of maintenance, rehabilitation and reconstruction.

FEEDBACK

SAPEM is a "living document". The first edition was made available in electronic format in January 2013, and a second edition in October 2014. Feedback from all interested parties in industry is appreciated, as this will keep SAPEM relevant.

To provide feedback on SAPEM, please email sapem@nra.co.za.

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1. INTRODUCTION

This chapter covers the design of the roadbed, the design of the earthworks and the utilisation of materials. The materials can be obtained from within the road prism, from sources outside the road prism, and manufactured products for the construction of the earthworks and pavement layers of roads. The materials include:

- Natural soils and gravels
- Crushed stone materials and screened rock (aggregates)
- Cemented materials
- Bitumen stabilized materials (BSMs)
- Primes, tack coats and precoating fluids
- Hot mix asphalt
- Bituminous seals and micro-surfacings
- Concrete
- Various proprietary products
- Concrete block pavers

The aim of this chapter is not to provide a comprehensive design manual, as no two projects and environments are similar in all respects. Rather, the chapter provides the user with guidelines as to the design and rational utilisation of materials. Where current and comprehensive guidelines and manuals are available, the manual does not attempt to reproduce these. Recommendations are made as to the appropriate usage, design principles and considerations of the various materials, and reference is made to these documents regarding the actual design.

Aim of this Chapter

The aim of this chapter is not to provide a comprehensive design manual, as no two projects and environments are similar in all respects. Rather, the chapter provides the user with guidelines as to the design and rational utilisation of materials and points to other recommended guidelines.

Standard Specifications

Note that when SAPEM written and updated, the 1998 version of the COLTO Standard Specifications was being used. However, these specifications are currently being reviewed. A revised version of the Standard Specifications is likely to be published in 2015 and is likely to be issued either by SANS or COTO.

In this chapter, reference is only made to the Standard Specifications, which currently refers to the 1998 COLTO version.

2. ROADBED

The roadbed describes the in situ materials at the level that either a cut is excavated to accommodate the road pavement, or the natural in situ material on which fill or imported pavement layers are constructed.

A stable and uniform roadbed needs to be provided for the successful construction and in service performance of a road embankment and/or pavement layers. In general, roadbed conditions differ along the length of a road as influenced by changes in topography, vertical alignment, hydrology, geology, land use and many other features as described in Chapter 6: 6.

For road building or road widening purposes, the approach generally followed is to identify areas of relative uniformity along the route. For each uniform section, the measures required to provide the required platform for the construction of the embankment and/or pavement layers are designed. These treatments are generally referred to as roadbed treatment types and are generally described in detail and shown on the design plans.

Road pavements comprise the combination of individually constructed layers including the surfacing layer, base layers and subbase layers designed to carry the structural loading and to provide the desired performance characteristics. These layers are almost always constructed from imported, processed materials.

Materials Depth
This is defined as that depth below the finished road level wherein the physical properties of the materials have a significant effect on the pavement behaviour. The materials depth is also the depth to which the moisture regime needs to be controlled.

On the more highly trafficked roads, the necessary support to these pavement layers is provided by specifying "selected" subgrade layers. Additional requirements may also be specified for such fill layers that lie within the "materials depth". Figure 1 and Figure 2 illustrate this concept in both cut and fill situations.

Typical material depths as a function of the road category are given in Table 1. Road categories are defined in Chapter 10: 3.1. They are a function of, inter alia, the traffic loading and constructed standard which determines the minimum pavement structure needed to meet the various performance requirements.

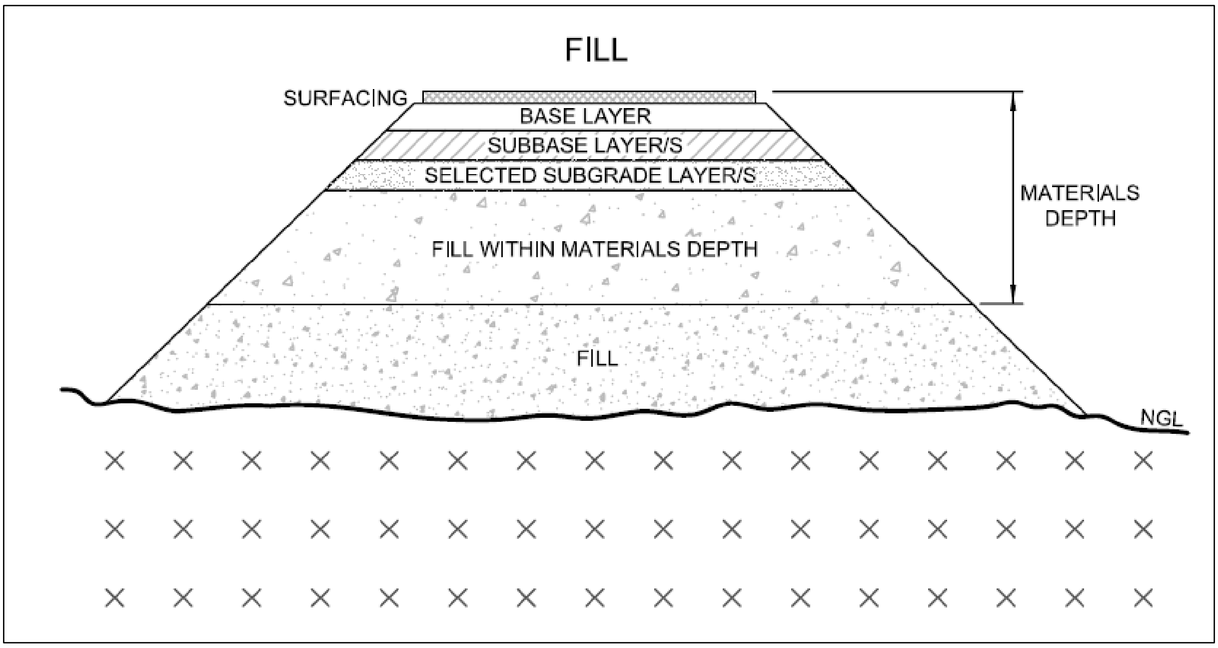


Figure 1. Materials Depth in a Fill

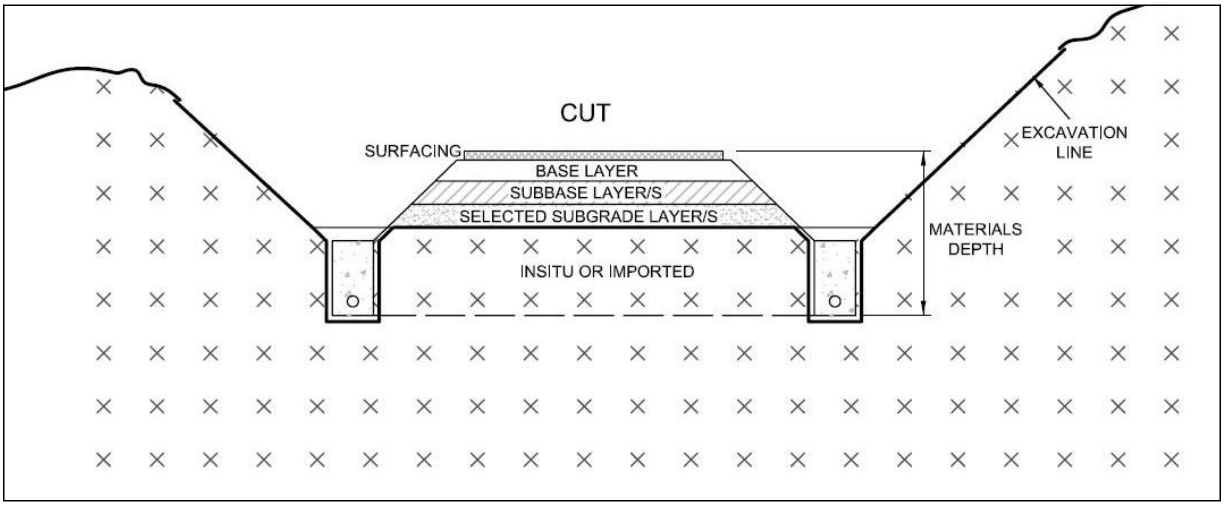


Figure 2. Materials Depth in a Cut

Table 1. Typical Materials Depths

Road Category	Materials Depth (mm)
A	1 000 – 1 200
B	800 – 1 000
C	800
D	700

Where the roadbed is within the materials depth and the existing materials have properties or conditions that do not meet the requirements for that position in the pavement structure, the materials have to be removed or treated. The materials may not meet the quality requirements and/or the density requirements. The materials can be removed and replaced with appropriate materials, or treated to improve their properties either in situ or in a temporary stockpile. Figure 2 illustrates this situation.

Where materials in the roadbed meet the quality requirements for layers in that position of the pavement, reworking may be required to ensure that uniformity, thickness and compaction requirements are met. Major cost savings can be realised by utilising in situ materials.

2.1 Assessment of Road Prism Investigation Data and Test Results

In carrying out the road prism investigations described in Chapter 6: 5, the following situations may be identified:

- **Conventional treatments:** Roadbed conditions are such that the treatments required before construction of the fill or pavement layers are conventional treatments as provided for in, for example, the Standard Specifications. The appropriate treatments for a specific section of the roadbed, however, have to take the reigning conditions and influences into consideration and must accordingly be designed to suit the situation.
- **Geotechnical issues**: Problem areas in the roadbed require the services of a geotechnical engineer, to investigate the extent of the problem and to design adequate solutions to provide the necessary stable foundations for the construction of the fills or pavement layers. The geotechnical engineer may be assisted by other specialists, such as, engineering geologists, hydrologists, mining engineers or other specialists, as necessary. Problem roadbed conditions include those listed in Table 2.

Roadbed Problems

The roadbed problems listed in Table 2 are discussed in detail in:

- Chapter 6: **Road Prism and Pavement Investigations**, Sections 6

Table 2. Possible Roadbed Problem Areas

Problem	Possible Solutions
Undermined Ground	• Realignment to avoid • Additional underground support • Bridging affected area
Dolomitic foundations	• Realignment to avoid • Bridging affected area • Infilling of cavities • Collapsing cavities
Heaving or expansive clays	• Realignment to avoid • Removal of clays • Removal to constant moisture, sealing off
Dispersive soils	• Encapsulate with non-dispersive soils • Ensure filtered drainage
Collapsible soils	• Collapsing by heavy rolling, with or without hydration • Collapsing by dynamic compaction • Vibroflotation (inserting a vibrating poker into the soil)
Made ground and landfills	• Realignment to avoid • Dynamic compaction, assisted drainage • Preloading with surcharge • Bridging

Whatever the case, the extent of the special measures to treat these problem roadbeds must be defined and adequately described, thereby enabling the project materials engineer to add these to those roadbed treatments described in the following section (Section 2.2).

2.2 Design of Typical Roadbed Treatments

Where the in situ roadbed materials lie within the selected subgrade or fill layers, i.e., within the materials depth, and meet the quality requirements, the materials can be used. The advantage is that materials need not be imported from another source. The needs of uniformity, constructability and density, however, still need to be met.

Where the in situ materials do not meet the requirements for use "in place", they have to be removed and replaced with materials meeting the specific requirements. These may be the same materials that have been dried out or blended with other materials to achieve a more suitable product, materials brought in under a cut and fill operation, or from a borrow pit or other source.

The assessment of the information gained from the road prism investigations, i.e., test pits, auger holes, borehole profiles or open face examinations, and the results of tests on recovered samples and borehole cores, should provide the materials engineer with sufficient information to make a reliable and accurate assessment of the quality of the materials and reigning subgrade conditions. This enables the design of measures required to provide the required support for the following fill or pavement layers.

In general, two typical conditions are encountered in the roadbed after clearing and grubbing, or excavating to the required levels. These are:

- Roadbed falls **within the materials depth**, as in low fill and/or shallow cuts. This situation is discussed in Section 2.2.1.
- Roadbed falls **outside the materials depth**, as in fills. This situation is discussed in Section 2.2.2.

2.2.1 Roadbed Falls within the Materials Depth

Figure 3 illustrates the situation where the roadbed falls within the materials depth. Table 3 gives scenarios for soft materials in the roadbed within the materials depth.

Table 3. Roadbed Treatment Types for a Roadbed within the Materials Depth

Situation	Roadbed Treatment Type
The quality of materials at the selected layers position meets the requirements for use.	
	• Excavate any surplus materials above the level of the selected subgrade layers to stockpile for use in selected layers elsewhere. • Cut the materials meeting the quality requirements for use in the selected layers to the windrow. • Rip or scarify in situ material (i.e., top of subgrade level) 150 mm deep. • Compact to the specified density (generally 90% of MDD[1]). If the required density cannot be attained by normal rolling, the engineer may direct that 150 mm layers be removed to the windrow until the density requirements are met. • Blade sufficient material for a 150 mm (or specified layer thickness) selected subgrade layer into position, wet the materials to the optimum moisture content for the compaction plant. Mix as required, shape and compact to the required density (generally 93% of MDD). • Similarly, construct the upper selected subgrade layer if the appropriate quality requirements are met and if sufficient material is available and windrowed.
Quality of materials in position of the selected layers is unsatisfactory for use.	
The quality of the materials in the position of the **fill layers within the materials depth is satisfactory for use** in these layers	• Excavate (cut) the materials occurring in the position of the selected or other pavement layers to fill if the requirements for such are met and the materials are required (see Chapter 6: 6), or cut to spoil. • Rip or scarify the in situ material 150 mm deep. • Compact the material to the specified density (generally 90% of MDD). If the required density cannot be attained by normal rolling, the engineer may direct that such multiples of 150 mm layers be removed to windrow until these requirements are met. • Follow thereon with selected subgrade layer works as required utilising imported materials.
The quality of the materials in position of **fill layers within materials depth is unsatisfactory for use** in these layers.	• Excavate the in situ materials within the materials depth to spoil or as otherwise directed by the engineer. • Apply a three roller pass treatment to the roadbed using a heavy pneumatic, impact or vibrating roller as appropriate. • If stable, compact the roadbed to the specified density (generally 90% of MDD or 100 % of MDD for cohesionless sands). • Import G10 quality fill materials and compact in 150 mm layers to the specified density (generally 90% of MDD). • Follow thereon with selected subgrade and other layer works as required utilising imported materials.
Materials within the materials depth or roadbed are **too wet to provide a stable platform** for the construction of the pavement layers	**Option 1: Install pioneer layer** • Excavate the materials to such depth as required to enable the construction of a pioneer layer of thickness just sufficient to provide a stable working platform for constructing the further fill layers to the required density. • Follow thereon with geofabric filter or filter materials and with conventional fill construction. **Option 2: Install subsoil drainage system** • Where persistent wet conditions, such as artesian conditions, are encountered but the materials are otherwise satisfactory, a subsoil drainage system may be required to ensure that the pavement layers do not become saturated by groundwater seepage from the edges, or from below. The design of such systems is generally site specific, but guidelines are given in TRH9, TRH15 and SANRAL's Drainage Manual. • In many such cases, these measures are supplemented by the construction of drainage blankets, comprising permeable gravels or sand filter blankets (as provided for in the Standard Specifications. These layers are generally protected from contamination by providing geotextile filters between the blanket and surrounding materials. • As a stand-alone measure, or as a supplementary measure, the superseding layer may be stabilized as a barrier to the migration of groundwater, while simultaneously contributing to uniformity and load distribution.

Note
1. MDD is the maximum dry density. Historically this density was specified as a percentage of the Mod. AASHTO density.

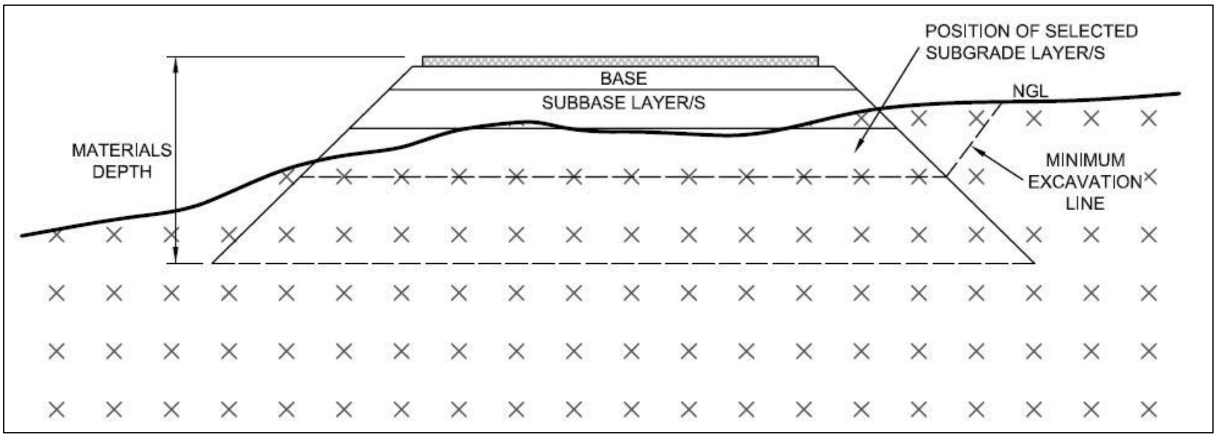


Figure 3. Roadbed Falls Within the Materials Depth in a Shallow Cut or Fill

Where intermediate or hard materials are encountered in the roadbed in shallow fills and in cuttings, factors such as uniformity of support and drainage necessitate special measures to ensure that:

- **Support** to the pavement structure is uniform.
- **Differential settlement**/consolidation is minimised.
- All **water** entering the pavement layers or the support, whether from below, the edges or through the road pavement, is quickly and efficiently drained.

Roadbed Treatments

When one of the measures listed in Table 3 and Table 4 are incorporated in the roadbed design, it must be fully described, and labelled or numbered accordingly so it can be shown on the design plans.

The Standard Specification provides for the in place treatment of rocky roadbeds by ripping and/or blasting to depths increasing from the centre of the roadbed to the edges, as illustrated in Figure 4 and Figure 5. The depth of ripping shall not be less than 300 mm at the centre and not less than 500 mm at the edges of the roadbed. Similarly, for drilling and blasting, the depth shall not be less than 700 mm at the centre of the roadbed and not less than 1 000 mm at the edges. On super elevated sections, a uniform crossfall is required and minimum depths of 400 mm for ripping and 850 mm for blasting are required. This is followed by ripping and breaking down the material to two thirds the layer thickness, and then compacting with 8 passes with an approved combination of various rollers.

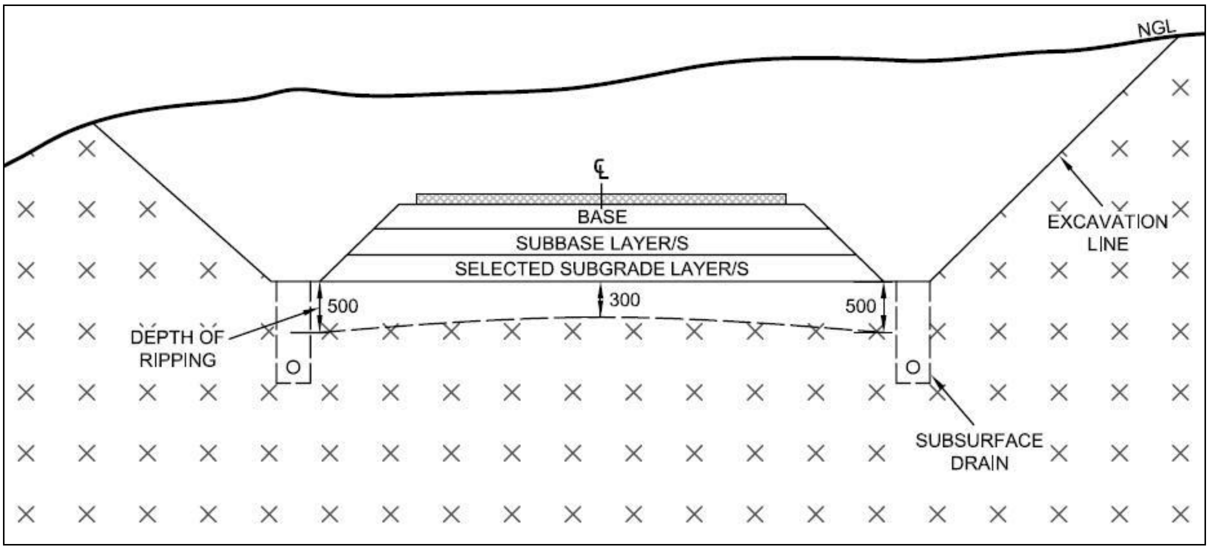


Figure 4. Rippable Roadbed

Where required, the appropriate measures should be selected, adequately described and labelled or numbered and added to the list of subgrade treatments required for the project.

Care should be exercised in the application of these measures as some rock types quickly degrade on exposure to the elements. Underblasting and ripping may produce unstable roadbeds in these materials (see Section 3). Free drainage should always be maintained and excavation may need to be preceded by subsurface drain installation or dewatering.

Rocks that Degrade on Exposure
Care should be exercised in the application of the road treatment measures as some rock types quickly degrade on exposure to the elements. Underblasting and ripping may produce unstable roadbeds in these materials (see Section 3). Free drainage should always be maintained, and excavation may need to be preceded by subsurface drain installation or dewatering.

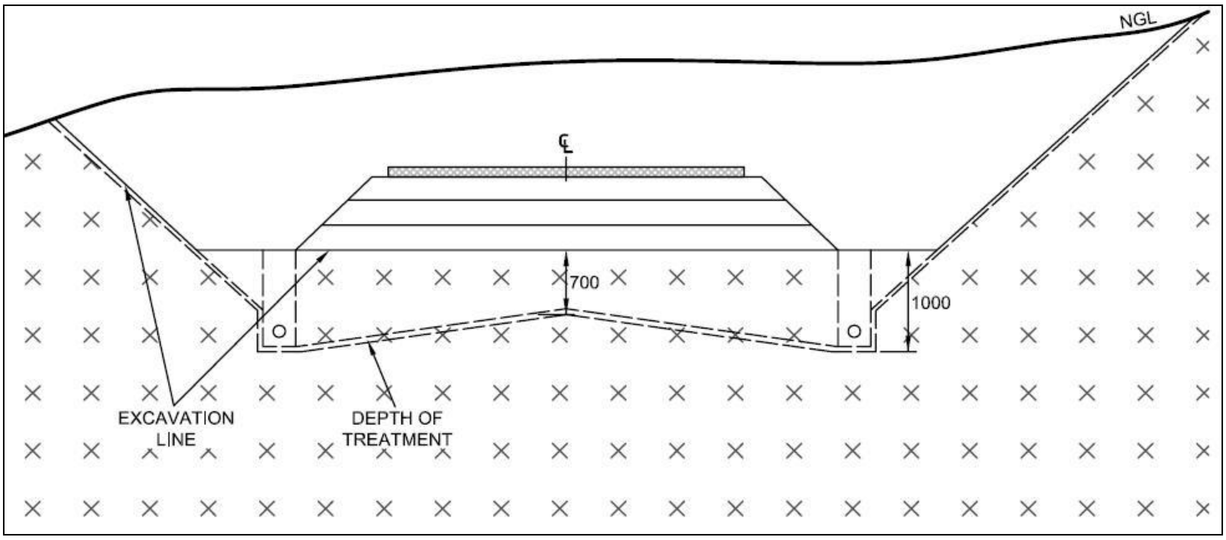


Figure 5. Hard Rock Roadbed, Treatment in Place by Drilling or Blasting

2.2.2 Roadbed Falls Outside the Materials Depth

When the roadbed falls outside the materials depth, and soft materials are encountered following clearing and grubbing, the scenarios in Table 4 may be present. Note that in this case it is essentially a fill situation. Appropriate treatment types are given for each scenario.

Where hard materials occur in the roadbed, the following situations may occur:

- **Abrupt hard to soft transitions**. This may result in differential settlement in severe cases. The likely remedy is benching to feather out this transition. This is discussed in Section 3.
- **Seepage along a hard to soft interface**. This may result in softening of the roadbed in places, resulting in non-uniform support and even instability. The designer should be wary of such conditions and ensure the prescription of whatever localised measures as may be required, such as interceptor drains, to address the problem.
- **Sliding of constructed works on sloping bedrock**. Possible solutions to this phenomenon are anchoring the toe to the slope (various geotechnical options exist) or blasting horizontal benches. A serrated surface promotes interlock, but drainage need attention. This may not be possible in some inclined strata, and the assistance of a specialist geotechnical engineer should be sought when confronted with this situation.

Table 4. Roadbed Treatment Types for a Roadbed outside the Materials Depth

Scenario	Roadbed Treatment Type
Generally stable and dry conditions in roadbed	• Rip/Scarify the in situ material to 150 mm deep • Compact the material to the specified density (generally 90% of MDD). If the required density cannot be attained by normal rolling, it may be necessary to remove additional layers, use rockfill or a sand blanket, or build additional layers • Follow thereon with selected subgrade layer works as required, utilising imported materials. This scenario is illustrated in Figure 6.
Saturated conditions in roadbed	**Option 1: Pioneer Layer** Construct a pioneer layer of thickness just sufficient to provide a stable working platform for constructing the further fill layers to the required density. **Option 2: Drainage Blanket** Construct a drainage blanket comprising permeable gravels, stone layers or a sand filter blanket (as provided for in the Standard Specifications). These layers are generally protected from contamination by providing geotextile filters between the blanket and surrounding materials. **Option 3: Subsoil Drainage System** Where marshy or artesian conditions are encountered, a subsoil drainage system may need to be designed and installed to drain the foundation. The design of such systems is generally site specific, but guidelines are given in TRH9, TRH15 and in SANRAL's Drainage Manual.
Poor, unstable materials in roadbed	**Option 1: Remove and Replace** Remove the unstable materials and the replacement thereof with suitable materials. The practicality and economic viability of this solution needs to be carefully assessed. **Option 2: Bridge the Area** Bridge the area with rockfill and possibly use geogrids for load spreading, and possibly use geotextiles to limit contamination, as shown in Figure 7. **Option 3: Specialist Geotechnical Investigation** Appoint a specialist geotechnical engineer to investigate the situation and to design appropriate measures to treat the roadbed prior to the construction of the fill.

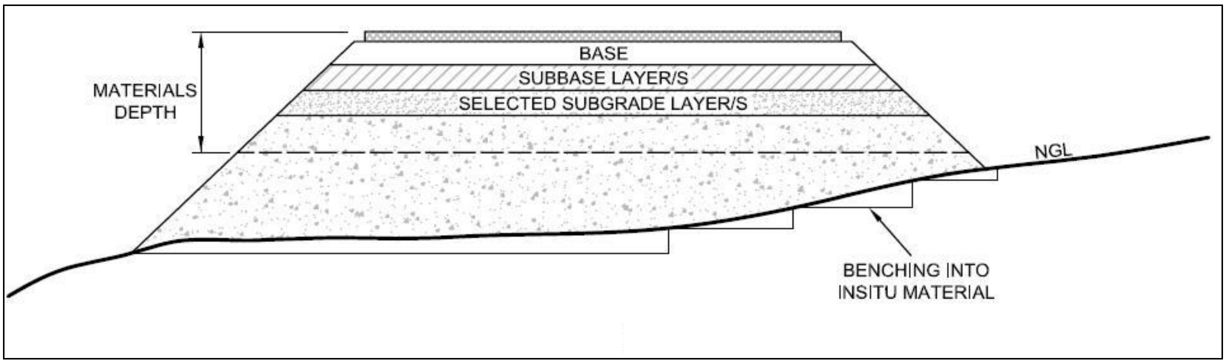


Figure 6. Roadbed Falls Outside Materials Depth

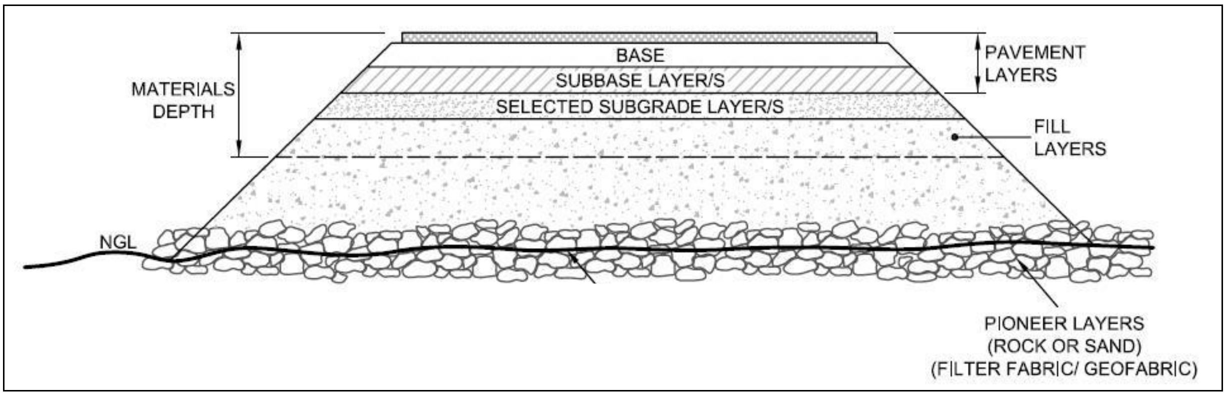


Figure 7. Roadbed Falls Outside Materials Depth on Unstable Roadbed

2.3 Compiling Roadbed Treatments on a Project

The varying scenarios described in the previous sections typify some of the roadbed treatments that may be prescribed on a road project. One of the major objectives of the road prism investigations, described in Chapter 6: 6 is to provide the information needed to design and typify specific treatments and prescribe the position and extent thereof on a project.

In deciding on a particular treatment, the extent must also be considered. Particular treatments, by their very nature, may be minor in extent. Where these are essential, it is imperative that these be brought to the contractor's attention, especially where the construction may be a pre-requisite to constructing other works, particularly the bulk earthworks. In most cases, however, the designer considers practicality and economics of scale, and minimises the number of different roadbed treatments, while attempting to keep the lengths of such treatments such that the plant can be effectively utilised.

For SANRAL projects, the characterisation of the roadbed is a major investigation and design item in view of the role it plays in the successful construction and long term performance, especially of major cuts and fills. As such, it is a prominent investigatory and design aspect and report back item in the various design reports. See Chapter 6: 9 for more on reporting.

In these reports, and in the detail as available and appropriate for each stage, the various treatments are motivated, described and summarised (generally in tabular form) as the proposed roadbed design for the project. The extent of each treatment is also shown in plan on the soils survey sheets (generally A0 plans with the road shown in section and in plan) as well as at the bottom of the plan coincidental with the stake values. A typical soil survey plan, indicating the prescribed roadbed treatments for the sections of road shown is included in Figure 8.

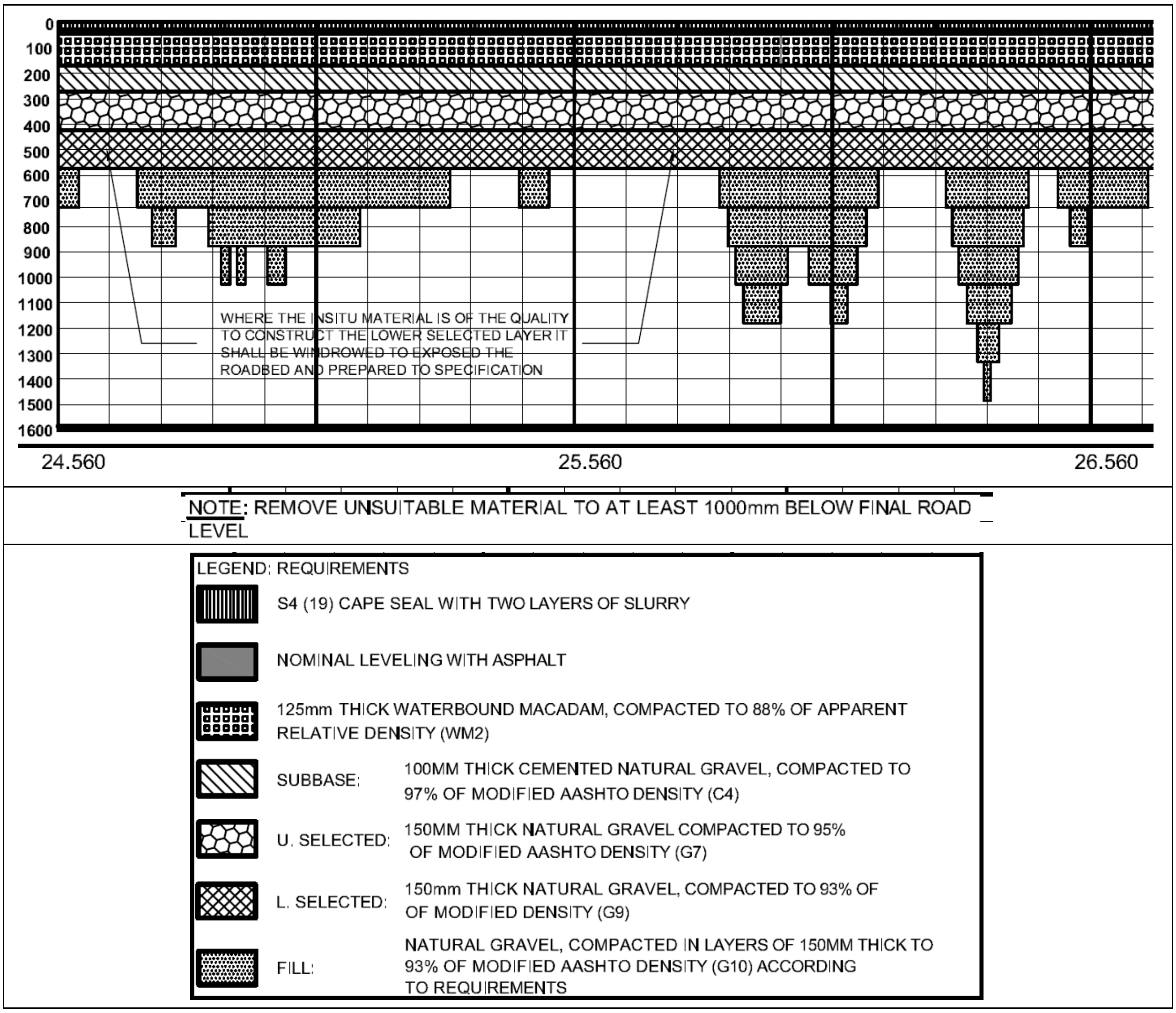


Figure 8. Pavement Design Showing Roadbed Treatments

3. EARTHWORKS

Earthworks in the context of this chapter includes all works in connection with the construction of cuts and fills, including the removal of unsuitable materials to spoil and importing suitable materials from approved borrow pits and other approved sources. This excludes pavement layers, which are covered in Sections 4, 5 and 6. The treatment of the roadbed prior to the construction of fills or road pavements and in cuts is covered earlier in Section 2.

In designing the earthworks, whether for a new facility or for the widening of an existing road, the designer strives to minimise the movement and handling of materials, especially the earthworks in view of the generally large volumes and associated costs. The ideal situation is an alignment that provides sufficient materials from cuts to provide about 50% of the adjacent fill's requirements, in an equal haul situation. In this case, there is no double handling of material (e.g., cut to stockpile and then cart to fill) and a normal cut to fill operation can follow. The earthworks are then balanced. An even more ideal situation is for most of the materials required for use in the pavement layers to come from excavated materials, albeit in an unprocessed state or as constituents in asphalt and concrete mixes. This, however, is seldom the case, as evident in the discussion in the following sections.

Earthworks

In this Chapter, earthworks include all works in connection with the construction of cuts and fills. This includes the removal of unsuitable materials to spoil and importing of suitable materials from approved borrow pits and other sources. Earthworks exclude pavement layers.

3.1 Cuts

Cuttings generally provide the most economical sources of acquired construction materials. An example of a cut is shown in Figure 9. There are various aspects to consider when using materials from cut, which are discussed below.

Figure 9. Cuts

3.1.1 Assessment of Road Prism Investigation Data and Test Results

The road prism investigation (Chapter 6: 5) provides the materials engineer with information on the expected position of the soils, gravels, boulders, weathered rock and hard rock horizons in each of the cuttings. These are obtained from visual observations, intrusive and physical tests. This enables plotting of longitudinal and cross sections, determining quantities of various materials types and enables the likely excavation methods, and the quantification thereof, to be determined.

An analysis of the test data enables an assessment of the suitability of the materials for use in the planned earthworks, the pavement layers and possibly for other uses such as aggregate for surfacing seals, concrete or asphalt mixes. The meeting of fill requirements is a primary objective because fills generally require large volumes of materials, with related acquisition and construction costs.

The first step is thus to quantify the available materials that are suitable for use in fill construction, and determine whether the required quantities can be met. Not all cut materials may be suitable for this purpose due to quality, maximum size and moisture content requirements. It is also necessary to identify and quantify unsuitable (too wet) and spoil (substandard quality) materials. Further important considerations for using materials in adjacent or nearby fills are:

- **Access:** There may not be immediate access to an adjacent fill due to topographical features, which forces construction from one end, or due to a structure needing to be constructed before access is possible.
- **Non suitability:** The adjacent fills may require a rock fill as a bridging layer or a rock toe, but the rock in the cutting may lie beneath substantial overburden, which is suitable for use as fill. In this case, cutting to stockpile or spoiling the fill and importing fill from other sources are options to be considered. Sourcing the rock from elsewhere is also an option.
- **Improved materials with depth:** Material quality tends to improve with depth. The upper materials in the cutting may be suitable for use in the upper layers of a high fill or in the centre core of the upper reaches of the fill, but not before the lower parts of high fills have been constructed with higher quality material as required by the design.
- **Haul distance:** The haul distance from the nearest cutting with suitable materials may be further and therefore more expensive to source than a nearby borrow pit.
- **Accessing an adjacent fill**: In some cases, a cutting may need to be excavated to gain access to an adjacent fill. This may necessitate temporary stockpiling, cross haulage or opening a borrowpit.
- **Fill over extended distances:** Where the road is in fill over extended distances, as for example in the crossing of a flood plain, the shortfall of fill materials available from cuttings may necessitate the use of borrowpits.

The materials engineer and design team have the additional option of minimizing or maximizing the quantities of materials from any particular cutting by:

- Flattening or steepening the **side slopes of cuttings**
- **Widening or narrowing** the cuttings
- **Daylighting hillocks** that are crossed by the road
- Adjusting the **vertical and horizontal alignment** of the road, for example, making changes to the vertical points of intersection (PI's).

It is important that when the materials in a cut are not of the required quality, exceed the maximum size requirements, or are too wet for use as construction materials, the engineer should strive to minimize the volumes of such materials by exercising the options stated. Care must, of course, be taken not be compromise the stability of the slopes. The mass haul diagram is a useful tool for this, see Section 3.2.5.

The preliminary design of the road as captured, in the Basic Planning Report and design drawings provides the materials engineer with a good estimation of other materials requirements, for example, pavement layer requirements, concrete requirements, and stone for gabions and pitching.

Other uses of cut materials include the following:

- **Sands** for use in drainage layers, concrete and asphalt
- **Gravel layers** for unsurfaced roads and haul roads
- **Selected subgrade or subbase** materials
- **Rock** for rock fill or rock toes
- **Aggregate**, after the crushing and screening of sound excavated rock for:
  - Crushed subbase and base materials
  - Concrete and asphalt
  - Stone pitching
  - Gabion rock

Further processing such, as mechanical stabilization and breaking down by rolling, widen the possible utilisation of cut materials. Many factors influence decisions to maximise cuttings to provide for such uses, but economics is, in most cases, the deciding factor. Acquisition and

Cut Materials Not of Required Quality

It is important that when the materials in a cut are not of the required quality, exceed the maximum size requirements or are too wet for use as construction materials, the Engineer should strive to minimize the volumes of such materials by exercising the options stated above. Care must of course be taken not to compromise the stability of the slopes. The mass haul diagram is a useful tool for this, see Section 3.2.5.

haulage are the most significant contributors. Other factors include the relative quality and durability of the materials, as well as the workability.

3.1.2 Excavation of Cuts

Different types of material such as soils, rocks and boulders require different methods of excavation. These may range from scrapers, excavators, loaders and dozers to rock breakers and blasting in hard rock.

Three of the four classes of excavation for materials to be excavated in a cutting in the Standard Specifications are defined by the type and power of given construction plant needed to efficiently excavate the material. These are soft excavation, intermediate excavation and hard excavation. The fourth class is that of boulder excavation where two sub classes are defined:

- **Class A:** The volume of material between 0.03 m^3 and 20 m^3 is **more** than 40% of the total volume of the materials to be excavated.
- **Class B:** The volume of material between 0.03 m^3 and 20 m^3 is **less** than 40% of the total volume of the materials to be excavated.

The following factors must be taken into account when deciding the most efficient means of excavation:

- **Material type**, i.e., soil, soft rock, hard rock or boulders
- **Physical condition**, i.e., dry or wet, soft, cemented, weathered to hard rock which may be massive, foliated, highly jointed or fractured
- **Engineering properties**, i.e., strength and durability

Where blasting is unavoidable, the blasting type needs to be decided, either pre-split or normal blasting. With pre–split techniques, overbreak is limited and thus quantities are more efficiently controlled. The aesthetic aspect of pre-split blasting should be considered. With normal blasting, over break is inevitable and quantities are less efficiently managed. The finish is, however, rough and on-going maintenance may be needed to remove debris emanating from rock falls caused by joint dilation during blasting, root wedging or water erosion. Widening of the cutting(s) may also be required to provide a safe drop zone for any rocks that may dislodge in time.

The size of the ripped or blasted fragments must also be considered. The fragments may require further processing to meet the specific requirements of the material. See Section 3.2.1.

Cut widening in hard rock may require pre-splitting to reduce over break. In some cases, greater use of mechanical or chemical means of breaking down hard rock may be necessary due to the proximity of urban developments.

3.1.3 Cut Slopes

The design of cut slopes is dependent on various factors:

- **Environment:** stability of area and upslope conditions, topography, side cut or box cut, natural slopes, surface and subsurface moisture conditions and movements, drainage patterns
- **Material type:** soil, colluvium or boulder material, soft or hard rock
- **Material characteristics:** wet or dry, soft or stiff, degree of cementation, degree of weathering, weathering characteristics, laminated, or jointed
- **Engineering properties of the exposed materials:** shear strength of soils and gravels, and dip of rock strata, as required to model behaviour and carry out stability analyses
- **Maintenance considerations:** erosion or degradation, and potential for re-vegetation

The designer is referred to Chapters 6 and 7 as well as TRH18 for the investigation and design of cut slopes. These references address the issues listed.

3.2 Fills

The investigation of the road prism, and the fills and cuts constructed to carry the road pavement, as well as the investigation by specialist geotechnical engineers of the problem areas in the roadbed are covered in Chapter 6: 6. The normal roadbed treatments, as identified in the road prism investigation, are covered in Section 2 of this chapter. In the reports and contract documentation, the normal treatments should be supplemented by those designed by the specialist geotechnical engineer. The areas of specific treatments may include deep removal by special machinery, ground improvement by jet grouting, compaction grouting, draining, vibroflotation, impact rolling, dynamic compaction, infilling of cavities, installing underground support, collapsing underground cavities and caverns, bridging areas, piling and bridging combinations and many others, as selected by the geotechnical engineer.

This section addresses the materials design and utilisation for constructing or widening fills. The designer is referred to TRH9 and TRH10 for the design and the investigation of road embankments. An example of a high fill is shown in Figure 10.

Design and Investigation of Fills

The TRH series has 2 good references for fills:

- **TRH9**: Construction of Road Embankments
- **TRH10**: The Design of Road Embankments

Figure 10. Fill

3.2.1 Materials Requirements

Specifications normally require, with the exception of rock fill, that the material should not contain rock fragments with maximum dimensions exceeding 500 mm. The specification also requires further breaking down on the road. The layer thickness is generally determined by the size to which the material can be broken down. A rule of thumb is that the maximum stone size is two-thirds of the layer thickness. A maximum thickness after compaction of 200 mm is usually specified, but allowance is generally made for the engineer to agree to thicker layers if the compaction requirements can be met. This is an aspect to be considered in deciding the type of excavation and blasting. Hole spacing and the width of lift have an effect on the size of the blast rock.

3.2.1.1 Rock Fills

Many specifications, e.g., the Standard Specifictions, TRH14 and Chapter 4 of this manual, specify limits regarding the maximum size of rock fill. Emphasis is placed on the filling of voids with end tipping and correcting rock fills deficient in fines.

The type of rock and its physical condition or state of weathering are of paramount importance in accepting materials for use as rock fill. Certain rock types, such as shales and mudrocks, deteriorate rapidly on exposure to air and moisture to their constituent materials, silts and clays.

Some rocks with high secondary mineral contents, such as some dolerites and basalts, decompose rapidly and massive rock can quickly turn into a gravel or grit under certain conditions. Some tillites also have rapid rates of weathering and these materials, except in their fresh state, are less frequently used as rock fill or as parent rock for the production of aggregates. If used inadvertently, collapse settlement (Chapter 7: 4.2) may occur as the interlock between fragments deteriorates when the rock structure breaks down. The situation is exacerbated by high moisture regimes.

When a material is suspect, the materials engineer should consult an engineering geologist. Early recognition of such characteristics is paramount to the economic and rational utilisation of materials.

3.2.1.2 Soils and Gravels

In the case of soil or gravel materials, a G10 material (TRH14 classification, see Chapter 4: Appendix A) is generally specified for the construction of fills. Grading and plasticity limits are not specified. The range of materials that are utilised is thus fairly wide. The Standard Specifications, for example, specify that, where possible, by virtue of the quality of the available materials, the CBR requirements given in TRH14 and in Chapter 4: Appendix A are met. The Standard Specifications thus allow for the use of materials not meeting these requirements, though great care should be exercised as materials of lower quality are:

- More **difficult to compact**.
- Generally at moisture contents close to or **exceeding the optimum moisture content** for the compaction equipment.
- Require more effort to **shape**.
- Often exhibit **high volume changes** leading to cracking when subject to wetting or drying in place. These cracks promote further drying out, and differential wetting and swelling in more active materials.

To limit the effect of these less desirable properties and behaviours, special placement techniques are utilised. These include encapsulating these materials in the centre of the fill with a specified cover of good material on the outer edges, above and below the encapsulated section. TRH9 recommends restricting the use of lower quality materials to within 1 to 6 metres below the subbase layer, and not closer than 3 metres to the outer edge.

Another technique is to place alternate layers of the poorer material and better quality materials. This allows for the release of pore water pressures that may develop, especially under compaction of moist clayey materials, and to a lesser extent also bridges the poorer materials. Where the stability of the fill may be compromised by the use of these materials, geogrids are placed at intervals, thereby adding tensile properties to the completed layer works and reducing differential consolidation. The design of such special measures should be done by, or in consultation with, a geotechnical engineer, especially for large or high embankments on major routes.

It is important that the investigations described in Chapter 6: 6 and Chapter 7: 4 and 5 recognise materials with the potential for rapid degradation after exposure and processing. Some materials types, such as shale and siltstones, and even some dolerites and basalts with high secondary mineral contents, may be excavated as hard rock but degrade to spoil material in a short time. If materials with these properties are not recognised, and appropriate steps are not taken to, for example, break them down, the degradation in place may be of such magnitude that the constructed fill may become un-trafficable due to deformation, or unstable with the ingress of moisture in moisture sensitive materials.

3.2.2 Compaction of Earthworks and Fills

The densification of earthworks/fill material is less about load bearing capacity and, particularly in high fills, more about reducing the capacity for collapse settlement. Typically 90% MDD is required on earthworks, with 100% for sands. In special circumstances, such as widening of an existing road prism, 93% of MDD is required in an attempt to avoid differential settlement between the newly widened portions and the existing structure.

All man made fills experience some settlement/consolidation after construction, which usually increases with increased height of fill. This is caused by traffic loading, vibration, gravity and moisture movements. It generally takes place gradually, decreasing over time (say 2 to 3 years). The intention of earthworks compaction is to reduce this settlement as far as possible. In the past, earthworks often took several months to construct and the fills were exposed to the elements, which allowed water ingress. Some of the settlement therefore took place before construction of the layer works and surfacing. In recent times, rapid completion of sections of road works has often resulted in sufficient post construction settlement of the higher fills to cause cracking (generally longitudinal), particularly at or outside the yellow line. Possible measures to counteract post construction settlement are:

- Insisting on **compaction at OMC**, instead of dry filling which can often achieve 90% MDD.
- Programming construction to allow some **time before constructing the layer works**.
- Compact in **layers of 300 mm**.

In the case of high fills, where the design is carried out with the assistance of a geotechnical engineer (as described in Chapter 7: 2 and 4), higher compaction limits are sometimes specified. For fill widenings, a G9 material is frequently specified (see Chapter 4 and TRH14). The Standard Specifications further require compaction to at least 93% of MDD in these operations.

Where the cut or imported fill material is cohesionless sand, compaction to 100% of MDD is required. These materials are described in detail in Chapter 4: 2.8.3.

3.2.3 Widening of Fills

Standard Specifications require that where fills are widened or where fills are to be constructed on steep side slopes (where the natural cross fall exceeds 1 in 10), benching into the existing roadbed or constructed layer works is carried out to bond the new works to the old or existing works. The tandard Specifications allow for smaller benches into rock, and maximum bench heights of 500 mm into existing fills in soft materials, except in the case of shallow fills where the height of the bench is similar to the layer thickness prescribed for that position. An example of a bench with a drain is shown in Figure 11.

Benches are generally constructed to have a slight inward fall, making it important that adequate drainage is maintained. Good practice is to install a collector or filter against the vertical face of the various benches, which intercepts groundwater seepage and seepage along soil and rock interfaces, and drains into a subsurface drain or drainage layer. The drainage layer, comprising of geotextile wrapped drainage stone, has slotted collector pipes and outlets, placed at regular intervals.

Figure 11. Drainage Bench

The maintenance of cross drainage when fills are constructed on sloping ground is essential. Cut off drains may need to be constructed to intercept seepage within strata or sediments. Specifications, such as the Standard Specifications and TRH9, are good references for the design of benches and the drainage thereof.

Where benching into rock is required, care must be exercised not to undercut unfavourably inclined strata. Where this is unavoidable, the advice of a geotechnical engineer should be obtained. Where benches are cut into rock, the outer edge or toe should also be constructed from rock with similar properties as that in the cut, taking care to ensure good interlock of the rock fragments. To some extent this provision minimises differential consolidation across the width of the road. See also Section 3.2.5.

For national and other major roads, a higher density is generally specified for fill widening to limit differential settlement. The Standard Specifications specifies that the materials be compacted to 93% of MDD. Similarly, G9 or even G7 materials are sometimes specified, as opposed to normal G10 materials, as these are generally easier to compact and provide higher shear strength.

3.2.4 Cut and Fill Transitions

In steep topography, generally associated with incised gorges, the transition from fill (essentially compacted soils or gravels) to a hard rock roadbed in a cutting, provides a risk of transverse shear cracking following the consolidation of the fill materials. This particularly occurs with horizontally or near horizontally bedded sediments. Generally, the higher the fill, the greater the potential settlement. The situation is often exacerbated by seepage from the cutting into the fill from an upgrade situation and can lead to collapse settlement if there are compaction deficiencies. It is therefore good practise to feather out such transitions by lowering the floor of the cutting in a series of steps or benches in a longitudinal direction, and continuing the upper fill layers into this transition zone. The benches cut into the rock must be adequately serviced with a drainage system possibly including vertical filters, horizontal drainage layers with collector pipes, and outlets protected against erosion. Adequately drained, impermeable barriers may also be used to prevent migration of moisture within the constructed layers from entering the fill from an upgrade position. For benches into rock, a rough or serrated surface increases the resistance of the constructed layers to sliding.

3.2.5 Mass Haul and Materials Utilisation Diagram

A basic mass diagram is a line diagram, related to a longitudinal section with the proposed grade line of the road indicating per position (km value), what quantity of materials are needed for fill or are available from cut. There is a balance line, termed the zero balance line, which defines fill and cut locations in the mass earthworks. An example of a mass haul diagram is shown in Figure 12. The same numbering of consecutive cuts and fills shown on the soil survey sheets must be used on the section. A table is generally provided below these drawings indicating:

- **Position and numbering** of cuts and fills
- Expected **volumes of cut** materials available for use in the earthworks
- **Bulking factors** used to derive these figures (see below)
- Allowances, if any, for **clearing and grubbing**, the removal of topsoil and for proof rolling
- **Volumes of fill** materials required

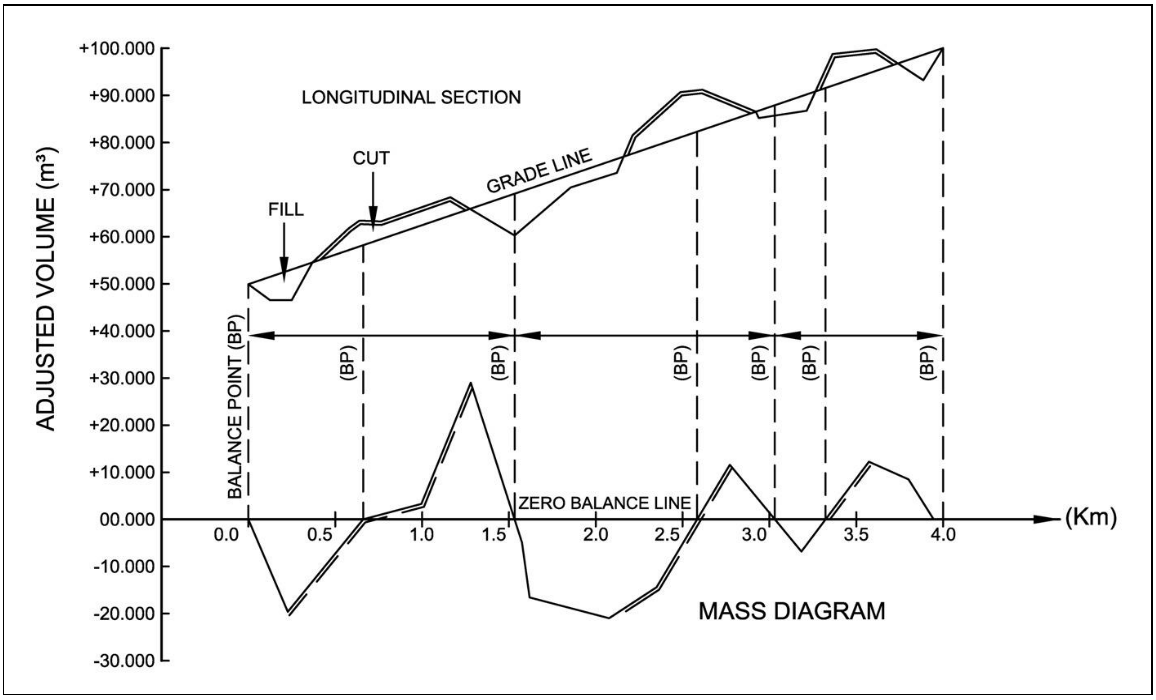


Figure 12. Mass Diagram Concepts

Adjustment for bulking is generally only applied to excavation quantities. Bulking factors for soft materials vary from 0.75 to 0.85, while for hard materials a bulking factor of 1.1 is often used.

A rising line shows cut while a falling line shows fill. Note that the line shows the accumulated amount of surplus or shortfall, not quantities of cut or fill at that point/station. A horizontal line, which closes a loop in the curves formed by the rising and falling line, is called a balance line and indicates positions where cut and fill quantities are balanced, see Figure 12.

A mass haul diagram, shown in Figure 13, is a further refinement of the mass diagram in that it shows for sections of the route:

- **Volumes of the materials**, in each cutting, to be hauled to which position, whether to fill (position indicated), to stockpile (for indicated use which may be for fill, gravel pavement layers or for processing) or to spoil. The effect of such usages is indicated by adjusted balance lines as shown in Figure 13. The vertical shift between consecutive balance lines reflects the quantities of borrow, spoil or materials stockpiled for other uses.

- **Direction of haul** as indicated by the arrows.
- The various calculated **Economic Limits of Haul** (ELH) for the sections indicated, where the ELH is calculated according to Equation (1).

$$\text{E.L.H.} = \text{FH} \times \frac{\text{unit price of borrow}}{\text{unit price of OH}} \qquad (1)$$

Where FH = Free haul, defined as the maximum distance through which material may be transported without added cost to the tendered rate

OH = Over haul, defined as the product of volume and distance. It is represented on the mass haul diagram as the area between the zero balance line and the curve after eliminating the free haul. Note that over haul is calculated using in situ volumes (i.e., unadjusted volumes).

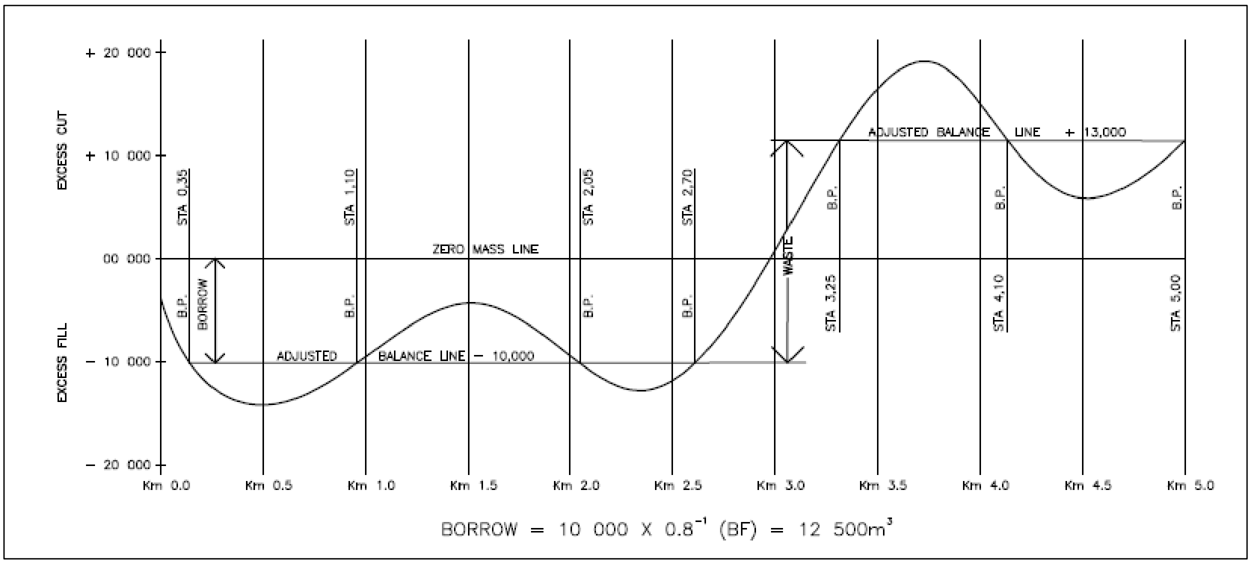


Figure 13. Use of Mass Haul Diagram

Computer programmes that cater for the various scenarios and options are used extensively to produce mass haul diagrams and materials utilisation diagrams. This allows for optimisation of alignment and economising the mass earthworks.

A materials utilisation diagram is a further refinement of the mass haul diagram. It gives the total materials utilisation for the construction of the road and all the ancillary and associated works. Each requirement is listed, generally in vertical order and showing in horizontal (km scale) segments the position and extent of the proposed use, its source, i.e., from cutting, stockpile, borrow pit, quarry or commercial source. This diagram is thus, in effect, a summary of the raw materials utilisation for the project and should be very carefully checked for accuracy and agreement with other documentation.

Excerpt from the Standard Specifications on Material Utilisation

"The engineer shall have full control over the use of all material from cut excavations, but the contractor shall plan the operations and particularly the cut - and - fill operations in such a manner that all cut material is used to the best advantage of the employer. This would mean that no material shall be unnecessarily spoiled, borrowed or hauled."

Materials Utilisation Diagram

This diagram is a summary of the raw materials utilisation for the project and should very carefully be checked for accuracy and agreement with other documentation.

4. PAVEMENT LAYERS: SOILS AND GRAVELS

The structural layers of most roads normally comprise of gravels, in either their natural state or after treatment with some form of stabilizer. For higher standard roads, they are likely to be constructed of crushed stone. Typical road materials are illustrated in Figure 14. These structural layers consist of the subgrade, selected materials, subbase and base course layer. These layers are probably the most important ones in the pavement structure, as it is their purpose to spread the applied traffic loads such that the subgrade and fill layers are not overstressed. Gravel materials may also be utilised for the construction of seals for lightly trafficked roads, particularly sand and Otta seals. This section discusses the use of natural gravel materials that are placed on the earthworks, and/or fill where necessary, as covered in the preceding chapters, as well as those for sand and Otta seals.

Figure 14. Typical Road Materials

4.1 General Concepts and Objectives

When designing gravel structural layers, whether for a new facility or for the widening of an existing road, the designer strives to minimise the quantity and handling of materials. For natural gravels, sufficient sources should be available along, or as close to, the road alignment as possible to provide the necessary materials within economical haul distances. The materials are generally obtained from within the road prism or borrow pits adjacent to the road reserve, where the materials can be worked using conventional bulldozers or excavation equipment. In some cases, suitable materials are obtained from cuttings of reasonably large dimensions. To enhance environmental sustainability, the use of materials extracted from within the road reserve is the optimum scenario.

A useful philosophy when using natural gravels, particularly, but not exclusively, for low volume roads, is to try and make the pavement design fit the available materials. Typically, a structural design is produced and then the search starts for the materials required for the design. If large quantities of local material with marginal properties are available, the pavement design can be revisited in an attempt to make optimal use of these materials.

Gravels for "low cost" surfacings consist of sands passing the 16 or 20 mm sieve, usually obtained from dry river courses or from suitable gravels obtained from borrow pits.

Material Utilisation from the Road Reserve

To enhance environmental sustainability, the use of materials extracted from within the road reserve is the optimum scenario.

4.2 Material Preparation

The proposed material for preparation of the natural gravel is located and identified as described in Chapters 6, 7 and 8. Prior to commencement of the operation, sufficient investigation of the borrow pit and/or cuttings is carried out to accurately delineate the limits of the suitable materials, both horizontally and vertically. The investigation should also identify the typical material properties within the borrow pit or cuttings, and the expected variation of these.

On the basis of this information, the material processing requirements and borrow pit operation can be planned. If the material contains excessive oversize material, i.e., larger than between 37.5 and 100 mm depending on the proposed use of the material, a decision must be made as to whether this will be screened out and spoiled, or crushed and added to the finer material. This decision is usually based on the quantity, size and hardness of the oversize material. If there are large volumes of oversize material, it is normally best to provide for crushing, to minimise the quantity of material spoiled and energy utilised in the processing.

Pioneer Layer

A pioneer layer is layer of sand, rockfill or other materials, and is constructed as a foundation or platform on which normal earthworks can continue.

If the oversize material is minimal, it is generally more economical to screen off this portion and spoil it, or use in drainage, pioneer layer or rock fill applications. These decisions must be made early to ensure that establishment of the necessary plant does not delay commencement of the material delivery.

4.2.1 Borrow Pit Operations

Potential borrow pits are identified during the initial site investigation, and areas of material of suitable quality for the various layers demarcated. It is essential that only these areas are processed to obtain the required materials.

The top soil containing organic material and vegetation, including roots and seeds, is stripped, usually by bull-dozing, and stockpiled alongside the borrow pit for later use during rehabilitation of the borrow pit. These stockpiles should be of limited height to avoid excessively anaerobic conditions and deterioration of potential seeds and rhizomes, which are necessary to encourage later revegetation.

Any material underlying the topsoil that is unsuitable for use shall be stockpiled separately from the topsoil, and can be used later for reshaping and smoothing the borrow pits, prior to replacement of the topsoil.

Once the required material is exposed, excavation can commence. Different types of material such as soils, rocks and boulders require different methods of excavation, ranging from scrapers, excavators and loaders to bulldozers, rock breakers and blasting for hard rock outcrops. If the material processed is uniform and consistent, it can be loaded directly into trucks, hauled to the road and dumped at the required spacing. This operation is frequently carried out using large backactors, making it a quick and relatively cheap operation.

However, more commonly there are relatively large variations in the material properties over short distances. In these cases, it is recommended that the materials are bulldozed and processed into suitable stockpiles during which significant mixing of the material occurs. The material is then hauled from these stockpiles directly to the road in a relatively consistent and uniform form.

Oversize material may need to be handled in the borrow pit to minimise the haulage of unsuitable material likely to be spoiled during construction.

In materials that are highly weathered (and possibly some moderately weathered) and relatively soft, it is usually possible to break down the oversize materials once placed on the road using grid rolling (Figure 15). This is best done where the bulk of the material is moderately to highly weathered, but does not contain hard spheroidal corestones (usually in acid or basic crystalline materials). This is because these are unlikely to be broken down by a grid roller, and need to be removed from the layer and spoiled, an often difficult labour-intensive operation that is seldom adequately done. Should grid rolling be considered to break down oversize material on the road, it is essential that trials are carried out to ensure that the technique is successful. If these trials indicate that the degree of breakdown of oversize material during grid rolling on the road is insufficient, the breakdown of oversize should take place in the borrow pit. Any materials broken down in the final layer on the road using a grid roller should be mixed after grid rolling to separate the individual broken particles. Compaction without separating the particles has the same result as compacting the unbroken material. Compaction should not be done simultaneously with the breaking down process, to ensure the individual particles are separated before compaction.

Figure 15. Grid Roller

Grid rolling in the borrow pit can be done instead of on the road. However, experience has shown that if the material cannot be successfully broken down on the road, it is unlikely that it will be effectively broken down using the same processes in the borrow pit. In these cases, screening or crushing should be employed.

Screening of the material involves the deployment of a portable screening operation in which material larger than a specific dimension is removed from the material. This is frequently done using a "grizzly" type screen consisting of a frame supporting a series of parallel steel bars separated by the required dimension. Material is fed through this screen using a front-end loader, either to stockpile or directly into vehicles. This obviously increases the material processing time considerably. The use of a grizzly is better than not removing the oversize at all, but usually results in a material in which only one dimension of the large particles complies with the specified maximum dimension, as large elongated particles pass through the "slats" easily. It should be noted that most specifications are very ambiguous in respect of the oversize dimensions.

The use of square "mesh" screens instead of a grizzly ensures that two dimensions of the particles comply with the requirements. However, the material processing time is longer.

There comes a point where single stage crushing (usually with selected screening) of the oversize material becomes cost-effective, although this can also be time consuming with considerably higher mobilization, set-up and operating costs. Although single stage crushing is relatively easy and cost-effective, achieving strict grading envelopes is usually very difficult. A set of screens may be necessary to achieve gradings complying with the G4 grading envelopes or better (see Chapter 4: 2.4 and Appendix A).

4.2.2 Material from Cuts and Other Excavations

In the majority of deep cuttings, the material being excavated varies from soil to weathered rock (natural gravel) to hard rock, which has the potential to be used as a high quality construction material for crushed stone layers or concrete. The possible application is identified during the material investigation stage and the necessary arrangements for collecting and separating the different materials are planned.

Only the moderately to highly weathered in situ rock is normally suitable for use as a natural gravel. This, however, may still require the removal of large boulders or crushing of this material to pass the specified maximum size for the specific gravel pavement layers. Typically, a simple single stage crusher (with or without appropriate screens) is sufficient to process this material.

During excavation of the cutting, suitably weathered materials are separated from the rock requiring blasting and the overlying topsoil, and transported to the nearby crusher site. This material is then passed through the crusher, stockpiled and available for use as natural gravel.

Similar principles apply to any other material excavated from the road prism outside cuts, although the quantities are normally considerably less. Where good material is available in the road reserve, use of this obviates the need for time consuming and costly Environmental Impact Assessment (EIA) studies, and it can be recovered for use in the pavement and replaced with material proposed for spoil.

4.3 Selected Layers

Selected layers need to comply with the simple requirements summarised and discussed in Chapter 3: 2 and Chapter 4: 2. The objective of a selected layer is to provide a platform for the subbase and base layers, and sometimes even another selected layer. This platform should be relatively uniform in strength and density, with a specified minimum bearing capacity, such that the performance of the overlying pavement is as uniform as possible.

Typical pavement designs in South Africa require this layer to be constructed with a G7 material on a G9 material.

4.3.1 Assessment of Materials for Selected Layers

As the selected layers are usually compacted in lifts of 150 to 200 mm, the largest allowable particle size is between 100 and 130 mm (two-thirds of layer thickness). The source shall be processed as described previously to ensure that this is achieved.

Materials for selected subgrade layers should be low cost and require minimal haulage. They are thus generally obtained from borrow pits alongside the road at frequent intervals. The material requirements are relatively few and not very high. Such materials are usually readily available.

4.4 Subbase Materials

The materials specified for subbase are of higher quality than the selected layers and are typical G5 quality with a minimum CBR of 45% at 95% MDD. For most rural roads, these are usually obtained from cuttings or borrow pits. Suitable materials are identified during the materials investigation and are generally worked as discussed in the previous sections of this chapter. On higher trafficked roads, i.e., more than about 1 million standard axles, it is often necessary to provide a stabilized subbase, usually C3 or C4 quality. Details on stabilization are discussed in Section 6. One of the reasons for designing a stabilized layer is to ensure that the overlying base course has adequate support for compaction.

Subbase materials of G5 quality have only one grading requirement, that is, a maximum stone size of 63 mm or two-thirds of the layer thickness. These materials may only require screening for use.

4.5 Gravel Base Materials

Apart from the higher material quality requirements, the design and utilisation of natural gravels for base courses differs very little from their use as selected layers and subbases.

The material for base layers must be significantly strong, with a minimum CBR of 80% at 98% MDD, and has more stringent grading and plasticity index requirements. Of particular importance is the maximum particle size criterion, as it is difficult to obtain an acceptable surface finish with materials larger than the specified 28 or 37.5 mm. It is often difficult to achieve the required grading with single stage crushing of hard materials removed from fill. Considerable experience is necessary to set up a multi-stage portable crusher to give the required grading.

The surface finish and tolerances are much stricter than for the underlying layers as this surface, under a thin seal, is essentially what dictates the final riding quality of the road. It is therefore essential that final trimming and shaping is of the highest order and oversize materials should be minimised to ensure a good base and seal adhesion. Figure 16 shows a seal that has stripped due to loss of adhesion between the base and seal.

Construction of Granular Layers

Chapter 12: 2.1 and 3.3, and Chapter 13: 3 discuss construction and quality management issues for granular layers.

Figure 16. Loss of Adhesion Between Base and Seal

4.6 Gravel for Shoulders

Unpaved shoulders are almost as important as the paved carriageway as these are the areas of deceleration in case of an emergency, and are often trafficked by large trucks. The materials used should thus provide adequate all-weather skid resistance, but should not provide any large particles that damage vehicles if they move onto the road. The paintwork of car bodies and many windscreens are damaged by particles as small as 13 mm (for example during resealing using chip seals) and larger particles can therefore have a severe impact on the damage to vehicles and overall road safety.

The best materials for unpaved shoulders are those that comply with the requirements for unpaved road wearing course gravels (TRH20), preferably with the maximum size reduced to 20 or 25 mm. This is most cost-effectively done by mechanical screening in the borrow pit. The material should be compacted to at least 95% MDD at optimum moisture content and rolled after final trimming, preferably with a large (> 20 tonne) pneumatic tyred roller. It is important that the crossfall is maintained at about 4%, as anything larger than this usually results in transverse erosion, and flatter crossfalls do not remove surface water adequately. The shoulder should meet the edge of the adjacent bituminous surfacing at the same level.

Figure 17 shows an unsatisfactory unpaved shoulder that has excessive oversize material, a drop-off from the surfacing to the shoulder and transverse erosion.

Like any unpaved road, unpaved shoulders wear and lose material with time, albeit at a much slower rate than unpaved roads due to being less trafficked. Routine maintenance is necessary, particularly to avoid high drop-offs from the bitumen surfacing, but also to avoid the possible collection of water in channels that frequently form at the junction of the surfacing and the shoulder gravel. This maintenance is usually carried out using a grader but to optimize it, this should be done after water spraying, and then rolled to produce a smooth compact surface. This is carried out, but not always routinely, by a number of road authorities in South Africa.

Drainage

Drainage is an extremely important consideration for pavements! Water is the primary cause of premature failure, accelerated distress and reduced structural capacity.

All aspects of drainage are comprehensively covered in SANRAL's Drainage Manual and not repeated in SAPEM. Download the Drainage Manual from www.nra.co.za.

Figure 17. Unsatisfactory Unpaved Shoulder

4.7 Natural Gravels for Surfacing Aggregate

Certain seals for lightly trafficked roads are constructed using natural gravels. These include sand and Otta (graded aggregate) seals. The different seal types are given in Figure 40.

4.7.1 Aggregates for Sand Seals

Sand seals consist of a combination of natural sand and a bituminous tack coat. Aggregate for a sand seal requires that 100% of the material is finer than 7.1 mm and the majority of the fines (< 0.3 mm) are removed. Many natural river sands comply with these requirements, although some scalping with the 7.1 mm screen is often required to remove the few larger particles present. Any large organic matter such as wood and roots must also be removed, which can be done at source in the river bed during the dry season. The use of such sands is considered environmentally friendly, as these sands are generally replaced each year when the river floods.

Residual sand deposits derived from the in situ weathering of coarse sandstones and fine granitic materials are also used for sand seals. However, these materials tend to contain more fine material and require screening at the lower sizes, and often scalping at 7.1 mm too. This process is usually slow and is only recommended for small projects.

4.7.2 Aggregates for Otta Seals

Otta (or graded aggregate seals) require material with a continuous grading between 20 mm and 0.075 mm, although the percentage passing 0.075 mm should be limited, and preferably less than 10%. Such materials are usually obtained from selected borrow pits occurring on residual acid crystalline rocks because the predominance of quartz in these materials ensures a sufficiently hard aggregate, or from alluvial gravels. The materials often require some scalping to remove oversize material. Requirements are outlined in TRH3 and in Chapter 3: 4.4.

River Sands

The use of such sands is environmentally friendly, as these sands are generally replaced each year when the river floods.

Seals

A good reference for seals is **TRH3:** Design and Construction of Surfacing Seals.

In **SAPEM**, seals are discussed in:

- Chapter 2, Pavement Composition and Behaviour, Section 2.3.1.2
- Chapter 3, **Materials Testing**, Section 4.4
- Chapter 4, **Standards**, Section 4.4
- Chapter 12, **Construction Equipment and Method Guidelines**, Section 3.10
- Chapter 7, **Quality Management**, Section 7

4.8 Mechanical Stabilization

Many local materials, and those from cuts, do not meet the requirements for the specified layers but can be improved by mechanical stabilization. The simplest and most-cost effective means of mechanical stabilization is by compaction, which is normally done in any case. The removal of oversize material is also a form of mechanical stabilization and is discussed in Section 4.2.

If material with the characteristics prescribed in the design cannot be located, the blending of two or more materials can be considered. Blending of materials is often a cost-effective means of improving the strength and quality of a material.

4.8.1 Blending

Blending Materials
Blending involves mixing materials with different properties, to form a material with improved characteristics.

Blending involves mixing materials with different properties to form a material with improved characteristics. Blending is typically done to improve plasticity and/or particle size distribution, and therefore the strength. The success of the blend typically depends on the limitations of the source materials.

Ternary diagrams, shown in Figure 18 and Figure 19, are useful for determining the optimal particle size distribution. The Atterberg Limit tests are then used to check plasticity. Input parameters for the ternary diagram are the percentages silt and clay (< 0.075 mm), sand (0.075 – 2.0 mm) and gravel (2.0 mm – 37.5 mm) of the material sources that will be used. A typical example of a material containing 25 per cent gravel, 25 per cent sand and 50 per cent silt and clay, is drawn on the ternary diagram in Figure 18 as follows:

- Read off the **25 per cent gravel** on the right hand side gravel axis. This is illustrated with the green arrow that goes horizontally into the diagram.
- Read off the **25 per cent sand** on the left hand side sand axis. This is the red arrow that goes parallel to the gravel axis.
- Read off the **50 per cent silt and clay** on the horizontal silt and clay axis as shown by the purple arrow that goes parallel to the sand axis.

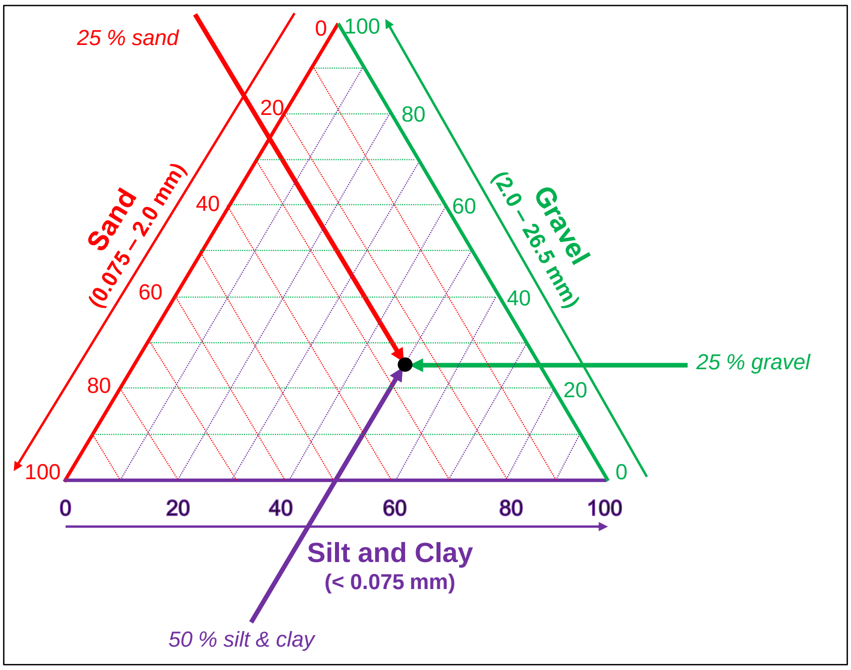


Figure 18. Ternary Diagram for Blending Road Materials

4.8.2 Blending Procedure

The following methodology can be used for determining the optimal mix ratio for blending two or more materials to meet the TRH14 road base specification. In Figure 19, the blue shaded area is the TRH14 specification for G4 materials. The methodology is illustrated with a specific example:

- Identify **potential material sources** that can be used to improve the available material.
- Determine the **particle size distribution**, i.e., the grading, of the material from each source (SANS 3001–GR1). Note that maximum size limits are specified for most "G" classes, and is either 37.5 mm, or not exceeding two thirds of the thickness of the compacted layer. For the example, the gradings of Materials A and B are given in Table 5.
- Determine the **percentages of silt and clay** (< 0.075 mm), **sand** (0.075 – 2.0 mm) and **gravel** (2.0 – 37.5 mm) for each source, see the bottom section of Table 5.
- **Locate the points** representing each material to be blended on the **ternary diagram**. See Material A in maroon and Material B in grey in Figure 19, with the arrows showing how the points are located.
- **Connect the** points with a straight line, shown in orange. When the two points are connected, any point on the portion of the line in the shaded area indicates a feasible mixture of the two materials. The optimum mixture should be in the centre of the shaded zone. For this blend, Point C in orange is the optimal mixture.
- The **mix proportions** are then the ratio of the line AC:BC. In this instance, the ratio is approximately 1:2.5, which indicates that one part of Material B should be mixed with 2.5 parts of Material A (i.e., two truck-loads of Material B for every five truck-loads of Material A). After blending, the grading will be within the general requirements for a G4 material.
- Once the mix proportions have been established, the **Atterberg Limits** of the mixture should be determined to check that the plasticity index is within the desirable range, that is < 6 for G4, < 10 for G5, etc. See Chapter 4: Appendix A or TRH14. For materials that are going to be stabilized or bound by other means, the quantity of binder added should be adjusted until the required Plasticity Index is obtained, while ensuring that the mix quantities remain within the optimal zone.
- If the **line does not intersect the shaded area** at any point, the two materials cannot be successfully blended and alternative sources have to be located, or a third source used for blending.

Table 5. Characteristics of Materials for Blending

Parameter	Material	
	A	B
Description	**Coarse material with insufficient fines**	**Fine silty clay**
% Passing		
37.5 mm	100	100
28 mm	90	100
10 mm	65	100
5 mm	32	87
2.00 mm	15	80
0.425 mm	4	51
0.075 mm	3	20
Plasticity Index (PI)	2	10
% silt and clay (P0.075)	3	20
% sand (P2 – P0.075)	15 – 3 = 12	80 – 20 = 60
% gravel (P37.5 – P2)	100 – 15 = 85	100 – 80 = 20

This blending procedure is based on three size fractions only (gravel, sand and silt and clay). Provided that the different materials used for blending are continuously graded with a smooth curve more or less parallel to the normal grading envelopes, the final grading will normally comply with the specification. If the materials are gap-graded or the particles have significantly different relative densities within a size range, the final grading curve may not be sufficiently smooth. Other techniques for blending, for example, Rothfuchs (TRH8) can also be used.

A flowchart summarising the procedure for mechanical blending is provided in Figure 20.

Blending: Mass vs Volume

The blending procedure described is based on material mass, whereas construction is usually based on volume. Provided that the densities of the materials do not differ significantly, the results can normally be transferred directly. Where significant differences in density occur, the appropriate volumes must be determined mathematically.

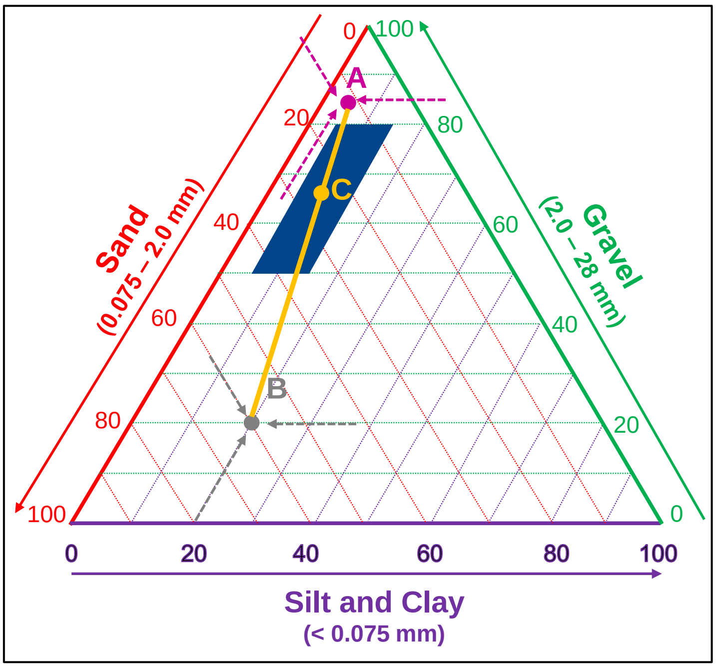


Figure 19. Example of the Use of the Ternary Diagram for G4 Base

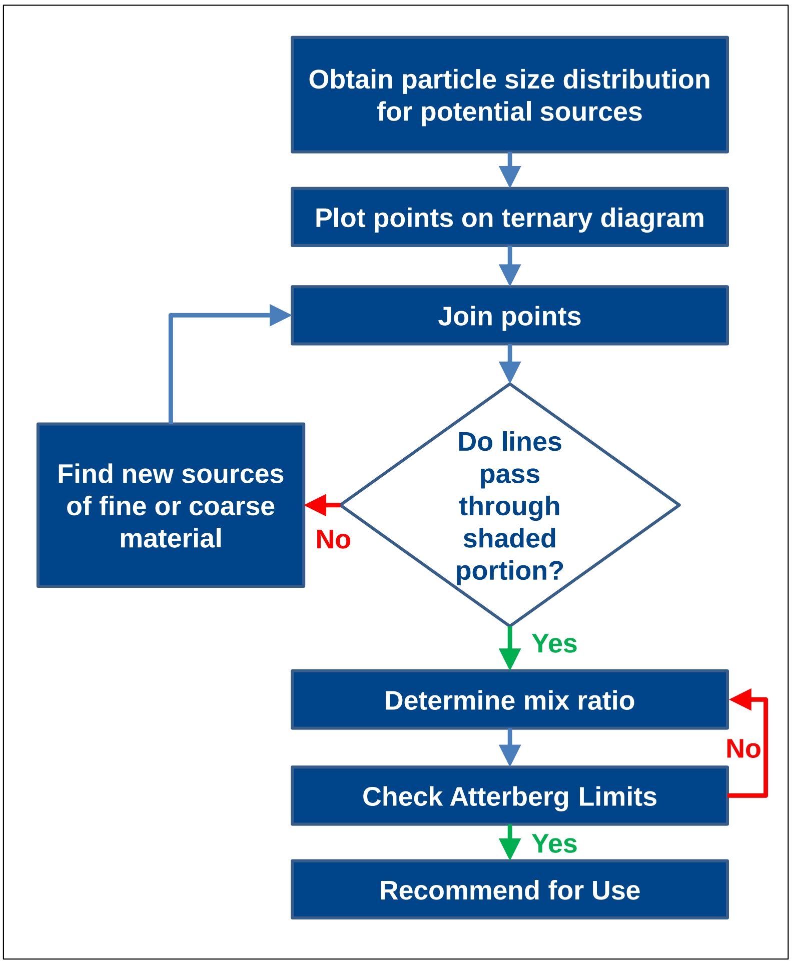


Figure 20. Flowchart for Material Blending

4.8.3 Construction with Blended Materials

Blending can be done at the gravel source, if the sources are in close proximity. However, is usually most effective when carried out on the road. When mixing on the road, the materials should be clearly separated, for example, the material of smaller quantity can be dumped on the opposite side of the road. The first material (larger quantity) should then be evenly spread to a uniform thickness. The second material is then spread on top of the first. The grader then mixes the two materials by cutting to the full depth of the material, turning it over and spreading it across the balance of the road. A plough can usually be used as effectively as a grader. In situ recyclers may also be used for blending, but because they mix predominantly in a vertical plane, the mixing must be done with care. Guidelines for effective mixing are given in Chapter 12: 2.7 and 3.5.

Careful supervision and control of the dumping location and spacing and mixing of the material is necessary to ensure that the correct proportions are combined, and mixing is complete and homogenous. Conventional quality control tests must be carried out on site to check the final grading of the material.

4.8.4 Plasticity Adjustment

As the plasticity index test is only performed on a selected portion of the entire material, any adjustment to the grading has a diluted effect on the plasticity index. It is suggested that the desired grading is achieved by blending as the first priority, after which adjustments to the material can be made to achieve the desired plasticity. This is usually done by the addition of non-plastic materials to decrease the PI. However, care must be taken to use materials with a similar apparent bulk relative particle density to the original material and to ensure that the grading curve is not disturbed too much, resulting in the development of irregularities and gap-graded materials. Particles of a single size, or a limited size range, with very high or low densities result in inconsistencies in volumetric terms.

5. PAVEMENT LAYERS: CRUSHED STONE AND WATERBOUND MACADAM

Many base courses and some subbases consist of high quality crushed stone that provide a stiff, yet adequately flexible layer, to resist and spread the high stresses applied by traffic in the upper portions of the pavement structure. Crushed stone is also used for aggregate in asphalt, chip seals and concrete, and is increasingly being used for waterbound macadam layers. A typical illustration of crushed stone is given in Figure 21.

These are particularly useful for labour based projects, but have the added benefit of being less water sensitive than conventional compacted crushed stone layers.

Crushed Stone Layers

Various aspects of crushed stone layers are discussed in:

- Chapter 3: **Materials Testing**, Section 3
- Chapter 4: **Standards,** Section 3
- Chapter 8: **Material Sources**, Section 3.2.2
- Chapter 10: **Pavement Design**, Section 7 and 8
- Chapter 12: **Construction Equipment and Method Guidelines**, Section 3.8
- Chapter 13: **Quality Management**, Section 3

Figure 21. Crushed Stone

5.1 General Concepts and Objectives

In designing crushed stone layers, the designer strives to minimise the required quantity and handling of materials. Crushed stone for subbases and bases are normally provided by a commercial quarry, although on large projects a quarry may be established at a suitable material source to supply aggregate to the project.

5.2 Waterbound Macadam

Macadam layers are an ancient form of a granular layer. There are several types of macadam bases that have been constructed in South African pavements. The different types are summarized below:

- **Dry-bound Macadam**. The dry bound macadam layer is constructed with an almost single sized aggregate (usually 53 mm nominal size) and a dry cohesionless fine aggregate. The single size stone is placed on the road and the voids in the layer are filled with fine aggregate through the use of compaction equipment only. No water is used during the construction of this layer.
- **Waterbound Macadam**. There are two waterbound processes, dry and wet.
  - The term waterbound is generally used to describe a **dry-bound macadam**, which has been slushed after all the voids have been filled with dry fine aggregate and thereafter slushed. The slushing process consists of saturating the macadam layer (coarse and fine aggregate) by spraying it with water, after which a number of passes are made with a steel drum roller, forcing the excess fines to the surface of the layer, from which they are then swept away.
  - A completely **wet process** has been used in areas where the climate did not allow the fine aggregate to dry out sufficiently to flow into the voids of the macadam layer. The fine aggregate is spread over the layer using a chip spreader and washed into the coarse aggregate layer with water-jets from a spray-bar.
- **Penetration Macadam**. Penetration macadam layer is constructed using only the coarse aggregate and tar or bitumen. Fine aggregate is not used to fill the voids between the coarse aggregate. Hot tar/bitumen is poured over the coarse aggregate layer and flows into the voids,

Waterbound Macadam for Road Widening

Waterbound macadam layers are useful when widening a road or working in restricted areas, such as wedges. Lighter equipment can be utilised to achieve the required "strength" of the layer without damaging the adjacent granular pavement. In addition, Waterbound Macadam layers allow lateral drainage of moisture, and thus prevent the development of the so called "bath-tub" situation, which invariably results in premature failures.

coating the large aggregate in the process. The voids are, however, not filled completely by the tar/bitumen.

- **Partially Penetrated Macadam**. A partially penetrated macadam layer is constructed using a coarse and fine aggregate in the usual way. The excess fines aggregate is thereafter swept off the surface of the layer. Some of the filler in the open voids at the top of the layer is, however, also swept away, resulting in a very rough surface with the coarse aggregate projecting from the layer. Slurry is then applied to the layer with a slurry-box to fill these open voids on the surface of the layer. Slurry penetration is normally relatively shallow for this type of macadam.
- **Slurry-bound Macadam**. A slurry bound macadam is constructed using a coarse aggregate and a slurry. As in the case of penetration macadam, no fine aggregate is used to fill the voids between the coarse aggregate. A slurry, produced from crusher sand (or a mixture of crusher sand and natural sand) and emulsion, is forced into the voids between the coarse aggregate. The slurry therefore performs the function of the tar/bitumen in a penetration macadam, but all the voids are filled with slurry in this case, in contrast to the partial filling of the voids in a penetration macadam layer.
- **Composite Macadam**. A composite macadam layer is constructed using a lower portion of dry or waterbound macadam, usually with a nominal 50 mm stone size, and of a top portion consisting of a slurry-bound macadam, usually with of a nominal 28 or 37.5 mm stone size.

The most commonly macadam used in the past is the waterbound macadam, and this manual therefore focusses on this type of macadam base. A waterbound macadam layer is shown in Figure 22. The design of waterbound macadam layers is essentially the same as crushed stone layers. Waterbound macadam is well suited to areas with sufficient quantities of non-plastic dry sand. Waterbound macadam is also useful for widening roads. Waterbound macadam is permeable, and, therefore, does not trap water. The stone-on-stone contact of the large aggregate can make it less susceptible to weakening upon water ingress than conventional crushed stone layers.

Figure 22. Waterbound Macadam

In most cases, the finished profile of a waterbound macadam layer is relatively uneven and rough. As a result, the application of a spray or cape seal is invariably inappropriate to achieve an acceptable riding quality. Some form of asphalt or a slurry levelling course is normally required. On high speed roads, two asphalt layers are often required to obtain a suitable riding surface.

5.3 Material Properties

Crushed stone layers can consist of various types of crushed stone, as discussed in Chapter 4: 3.3. The use of crushed waste materials such as mine dump rock, slag and ash also fall under crushed aggregate and should not be forgotten, particularly with respect to environmental and sustainability benefits. The highest quality crushed stone material, G1, must be non-plastic, must not contain clay minerals susceptible to rapid chemical weathering, and must have a very tightly controlled grading to ensure that the required compaction density specified, normally 86 to 88% of apparent relative density, can be achieved. Any significant deviation from the specified grading outside the recommended tolerances given in Table 6 negatively affects the chances of obtaining the specified density.

Most specifications require that the material is produced from fresh stone by quarrying. Many industrial waste materials can be processed to G3 or better quality. Their use should be considered in the project specifications if they are readily available within economical haul distances.

Table 6. Tolerances on Target Grading for G1, G2 and G3

Sieve size (mm)	Permissible deviations for mean values (% by mass)		Permissible deviations for individual values (% by mass)	
	Nominal Maximum Size			
	37.5 mm	28 mm	37.5 mm	28 mm
28	± 5	–	± 5	–
20	± 5	± 5	± 7	± 7
14	± 5	± 5	± 7	± 7
5	± 4	± 4	± 7	± 7
2.00	± 4	± 4	± 5	± 5
0.425	± 3	± 3	± 5	± 5
0.075	± 2	± 2	± 3	± 3

5.4 Crushing of Aggregate

The material to be crushed for aggregate is identified during the site investigation phase and consists of unweathered rock sources of the required quality. These sources are exploited by drilling, blasting and crushing at an approved quarry site complying with the requirements of the Department of Mineral Resources (DMR) and Energy. See Chapter 8: 2.5 for the DMR requirements.

The crushing process is best carried out by commercial crusher operators. However, on large contracts, the contractor may set up a crusher if the necessary skills, experience and qualification to mine and process the crushed stone are available. The availability and use of mobile crushing equipment is significantly increasing. It must, however, be noted that the selection of appropriate crushing equipment for specific material types is important. Experience in choosing the correct equipment is available from the equipment suppliers, as well as many of the local quarrying companies, and their advice should be sought. The prevailing geology often dictates the crushing needs and plant configuration to handle specific geological conditions.

5.5 Re-blending Crushed Stones

For high quality crushed materials, G1 and G2, it may be necessary to separate, screen and re-blend various fractions to achieve the necessary particle size distribution. Bearing in mind that the specified gradings are for the material after construction, suppliers need significant experience to provide a material that complies with the grading, especially the tight G1 tolerances, after compaction.

5.6 Utilization and Design Using Crushed Stone

The use of crushed stone in the pavement design requires no specific adaptations or innovations other than a good understanding of pavement design and material requirements. The material requirements are clearly defined in the Standard Specifications. Any deviations from these should be based on significant experience with such materials, or minor corrections on site as a result of material variations.

Aspects such as decisions regarding the target densities, for example, 86 or 88% of ARD; or, 98 or 102% of MDD, should be based on experience and the requirements of the pavement design.

Crushed stone for seal surfacings is also clearly specified and a number of the material properties may require specific modifications to the seal design. Obviously, aspects such as the particle shape, particularly in terms of the Average Least Dimension (ALD), affect the seal design significantly and their consideration is essential. Other aspects such as binder adhesion of specific material types may dictate whether to precoat the aggregate.

Using Industrial Waste Materials

Most specifications require that the material is produced from fresh stone by quarrying. Many industrial waste materials can be processed to G3 or better quality. Their use should be considered in the project specifications if they are readily available within economical haul distances.

Selecting Crushing Equipment

The selection of appropriate crushing equipment for specific material types is very important. Equipment suppliers as well as many of the local quarrying companies, have great experience in selecting the appropriate equipment, and their advice should be sought. See Chapter 8: 3.2.2 and Chapter 12: 2.1 for information on crushers.

The durability and performance of crushed stone depends primarily on the possible presence of deleterious minerals and components. Materials such as clay, shale, soluble salts, unhydrated calcium or magnesium oxides, cause durability problems and are discussed in Chapter 4: 2.7.

5.6.1 Suitable Materials for Waterbound Madacam

The coarse aggregate to be used in the construction of a waterbound macadam base should be obtained from the crushing of unweathered hard durable rock, with an ACV not greater than 29 and a Flakiness Index of less than 35. The grading envelope for the course aggregate has the common characteristics of an almost single particle size distribution.

The fine aggregate to be used in the construction of a waterbound macadam can be obtained from two sources: the crushing of the same parent material used for producing the course aggregate, or, a natural fine sand source. The natural sand sources should, however, be screened for PI and salt content. The PI should preferably be between 4 and 9. The grading envelope for the fine aggregate should have a continuous particle size distribution. A blend of crusher dust and natural sands may also be used to achieve the required particle size distribution of the fine aggregate.

6. PAVEMENT LAYERS: CEMENTITIOUS

It is frequently found that the local materials are of inadequate quality to provide the required structural capacity. The alternative is to import a higher quality material from further away, or to treat the local material with some form of mechanical or chemical action to raise its quality. This section discusses the treatment of natural gravel materials using chemicals to improve their properties for use in selected, subbase or base layers.

6.1 General Concepts and Objectives

In deciding on mechanisms for improving natural gravel materials by treatment, it is essential that the most appropriate and cost-effective technique is identified and employed. This must involve the minimisation of the quantity and handling of materials. Sufficient sources should be available along, or as close to, the road alignment as possible to provide the necessary materials for treatment within economical haul distances, as the treatment action usually increases the cost of the layer significantly. The materials for treatment are generally obtained from borrow pits in which the materials can be processed using conventional bulldozers or excavation equipment. Attempts should be made to offset as much of the stabilization cost as possible by reducing haul distances.

Various types of treatment can be applied to the materials. The use of conventional or traditional products such as lime, cement, fly ash, ground granulated blast furnace slag (GGBS) or other pozzolanic materials as well as bitumen can all be considered for material improvement. It should be noted that many of the available cement types are sold with extenders, e.g., fly ash, GGBS, and limestone, as part of their composition and it is not common now to blend different cement-extender combinations on site. There is also a plethora of proprietary or non-traditional chemical soil stabilizers available locally and these can be considered for use under certain circumstances, mostly for the upgrading of unsealed roads to low volume sealed standard. Brief discussions on these are included in Section 6.3 and Chapter 3: 6 and Chapter 4: 6.2.

6.2 Traditional Chemical Stabilization

The traditional chemical stabilization design process involves the following procedure:

- **Sampling and laboratory testing** of the borrow pit or in situ material.
- Determine the **required structural properties** of the stabilized layer, i.e., modification or stabilization.
- Initial proposal of **types and quantities of stabilizer** required.
- Checking the proposal through **appropriate laboratory tests,** leading to acceptance or adjustment of the type and quantity of stabilizers, in accordance with the results achieved.

The flow diagram shown in Figure 23 gives this process in more detail and can be used to ensure that all of the necessary steps in the stabilization design are followed.

Sourcing Materials for Stabilization

Sufficient sources should be available along, or as close to, the road alignment as possible to provide the necessary materials for treatment within economical haul distances, as the treatment action usually increases the cost of the layer significantly. Attempts should be made to offset as much of the stabilization cost as possible by reducing haul distances.

Cementitious Stabilization

Various aspects of cementitious stabilization are discussed in:

- Chapter 3: **Materials Testing**, Section 5
- Chapter 4: **Standards,** Section 5
- Chapter 10: **Pavement Design**, Section 7, 8 and 9
- Chapter 12: **Construction Equipment and Method Guidelines**, Section 3.4
- Chapter 13: **Quality Management**, Section 4

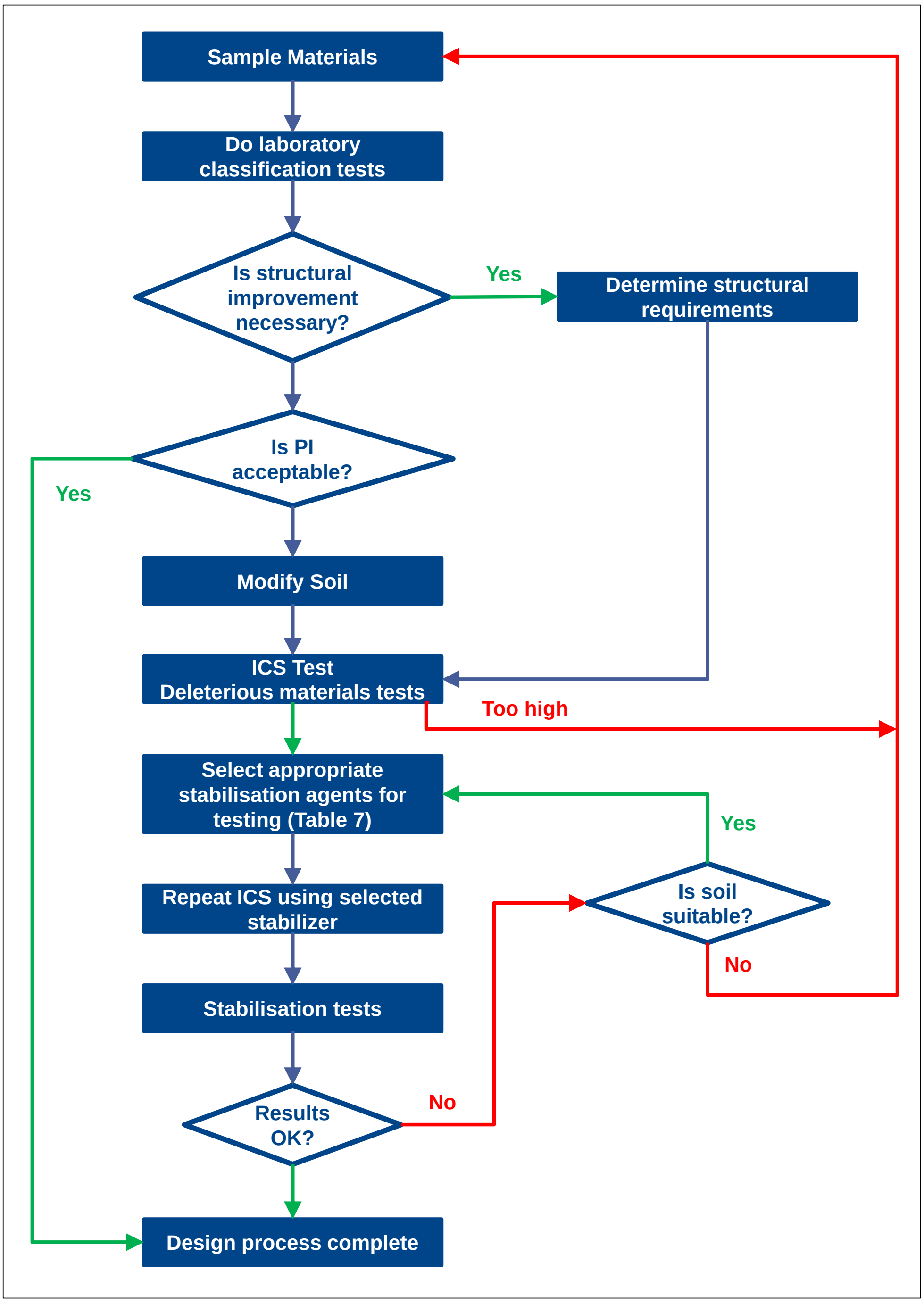


Figure 23. Stabilization Design Process

6.2.1 Initial Sampling and Testing of Material

It is essential that the samples for testing are representative of the material to be used during construction. The number of samples tested is dependent on the variability of the source. Details regarding sampling are discussed in TMH5. Each sample should consist of sufficient material to allow a full range of natural and stabilized soil tests to be performed. Approximately 150 kg is usually sufficient, but this needs to be increased for materials that have a high percentage of coarse aggregate that are scalped and discarded during testing.

Initial testing entails establishing the grading, plasticity and CBR of the natural soil. This provides sufficient information to determine whether the soil requires structural improvement or PI modification. Should the natural material require stabilization, the Initial Consumption of Lime or Stabilizer (ICL or ICS, SANS 3001 GR–57) should be carried out to determine whether chemical stabilization would be cost-effective. The ICL/ICS test is discussed in

Chapter 3: 5.3.1. Typically, a high ICS indicates that excessive stabilizer is required. If cost analyses show that this is not cost-effective, alternative materials should be sought.

It is important that the natural material to be treated is of suitable quality to produce an appropriate stabilized layer. Materials of highly variable quality are extremely difficult to stabilize and construct effectively. Variability in the required stabilizer content and the compaction moisture can result in localised areas of poor quality. It is suggested that material of at least G6 quality be selected for use in stabilized layers. Material inferior to this is more likely to be variable, contain deleterious components and require higher stabilizer contents. Typical deleterious components include:

- Sulphates
- Soluble salts
- Sulphuric acid
- Secondary minerals, especially smectite clays
- Organic compounds

Upper Selected Layers in Cuttings

It is recommended that upper selected layers of all pavement classes within cuttings are stabilized.

6.2.2 Pavement Strength Requirement: Cementation or Modification

The decision on whether to modify or stabilize with cement, or both, depends on the function of the pavement layer in which the material is used. Stabilized base and subbase layers must have adequate strengths and durability, and use materials that generally contain some coarse aggregate. Cementation is used to achieve the required strengths, whilst modification may sometimes be used to reduce the plasticity of materials, which would otherwise be unacceptable. In the case of lime stabilization, higher quantities of lime may be required to ensure that cementation develops.

The practice of modifying a material and then cementing it, i.e., a double handling process, is a costly operation and the economics of the process should be carefully assessed before commencing such an operation. It can, however, in certain situations be the only option, for example, with certain weathered basic crystalline materials.

Strengths specified for selected layers can usually be achieved using uncemented finer grained materials with PIs limited to acceptable levels. Modification of otherwise acceptable materials with marginal PI's, especially in base materials, would reduce the PI as well as improve the strength and workability of that material. This strength gain, given similar compactive efforts, is normally not due to cementation, but results from the improved soil structure and increased density of the soil. It should be ensured that the strength of the natural material meets the required specification. It is possible that the PI of some materials may return with time should the treated material undergo carbonation and the pH of the modified material drops.

Roadbed materials are not stringently specified, but in situ materials can often have sub-standard strengths or high clay contents. The roadbed material can also be in a saturated condition during construction and require drying. In these cases, modification could be (but is seldom because of costs) used to:

- **Raise the CBR** by improving the structure of the soil
- **Reduce shrinkage and swell** in clayey materials by reducing the plasticity
- **Improve the consistency and workability** of the material by the apparent drying effect of stabilizers
- **Dry out the material** to increase workability if slaked lime is used.

It is often recommended that upper selected layers of all pavement classes within cuttings should be treated.

6.2.3 Initial Stabilization Proposal

6.2.3.1 Selecting the Stabilizer Type

When selecting the type of stabilizer, the following factors need to be considered:

- **Physical properties** of the material, e.g., liquid limit, plasticity index, clay content and ICS.
- **Purpose of stabilization**, e.g., for modification or cementation; for a rapid or slow increase in strength.
- **Availability and costs** of different stabilizers. Cost alone should, however, not dictate the use of a potentially inferior or inappropriate stabilizing agent over a better, but more expensive, one. Where cement stabilization is considered, the availability of specific cement types can be a problem in certain areas and it is imperative that only those cement types that are likely to be cost effective and available on site be used during the testing for the stabilization design. The different types of cement are discussed in Chapter 4: 5.1.2.

The following can be used as a guide to choose a stabilizer:

(i) Plasticity Index of the Material

The choice of an appropriate stabilization proposal depends on the PI of the material. Table 7 recommends appropriate actions for ranges of PI values.

Table 7. Recommended Stabilization Proposal Based on Plasticity Index

PI of Material	Recommended Stabilization Proposal	Comment
< 10%	Stabilize with cement	The modification reactions from the cement typically reduce the PI to less than 6% for these materials.
10 to 15%	Modify and stabilize with cement	Lime, lime-slag, lime-fly ash mixtures or possibly CEM V type cement should be evaluated.
>15%	Modify and improve strength through cementation.	These materials would normally not meet the requirements even for upper selected layers and should only be used if better material cannot be located. Lime is the recommended stabilizer and should always be used to stabilize basic crystalline materials at these PI levels.

Heavy clays or wet materials can be modified for improved workability and compactibility. Under these circumstances lime is usually used as the modifying agent. Materials that contain kaolinite as the predominant clay mineral usually have a fairly low PI with a high liquid limit. These materials should also be stabilized with lime.

After stabilization of materials derived from basic crystalline rocks, it is important that the PI is reduced to completely "non-plastic" (NP), and not only "slightly plastic" (SP).

(ii) Activity of the Material

In addition to the various physical properties of the material, it may be beneficial to consider the activity of the material in the selection of a stabilizer. The activity is defined in Equation (2), and is interpreted as follows:

- When A > 0.5, modification is necessary and lime should be considered.
- When A < 0.5, modification may not be necessary and cement, cement-lime blends should be considered.

$$A = \frac{PI_{gross}}{P_{0.002}} \tag{2}$$

where:

PI_{gross} = Weighted plasticity index of the sample (PI x $P_{0.425}$)
$P_{0.425}$ = Percentage passing 0.425 mm
$P_{0.002}$ = Percentage smaller than 0.002 mm

(iii) Grading

Table 8 serves as a guide to the identification of appropriate stabilizers for use in laboratory testing based on the material class. Material of poorer quality than G6 should only be treated for use as base or subbase where no other material is available.

G6 and Poorer Quality Materials

Material of poorer quality than G6 should only be treated for use as base or subbase where no other material is available.

Table 8. Suitable Stabilizers for Soils

Grading	TRH14 Class	Requirement	Application	Stabilizer
Fine	G8, G9, G10	Reduce plasticity	Modification	Lime
Coarse	G4, G5, G6, G7	Increase strength	Cementation	Cement Extended cements Lime blends

(iv) Initial Consumption of Stabilizer (ICS)

Where the lime demand of a material is found to be high, > 2% (see Chapter 4: 5.3.1), lime and lime blends with GGBS or fly ash should be considered. The use of low clinker CEM IV and CEM V cements could also be attempted. This is particularly relevant to soils derived from basic crystalline rocks. Where cement is proposed, the ICS should be determined using the type of cement that will be used.

(v) Selection of Cement Type

The performance of cements complying with SANS 50197-1 appears to be different from those complying with the previous SABS tests, i.e., 471 and 626. The test methods are different and the strength classes of cements are now based on the strength after 28 days and not 7 days. The tests and classes of cements are discussed in detail in Chapter 4: 5.1.2.

An investigation by Paige-Green and Netterberg (2004) was carried out on behalf of the cement producers through the Cement and Concrete Institute (C & CI). The study indicated that CEM II A and B cements using fly ash or granulated blast furnace cement as extenders, and CEM III A cements, appear to allow greater flexibility during construction than CEM I cements. The strength class of the cement should not generally exceed 32.5, although testing with Class 42.5 cement can be carried out for comparative purposes. It is, however, recommended that preliminary testing using a range of cements available in the area, and within the boundaries described, is carried out during the stabilization design process. As the actual composition of any cement can change periodically, the final mix design must always be checked with the materials that are eventually selected for use on site.

Check Mix Design with Available Cements

The actual composition of any cement can change periodically. The final mix design must always be checked with the materials that are eventually selected for use on site.

6.2.3.2 Quantity of Stabilizer

The quantity of stabilizer is based on that required to achieve the specified standard for the pavement layer. Probably the most important component of the design is to ensure durability. Insufficient stabilizer results in marginal strengths, an increased possibility of the pH of the material dropping, and the cementation products of the material becoming unstable.

Strength tests (UCS or ITS, Chapter 3: 5.3) are carried out on the material at different stabilizer contents. The stabilizer content versus the strength curve indicates the quantity of stabilizer required to produce design strengths according to TRH14, shown in Table 9.

Table 9. Requirements for Strength and Plasticity of Chemically Stabilized Materials

Criteria	C1	C2	C3	C4
Material classification before treatment	At least G2	At least G4	At least G6	
PI after treatment	SP	SP	≤ 6	
Design strength Lab UCS (MPa) @ 100% MDD @ 97 % MDD	 Min 6 Max 12 Min 4 Max 6	 Min 3 Max 6 Min 2 Max 4	 Min 1.5 Max 3 Min 1 Max 2	 Min 0.75 Max 1.5 Min 0.5 Max 1
Indirect tensile strength (kPa) @ 100% MDD			Min 250	Min 200

For C3 and C4 materials, i.e., with strengths less than 1.5 MPa, it is more important that the minimum tensile strength is achieved than the UCS. Experience has shown that problems associated with loss of stabilization are generally related to materials that do not achieve the specified ITS before carbonation occurs. It has been suggested that when the strains associated with any volume changes in stabilized materials exceed the tensile strength of the material, failure occurs.

A maximum limit is placed on all of the UCS values in Table 9. This was originally recommended to minimise cracking. In many instances, the maximum recommended

Importance of ITS

Experience has shown that problems associated with loss of stabilization are generally related to materials that do not achieve the specified ITS before carbonation occurs.

It is recommended that achieving the specified ITS should take precedence over exceeding the maximum UCS.

UCS has been exceeded and yet problems unrelated to cracking, such as weakening and carbonation of the top of the layer, have been noted. In these cases, the specified tensile strength was invariably not met. On this basis, it is recommended that achieving the specified ITS should take precedence over exceeding the maximum UCS.

Investigation of numerous stabilized materials has shown that the strength of stabilized materials (in terms of the UCS) is not synonymous with durability. Materials that easily achieve the required UCS can deteriorate in service. It is thus essential that adequate durability be achieved during the design phase. The conditions that the materials are likely to be exposed to in service should be simulated in the laboratory to give an indication of likely performance. Testing using accelerated carbonation, including the residual strength, and the wet/dry brushing test (Chapter 3: 5.3) should be carried out.

It has, however, been proposed that, to ensure adequate durability, the stabilizer content should exceed the ICS by at least one per cent. It is also suggested that the stabilizer content is further increased appropriately to account for variations in spreading, mixing and material properties, but this should be based on engineering judgement and the construction process to be followed.

For subbase layers beneath rigid pavements, erosion resistance as well as conventional durability is required. For these materials, UCS, ITS, wet/dry brushing and erosion tests (Chapter 3: 5.3) should be carried out. These tests should be done at different stabilizer contents to determine the optimum. ICS testing should also be performed to determine the lower limit for the stabilizer content.

6.2.3.3 Laboratory Testing of Stabilization Proposal

The final choice of type and amount of agent that will satisfy the design requirements can only be made after laboratory tests have been carried out on the material with the proposed type and quantity of stabilizer. This is part of the process of the stabilization design and permits the estimation of project costs and preparation of tender documents. However, once the project has commenced, the actual material to be used on site should be comprehensively tested using the proposed type and quantity of stabilizer.

It is often advisable to test with two different stabilizers, selected from those with the optimum results in the pre-design testing, based on strength, durability and the effect of time and temperature on density and strength. The type of cement selected needs to be available, and economical, for the project. Laboratory tests using two or three different stabilizer quantities should also be carried out to confirm the decision. Large relative differences between different materials and their reaction to stabilizers may be found. Stabilizers should therefore not be substituted without repeating the necessary tests. Through the testing of the stabilizer-soil mixture, problem soils are identified before they are used on the road.

Once the best stabilizer has been identified, it is important to carry out testing to determine the optimum working time at various temperatures likely to be encountered on site. The method is discussed in Chapter 3: 5.3.1(viii).

6.2.3.4 Densities of Lime and Cement

The relative densities of lime and cement differ considerably with averages of about 2.35 and 3.14, respectively, and bulk densities of about 640 – 720 and 1520 kg/m^3, respectively. This has a significant bearing on stabilization design and testing where laboratory investigations typically use mass for calculating stabilization quantities. In practice the volume of 1 ton of cement is about 40% of that of 1 ton of lime, which needs to be taken into account when working with these two materials in terms of specifying stabilizer contents, spread rates on site, and the amount added during laboratory testing. A consequence is that it is much easier to mix in lower percentages (< 2.5%) of lime than the same mass of cement to ensure acceptable mixing/distribution of the stabilizer.

6.2.4 Re-Stabilization

Problems have arisen when the construction of stabilized layers goes wrong and a layer is rejected. There are many reasons for possible rejection, such as, densities or strengths are not met or laminations are observed. It is normal practice to rework the layer with the addition of half of the original quantity of stabilizer. If the layer is rejected a second time, it should not be re-stabilized, but removed to spoil and new material brought in for stabilization. Questions are still being asked about recycling existing stabilized materials. The stabilization design should identify the optimum stabilizer content using the recycled material. It should be noted that if the pH of this material is above about 10, there is a high potential for self-cementation and unexpectedly high strengths could be obtained with time. Testing for this phenomenon should be carried out.

If recycled materials are re-stabilized and then rejected as a result of poor workmanship, extreme caution should be taken when reworking them with additional stabilizer, as a similar problem to that described above is likely.

6.3 Non-Traditional Stabilizers

The use of non-traditional chemical soil stabilizers is far more complicated than the use of conventional chemical stabilizers. There is a wide range of these products, all based on different chemical processes, and following different reaction mechanisms. These stabilizers are discussed in detail in Chapters 3: 6 and Chapter 4: 6.2.

6.3.1 Fines Retention

The use of chemicals for fines retention does not require a specific design. The application rate is usually based on recommendations of the supplier, in conjunction with the product performance guarantee (PPG). As these products are only sprayed on the road surface, any change in application rate or re-application can be carried out without significant disturbance to the road. All of these products require periodic rejuvenation, and the cost and implications thereof need to be considered.

6.3.1.1 Stabilization Design

To carry out a soil stabilization design using proprietary chemicals, it is essential that the mode of stabilization is first identified. This may include chemical reactions at the micro-level, e.g., cation exchange or alteration of the double layer water, or at the macro-level, e.g., conventional cement hydration, or purely a cementing or gluing stabilization. Only once this has been ascertained, can the stabilization design be effectively done.

(i) Chemical Stabilization

The basic principles of stabilization design remain the same for chemical stabilizers such as ionic, enzyme or cement-based. The required strength is identified in terms of a G-class or actual CBR or UCS, and the material should be treated to ensure that this property is obtained cost effectively, and can be retained through the life of the pavement.

Experience has shown that the ionic and enzyme stabilizers can be highly material and concentration sensitive. It is thus essential that a number of samples from the material source are obtained and tested. All of the samples should give similar results. If there are large discrepancies in the results obtained, or behaviour of the materials, such variations can be expected on site and an alternative product or treatment should be sought. A range of application rates (concentrations) must be tested to determine the best. Typically the rates are about 0.03 ℓ of chemical/m^2 for stabilization. Lower rates are employed only when improved compaction is required, or low PI materials are used.

The stabilization design requires that a number of CBR moulds, preferably using a range of chemical products and 2 or 3 application rates (0.02 to 0.04 ℓ/m^2) are compacted at the design density. The CBRs of these moulds should be determined in the normal way and the results compared with the required design property.

In some cases, the chemical product supplier may specify that a certain curing regime be followed prior to soaking and testing. This should be followed, but it must be borne in mind that this procedure will also be necessary in the field, which, depending on the process and duration, may make the procedure impractical on a full scale project.

If the stabilization design procedure shows that the treated material achieves the design strengths required, the chemical can be used. Although there is little substantiated proof, the enzyme and ionic stabilizers should remain durable indefinitely. Cement or lime based materials are subject to the normal durability requirements for a chemically treated material as specified in the Standard Specifications or the project specifications.

(ii) Mechanical Stabilization

Although a number of products are marketed as providing mechanical bonding or gluing, the effect on the materials is effectively the same. There is no chemical or physical change to the material, other than the particles being glued together. The quantity of product added depends on its concentration. Most products are emulsified, and the product cost can be reduced by increasing the volume of the emulsion medium (usually water) and the grading of the material being treated. For effective gluing, the individual particle surfaces should be coated with the product so that all contact points between the individual particles stick together. Obviously, the higher the fines content, and thus the particle surface area, the more product required for coating.

The strength of the treated material is a function of the friction angle of the coated particles as well as the shear strength of the gluing medium. It is thus not always a good idea to increase the “binder” content as excessive product may in fact reduce the strength.

Once the emulsions have “broken” and the products have been deposited on the material, they are generally insoluble in water and should prove to be durable under most circumstances. It has been noted that some of the acrylate emulsions are not UV resistant and decompose with time when exposed to sunlight. This should not be a problem when they are used in layers in sealed roads.

6.3.2 Cost Effectiveness

It is essential that the cost-effectiveness of non-traditional stabilized materials is assessed before their use. Many of the products are very expensive. It has been shown both locally and internationally that it may be more cost-effective to provide a chip seal than to treat unsealed roads, or to use conventional materials and/or stabilizers in sealed roads.

A good understanding of the overall costs, including product cost, specialist equipment and complex curing regimes, is necessary before making use of these products. The cost of different products is often difficult to compare as application rates and product concentrations vary considerably. Only once the products are tested in the laboratory and the optimum application rates identified, can realistic cost comparisons be made. It is very difficult to determine life cycle costs as there is very little information on the long-term performance of most of the chemicals available.

6.3.3 General

The use of proprietary chemical products requires a good understanding of their properties, stabilization action, construction requirements and long-term properties. Many of these have not been fully quantified and good engineering understanding and judgement should be employed when considering these products for use. There is no doubt that they can provide cost-effective solutions in certain cases and can make local marginal quality materials feasible for use. However, the following questions must be clearly answered:

- Is the stabilizer **readily available** and **effective**?
- Is the stabilizer **safe** to use?
- Can the stabilizer be **effectively distributed** through the materials?
- Does the stabilized material require **curing** or other **time consuming processes**?
- Are **special contract conditions** available or in place?
- Has the stabilizer **stood the test of time** on other projects?

7. PRIMES, STONE PRECOATING FLUIDS AND TACK COATS

This section addresses the selection and utilisation of primes, stone precoating fluids and tack coats in the construction and maintenance of roads. It is closely related to:

- Chapter 3: 4.5 which deals with testing of these products
- Chapter 4: 4.5 which covers the standards that are applied to these materials
- Chapter 12: 3.9 in which construction processes are covered

Detailed information on the selection and use of primes and stone precoating fluids can be found in Sabita Manual 26 as well as in the Standard Specifications, while there is reference to the use of tack coats in Sabita Manuals 5, 22 and 27.

7.1 Primes

7.1.1 Definition and Functions of a Prime

A prime consists of a bituminous binder applied to a non-bituminous granular pavement layer as a preliminary treatment prior to the application of a bituminous base or surfacing, as illustrated in Figure 24. The main function of a prime is to penetrate the layer to which it is applied, while leaving a small residual amount of binder on the surface to:

- Promote **adhesion** between the base and the newly applied bituminous base or surfacing.
- Inhibit the **ingress of rainwater** into the base, while not hampering the evaporation of water vapour from the base layer.
- Limit the **absorption of binder** into the base from the next spray operation.
- **Bind the finer particles** on the surface of the base to **accommodate light traffic** for a short period until the new surfacing can be placed.

Prime

A prime is used as a preliminary treatment on a granular layer prior to application of an asphalt layer. The idea is to promote adhesion between the two layers.

Figure 24. Application of a Prime

Priming is a normal part of the road construction process whenever road pavements with granular type bases are constructed. There are, however, some very specific cases where priming the base may not be necessary, such as:

- The surface of the base has been **enriched with bitumen emulsion** or the base is a **BSM-foam**.
- Spraying a semi-priming binder for the **construction of a graded seal,** such as an Otta Seal.

In individual cases where the prime is omitted, the construction of the seal should proceed without delay and the first spray application should be increased by ± 0.15 ℓ/m^2 to allow for some absorption of the binder into the base.

7.1.2 Requirements of a Prime

Low viscosity cutback bitumen or inverted bitumen emulsions, which are able to penetrate the granular base layer, are suitable as primes. Ordinary oil-in-water type bitumen emulsions, such as stablemix and spray grade emulsions are not suitable as primes due to their inability to penetrate dense base materials. On penetration of these emulsions, the bitumen droplets coalesce to form a skin on the surface as the emulsion breaks.

Primes are formulated so that, once sprayed, they dry within a reasonably short period. This enables the construction of the next layer to proceed without undue delay and without pick-up by the tyres of the construction plant. They are formulated to enable penetration at ambient temperatures without requiring heating to reduce their viscosities. Heating may only be necessary for spraying purposes

7.1.3 Types of Primes

7.1.3.1 Standard Products

Standard products include:

- **MC–30 or MC–70** cutback bitumen grades complying with SANS 4001-BT2
- **Inverted bitumen emulsion** complying with SANS 4001-BT5

See Chapter 3: 4.5.1.

7.1.3.2 Proprietary Products

Should primes be used that do not comply with SANS specifications, the supplier should provide specifications against which the product has been tested for compliance. These specifications should however meet the following SANS requirements for the distillation test:

- Minimum **residue from distillation** of 50% of the total volume
- **Penetration** at 25 °C of the residue between 90 and 180 dmm

Bitumen emulsion based primes, known as "Eco-primes", have been developed, and are more environmentally friendly than the cutback primes, with solvent contents around 50% less than those used in MC–30.

7.1.4 Selection of Primes

Several factors should be taken into account when selecting the most appropriate type of prime for a particular project. The main selection criteria are shown in Table 10.

Coal Tar Based Primes

The use of coal tar based primes is prohibited as these products introduce carcinogenic hazards which are injurious to health and pose serious contamination threats to the environment.

Rainfall on a Prime

A prime cannot be assumed to be an impermeable membrane and that it will prevent any ingress of moisture during rainy weather. Even though the prime is applied when the moisture content of the base is less than 50% of OMC, its **moisture content must always be rechecked** before surfacing if there has been rain subsequent to the original testing.

Table 10. Prime Selection Guidelines

Type of Base	MC–30	MC– 70	Inverted Emulsion
Weathered graded natural gravel, e.g., G4 to G5	1	2	1
Unweathered crushed stone, e.g., G1 of G2	2[1]	NS	1
Lime or cement stabilized [2]	1	2	1
Bitumen stabilized	NS	NS	2
Calcrete	1	2	1
Materials containing soluble salts	NS	2	NS
Absorptive Properties of Base Material			
High moisture content	NS	NS	NS
Low moisture content	1	2	1
High degree of densification	NS	NS	2
Low degree of densification	1	2	1
High porosity	2	1	2
Low porosity	NS	NS	2
Plasticity Index > 7	NS	NS	NS
Plasticity Index < 7	1	1	1
Open graded	2	1	2
Climatic Conditions			
High humidity	1	2	NS
Wet	NS	NS	2
Road temperature > 25 °C	1	1	2
Road temperature < 25 °C	2	NS	1

Ratings:
1 = first choice
2 = second choice
NS = not suitable

Notes:
1. Viscosity can be reduced by cutting back with illuminating paraffin
2. Primes are not suitable to act as curing membranes to prevent the loss of moisture and to reduce carbonation of cementitious stabilized layers. Normal bitumen emulsions should be used for this purpose.

Moisture in Base before Priming

The moisture content of the material in the base should not be more than 50% of its optimum moisture content before priming.

7.1.5 Moisture and Weather Limitations

Priming should only be carried out when the base has dried out sufficiently to not trap excess moisture under the primed surface. High moisture contents in the base could result in the early distress of this layer. The moisture content of the material in the base should not be more than 50% of its optimum moisture content. Priming should not be carried out when:

- Weather is **foggy or wet**
- **Rainfall** is imminent
- Surface of the **base is visibly wet**
- **High wind speeds** will cause uneven spraying
- Surface temperature immediately prior to spraying is **below 10 °C**

7.1.6 Preparing the Base

The base is broomed to remove any loose material, with the sweeping carried out carefully so as not to damage the surface of the layer. A light, uniform spray of water is applied to dampen the surface. Any excess moisture that results in saturation and ponding on the surface should be allowed to evaporate before the prime is applied.

7.1.7 Base Materials

Materials containing soluble salts are prone to problems with the adhesion of bituminous products. To reduce this risk, bases constructed using material containing soluble salts should not be dampened before priming, as this can cause the dissolved salts to migrate to the surface and re-crystallise during the curing of the prime.

Bases with Soluble Salts

Special precautions are required when priming bases that contain soluble salts.

The base should be primed immediately after completion, and should be surfaced within 24 hours using either penetration grade or modified bitumen.

7.1.8 Preparation and Spraying

Cutback and inverted emulsion primes can be pumped at ambient temperatures and thus do not need to be heated during loading, transport, and offloading into storage. However, to reduce the viscosity of these products so as to produce a proper fan when they are sprayed through the nozzles of the distributor's spray bar, heating is required. Details of maximum storage temperatures and spray temperatures for primes are given in Table 11.

Table 11. Storage and Spray Temperatures for Primes

Prime Type	Maximum Storage Temperature (°C)		Spray Temperature Range (°C)
	Up to 24 hours	Over 24 hours	
MC–30	65	30	45 – 60
MC–70	80	50	60 – 80
Inverted bitumen emulsion	–	–	20 – 60

When it is necessary to reduce the viscosity of either cutback or inverted bitumen emulsion primes, this is done by blending with illuminating paraffin. Paraffin has a flash point of 40 ^{o}C, therefore where it is used as a cutter, great care should be exercised not to exceed this temperature. Blending should be carried out in a proper blending facility, where proper health and safety precautions can be implemented, and not on site.

The heating of the cutback primes should be carried out as soon as practical before spraying, to reduce the loss of the volatile fractions.

7.1.9 Application Rates

The application rates for primes depend on the type of material used in the base as well as the surface texture of the base. Typical prime application rates for natural gravel bases are between 0.6 ℓ/m^2 and 1.0 ℓ/m^2 and for crushed stone bases between 0.6 ℓ/m^2 and 0.8 ℓ/m^2. On larger jobsites, consideration should be given to optimising prime spray rates by carrying out trials along short sections using different application rates.

7.1.10 Drying Times

The time taken for primes to dry varies according to the prevailing weather conditions (temperature and humidity), as well as the porosity of the base layer. Before proceeding with the construction of the next layer the prime should be allowed to dry. Any pooling of prime on the surface should be blinded with crusher dust and removed.

In cases where it is not possible to divert the traffic, the prime should be allowed to penetrate as long as possible before it is blinded with crusher dust or coarse river sand. Any caking of the aggregate that could cause problems, should be removed before the final surfacing is undertaken.

7.2 Stone Precoating Fluids

7.2.1 Purpose and Functions of Stone Precoating Fluids

Aggregates used in spray seals are sometimes precoated to reduce the risk of poor adhesion, the possibility of early chip loss and stripping.

Precoating fluids consist of low viscosity bitumen based products containing petroleum cutters and a chemical adhesion agent, their purpose being to precoat surfacing aggregates to improve the adhesion of the aggregate to the bituminous binder.

The precoating fluid assists in reducing the surface tension between the cold surfacing aggregate and the freshly sprayed hot, viscous binder during the construction of the seal, thereby enhancing the initial bond between the aggregate and the binder as the binder cools down.

Precoating

Aggregates used in spray seals are sometimes precoated to reduce the risk of poor adhesion, the possibility of early chip loss and stripping.

7.2.2 Precoating Fluid Requirements

The viscosity of the precoating fluid should be such that the fluid:

- **Coats** damp and dusty surfacing aggregates.
- **Dries within a reasonable period**.
- Does not leave a **non-tacky residual film** on the surface of the aggregate particles.

Any deposit of the precoating fluid left on site after the completion of the precoating process should not be harmful to the environment.

The precoating fluid should contain at least 0.5% of a chemical adhesion agent to:

- Enhance the adhesion of the aggregate to the binder in the **presence of moisture.**
- Improve the adhesion of aggregates that have a **poor affinity for bitumen**, for example, quartzite and other siliceous aggregates.

Coal Tar Based Precoating Fluids

The use of coal tar based precoating fluids is prohibited. These products introduce carcinogenic hazards which are injurious to health and pose serious contamination threats to the environment.

Typical specifications for precoating fluids are given in Chapter 4, Standards: 4.5.2.

7.2.3 Factors Influencing the Need to Precoat

When to precoat aggregate essentially depends on the type of aggregate, the binder, and the seal type. Guidelines are given in Table 12.

Table 12. Guidelines for When to Precoat Surfacing Aggregates

Type		Recommendation
Seal Type	Single seal	Essential, but not required if bitumen emulsion is used as the tack spray.
	Multiple seal	Optional, but not required if diluted emulsion is used as a cover spray.
	Cape seal	Optional, but not required if diluted emulsion is used as a cover spray.
	Graded seal	Not required if binder is a cutback or if an adhesion agent is added to the binder.
Binder Type	Penetration grade bitumen	Essential
	Cutback bitumen	Optional
	Bitumen emulsion	Not required
	Hot polymer modified bitumen	Essential
	Bitumen rubber	Essential
Type of Aggregates	Hydrophilic	Essential
	Dust content > 2%	Essential
	Porous	Essential
	Quartzitic	Essential
	Granite	Essential

Other factors that influence the precoating of surfacing aggregates are:

- There is a need to **create a dark surface** to make line marking more visible, especially when light coloured aggregates are used.
- Lower than normal **construction temperatures** are expected. The risk of **stripping** during the early life of the seal is improved as the precoating fluid promotes adhesion between the binder and the aggregate
- **Design traffic and/or vehicle speed is high**. The risk of stripping can be reduced as the precoating fluid enhances the adhesion between the binder and the aggregate.
- **Tight curves or frequent sharp vehicle turning movements** that require good adhesion of the surfacing stone.

7.2.4 Precoating the Aggregate

Mixing is carried out either using a front-end loader or a mixing plant. The following items should be noted when precoating aggregates:

- Stone should be **slightly damp** but not saturated.
- **Thorough mixing** is required to ensure uniform coating.
- **Stockpiled** precoated aggregate should be **protected from rain** to prevent the binder washing off.
- Precoated aggregate should be allowed to **dry for at least 4 days** before use.

The precoated aggregates have an unlimited stockpile life, as long as they are protected from dust and rain.

7.2.4.1 Application Rates

Application rates for precoating fluids should be adjusted so as to ensure the uniform coating of each aggregate particle. Application rates are affected by:

- **Stone size**
- Whether the stone is **clean or dusty**
- Surface **texture** and **porosity** of the stone

Table 13 gives typical application rates. However, it is recommended that trials be carried out by precoating small qualities of the aggregate using different amounts of precoating fluid to optimise the application rate.

Table 13. Typical Precoating Fluid Application Rates

Stone Size (mm)	7.1	10	14	20
Application Rate (ℓ/m^3)	13 – 18	12 – 17	11 – 16	10 – 15

7.3 Tack Coats

7.3.1 Purpose and Function of Tack Coats

A tack coat is a bituminous product that is applied either on top of a primed granular base or between layers of asphalt, its function is to promote adhesion. Tack coats are also used to enhance adhesion along transverse and longitudinal joints in asphalt layers.

While the prime serves to bind the surface of the granular base and affords protection against water and the scuffing effects of construction traffic for a short period, it does not provide adequate adhesion between the base and the asphalt layer placed on top. The tack coat serves this important purpose.

Tack coats invariably consist of bitumen emulsions and are normally applied to:

- Primed granular base
- Asphalt layers before paving another asphalt layer on top
- Transverse and longitudinal joints in asphalt layers

Tack Coats

Tack coats are a bituminous product that are applied either on top of a primed granular base or between layers of asphalt, its function is to promote adhesion. Tack coats are also used to enhance adhesion along transverse and longitudinal joints in asphalt layers.

In certain situations, a tack coat may be needed before applying a microsurfacing on an existing bituminous surfacing. A typical case is when the existing bituminous surfacing has been in service for some time and the surface has aged. In this case, the tack coat improves adhesion between the existing surfacing and the microsurfacing.

7.3.2 Tack Coat Constituents

Tack coats consist of anionic or cationic stable grade bitumen emulsion diluted 1:1 with water. These emulsions usually have a bitumen content of 60% and the dilution with water thus results in the tack coat with a bitumen content of 30%. The dilution of the bitumen emulsion is necessary to achieve full coverage of the surface with a thin residual bitumen film. Undiluted emulsion, with its higher bitumen content, results either in an uneven spray pattern or too high a binder content.

Use of Tack Coats

Due to the risk of poor adhesion, the use of a tack coat is always recommended, in all cases.

Some suppliers manufacture a **specially formulated emulsion** for use as a tack coat, which has reduced tackiness and pick up at ambient temperatures. There is currently an initiative to develop generic specifications for this type of product, which will allow it to be specified for use in a project.

Preparing the Surface for a Tack Coat

To ensure that the tack coat is effective, the surface should be free from any loose matter that could cause slippage. It must be thoroughly cleaned and all dust, grit and debris should be removed by mechanical and manual brooming. In some cases the use of compressed air, or even washing the surface with water, may be warranted.

7.3.3 Preparation before Applying the Tack Coat

The application of the tack coat should precede the asphalt paving operations to ensure a good bond between the existing pavement surface and the new layer of asphalt. When the bond between the layers is inadequate, slippage under the rollers during the compaction process may occur, causing the new asphalt layer to shove and deform. Loss of adhesion between the layers may manifest in time under traffic, especially along sections where vehicles accelerate or apply their brakes.

To ensure that the tack coating is effective, the surface should be free from any loose matter that could cause slippage. It must be thoroughly cleaned and all dust, grit and debris should be removed by mechanical and manual brooming. In some cases, the use of compressed air or even washing the surface with water may be warranted.

7.3.4 Application Rates

Typically residual bitumen application rates of 0.15 ℓ/m^2 to 0.25 ℓ/m^2 are used. Based on the 1:1 dilution rate and the 60% bitumen content of the emulsion, this is achieved by spraying the prepared tack coat at application rates between 0.5 and 0.8 ℓ/m^2.

The optimum spray rate depends on the condition of the surface. An open-textured surface requires a higher application rate to achieve uniform coverage than a tight, dense surface. A dry, aged surface requires a higher application rate than a surface that is rich and flushed.

Other instances where tack coat application rates should be adjusted are when:

- Applying a tack coat to **concrete bridge decks**, where very little absorption of the tack coat takes place. The application rate should be reduced to around 0.4 ℓ/m^2 of the dilute bitumen emulsion.
- **The asphalt has been milled**. The rough surface texture produced by the milling machine's cutting tools may require an increased application rate of up to 0.8 ℓ/m^2 of diluted emulsion tack coat.
- Applying a tack coat between layers of **freshly paved asphalt**. The net binder content of the tack coat should be reduced by up to 50%.

7.3.5 Applying the Tack Coat

Tack coats on the surface of primed granular base, or between layers of asphalt, should be applied so as to achieve a uniform application rate of adequate thickness to ensure proper bonding. The following key elements should be taken into account:

- A **certified binder distributor** should be used, and should be checked on site to ensure that all the nozzles are fully functional and are set at the correct angle and height above the road surface. See Chapter 12: 2.4 for more on binder distributors.
- In cases where **tack coating is carried out by hand**, precautions should be taken to ensure that a uniform spray pattern at the prescribed application rate is achieved. This requires focussed supervision, an experienced operator, as well as suitable and properly maintained equipment.
- The **timing of the tack coating operation** is important. It is good practice to spray tack coat well in advance of paving (but not more than about 18 hours, the Standard Specifications specify a maximum delay of 24 hours). This enables the diluted bitumen emulsion tack coat to break and "set up", which reduces pick-up of the tack coat on the tyres of supply trucks and the paver wheels.
- Tack coats should **not be exposed to general traffic**. The material tends to "pick-up" on tyres and carry over onto other areas leaving unsightly lines and could mark the paintwork of the vehicles. The surface has a low skid resistance which could cause accidents. In instances where it is not possible to accommodate the traffic off the tack coated surface, precautions should be taken to post a low, 60 kph speed limit along the section. In exceptional cases, the surface may need sanding.
- **Tacking of all transverse and longitudinal joints** in asphalt layers is crucial to prolong the service life of the asphalt layers. The full depth of the joints should be completely coated with the same dilute bitumen emulsion

tack coat material used on the surface, using a paint brush. An exception to this is when **open-graded surfacing mixes** are used, and the tacked joint tends to prevent water drainage across the pavement.

- Tacking of the joints is not necessary when the **asphalt layer butting against the joint** is paved on the same day and the mix has not yet cooled to below 50 ^{o}C.

7.3.6 Use of Spray Pavers

Some asphalt pavers are equipped with integrated spraying systems, which spray the tack coat at the prescribed application rate onto the surface just ahead of the paving screed. In particular, this type of paver is used to pave Ultra-Thin Friction Courses (UTFC), where a strong, impermeable bond between the underlying surface and the friction course is essential. Again, good preparation of the surface is important.

8. BITUMINOUS BINDERS

In broad terms, bitumen can be classified according to the processes followed in its manufacture from crude oil in refineries. The types of bitumen are shown in the flowchart in Figure 25.

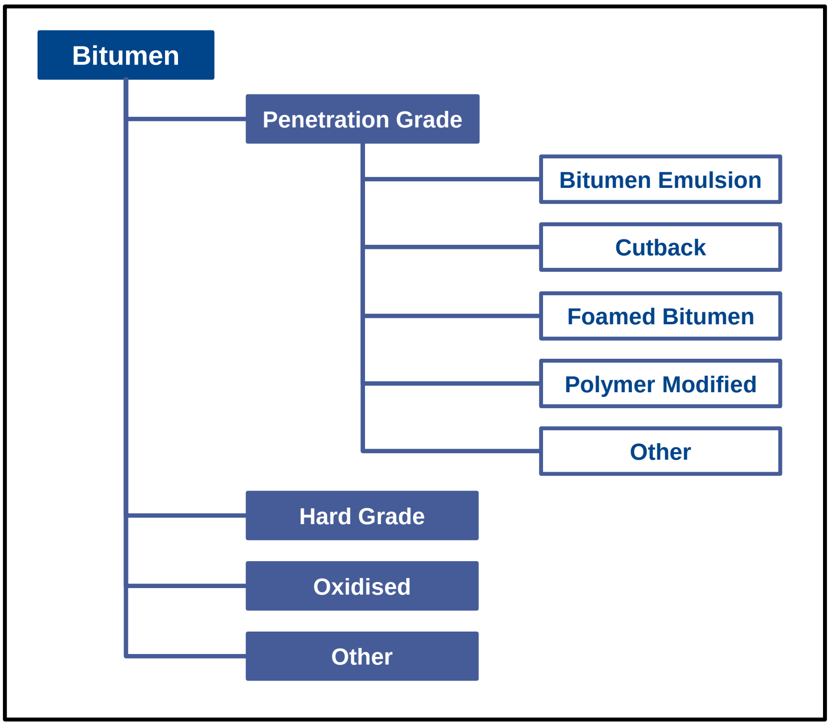


Figure 25. Uses of Bitumen

8.1 Types of Bitumen

In South Africa, the most commonly used bituminous binders include:

- Penetration grade bitumens for asphalt, seals or foamed bitumen
- Cutback bitumen
- Modified bitumen
- Bitumen emulsions
- Stone precoating fluids

8.1.1 Penetration Grade Bitumen

Penetration grade bitumen is manufactured either by straight-run distillation of crude oil or by blending two base components, one hard binder such as 35/50 penetration and a soft binder, such as, 150/200 penetration. Penetration grade bitumen is used either as a primary binder or base bitumen for the manufacture of cutback bitumen, modified binders or bitumen emulsions.

8.1.2 Cutback Bitumen

Cutback bitumen is a blend of penetration grade bitumen and petroleum solvents. The choice of solvent determines the rate at which the bitumen "sets up" or cures, when exposed to air. A rapid-curing (RC) solvent evaporates quicker than a medium-curing (MC) solvent. The viscosity of the cutback bitumen is determined by the proportion of solvent added; the higher the proportion of solvent, the lower the viscosity of the cutback.

When the solvent has evaporated, the binder reverts to the original penetration grade. The advantage of cutback bitumen is that, because

Reference for Binders

The manufacture, properties, specifications and test methods for handling, and application of bitumens are described in detail in Sabita Manual 2: **Bituminous Products for Road Construction**.

of the lower viscosities, it can be applied at lower temperatures than penetration grades and it penetrates into material better.

8.1.3 Modified Bitumen

The requirements imposed on asphalt layers and surface treatments in terms of resistance to rutting, fatigue, or adhesion, are reaching the limits of what can be achieved with conventional binders, and therefore modified binders are required. Bitumen modification is used quite commonly in many countries over the world, for more than 50 years. In Europe, the development of modified binders was specifically stimulated in countries where the clients, typically road authorities, required the contractor to give performance guarantees for several years. The same trend can currently be observed in South Africa. Developments in polymer modified binders have had significant impacts on the performance of asphalt.

Reference for Modified Binders

The Technical Guideline published by the Asphalt Academy titled "**The Use of Modified Bituminous Binders in Road Construction,** TG1 provides comprehensive guidelines for modified binders. It can be downloaded from www.asphaltacademy.co.za.

The increased interest in bitumen modification is attributed to the following factors:

- **Increased demand** on asphalt pavements in terms of traffic volumes and wheel loads. Specifically during very hot summers this is often associated with a premature rutting problem.
- Normal binders have difficulty in meeting the requirements in regions with **extreme climatic conditions,** or in special areas of application that **experience extreme loading conditions**, such as approaches to intersections, long and/or steep slopes, traffic circles or stationary traffic.
- In many cases, **regular maintenance** has not been done and there is a need to provide an economically viable solution to poor road conditions.
- **Environmental and economic considerations** to reuse old asphalt and some waste materials (e.g., tyres) as additives in bitumen and asphalt.
- Availability of **high performance binders.** Modification with polymers for asphalt layers and seal applications can improve the performance.
- Materials like **Porous Asphalt and Stone Mastic Asphalt** (SMA) perform well with modified binders.
- Public agencies are willing to pay a **higher initial cost for pavements with a longer service life,** i.e., less maintenance and road user delays. Life cycle cost analysis can be used to justify this approach.

The Asphalt Academy has published a very useful and comprehensive guideline for modified binders: TG1, "The Use of Modified Bituminous Binders in Road Construction".

8.1.3.1 When to Use Modified Binders

Modification is not a means to correct poor quality bitumen, but rather an enhancement of the engineering properties of acceptable bitumen. Modified binders must only be used with careful consideration. Improvements in mix behaviour through changes in the mix design should be attempted before introducing modified binders. When using a modifier, the effects must be carefully considered, as they may compromise some performance aspects of the mix. For example, some liquid anti-stripping agents reduce moisture damage, but have a tendency to soften the binder, thus compromising the rut resistance of the mix.

Modified Bitumens

Modification of bitumen is not a means to correct poor quality bitumen, but rather an enhancement of the engineering properties of acceptable bitumen.

Many questions should be when deciding to use modified binders, the most important are:

- What **improvement** is needed?
- How should the binder be **specified**, and how should routine test results be interpreted?
- How should the **modifier be added**?
- Is there a **compatibility** problem and how is storage a factor in this?
- What are the possibilities for **recycling**?
- What are the **Health, Safety and Environment** aspects (HSE)?
- Probably the most important question is: What is the effect on **life cycle costing**? Can the initial cost be justified as a benefit in terms of long-term performance and suitability?

Demand for Modified Binders

Certain factors influence pavement engineering requirements. For example, economic incentives and environmental considerations stimulate higher axle loads and the use of so-called "Super Single" tyres. The result is higher wheel loads, requiring high stability asphalt mixes. As a result of the onerous requirements, in a number of such cases, it is necessary to make use of modified binders with specific performance properties.

Modified binders always increase the initial construction cost. In countries where a low initial cost is the governing factor in awarding contracts, the use of modified binders is discouraged.

The requirements of binders can be summarised as follows:

- At **very high temperatures** for mixing, transport, paving, compaction, the consistency needs to be low enough to facilitate pumping, mixing and compaction.
- At **high service temperatures** during a hot summer, the consistency needs to be as high as possible, to reduce rutting.
- At **low service temperatures/aged conditions** the consistency needs to be lower with good relaxation behaviour to reduce thermal cracking.
- **Increased adhesion** between the binder and the aggregate in the presence of moisture to reduce stripping.

Common modes of failure of bituminous pavement layers are given in Table 14, and are discussed in Chapter 2: 5 and Chapter 14: 4.1. Based on failures, but also manufacturing aspects, a number of technical reasons for using additives or modifiers in asphalt are:

- Create **stiffer mixes at high service temperatures** to minimize rutting.
- Create **softer mixes at low service temperatures** to minimize non-load associated thermal cracking.
- Improve the **fatigue resistance** of mixes.
- Improve the bitumen-aggregate bonding to reduce **stripping or moisture susceptibility**.
- Improve of the **abrasion resistance** to reduce ravelling.
- Minimize **tender mix** problems during construction.
- **Rejuvenate the binder** in recycled asphalt.
- Reduce bitumen content by adding an **extender**.
- Thicker bitumen films on aggregate **for increased durability**.
- **Prevention of run-off** during transport and construction.
- Reduce **flushing or bleeding**.
- Improve resistance to **ageing or oxidation**.

Table 14. Common Modes of Failure of Bituminous Pavement Layers

Hot Mixes	Seals
• Rutting • Cracking (fatigue and thermal) • Loss of adhesion (stripping)	• Chippings loss • Bleeding and fatting up

8.1.3.2 Types and Classification of Modifiers

Different types of modifiers, and their functions, are given in Table 15. The list is not exhaustive and just summarises the common modifiers.

Safety Aspects of Bitumen

Bitumen presents a low order potential hazard, as long as sound and responsible practices are observed during the handling. These practices are covered in detail in Sabita Manual 8: **Guidelines for the Safe and Responsible Handling of Bituminous Binders** (2004).

Bitumen users should be aware that there is an obligation on the part of the supplier of bituminous binders to compile and issue Material Safety Data Sheets (MSDS) for each product in accordance with the regulations governing hazardous chemical substances. The MSDS is the primary source of information and advice on the safe handling of a specific product.

Common Modified Binders

Types of modifiers in general use are:

- Rubber crumb
- Styrene-butadiene-rubber (SBR) latex
- Styrene-butadiene-styrene (SBS)
- Ethylene-vinyl-acetate (EVA)
- Aliphatic synthetic wax
- Naturally occurring hydrocarbons

Table 15. Uses of Bitumen Additives and Modifiers

Types	Function	Examples
Fillers	• Supplement aggregate fraction in mastic	• Mineral fillers: crusher fines, lime, portland cement, fly ash • Carbon black
Fibres	• Improving the tensile strength and cohesion in hot mix asphalt • Allows higher binder content with reduced risk of drain-down in open-graded asphalt and SMA • Improves durability through increased binder film thickness	• Natural: asbestos • Man-made: polypropylene, polyester, fiberglass, mineral, cellulose
Extenders	• Substitutes a portion of bitumen to decrease the amount of bitumen and/or polymer required • Improves the storage stability of SBS modified binders	• Sulphur • Lignin
Polymers	• Improves resistance to rutting (plastomer) or fatigue (elastomer)	• Thermoplastic: PE, PP, EVA • Thermosetting: Epoxy • Elastomer: Natural latex, synthetic latex (SBR), block copolymer (SBS, SIS), reclaimed rubber (crumb rubber modifier)
Hydrocarbons	• Extends the plasticity range of bitumen	• Recycling and rejuvenating oils • Hard and natural bitumen: Gilsonite, TLA • Long-chain hydrocarbons: i.e., high molecular weight wax produced in the Fischer-Tropsch synthesis process, known as FT wax
Surface active agents	• Reduces stripping of binder from aggregate	• Emulsifiers • Anti-stripping agents: Amines, lime
Waste material		• Roofing shingles • Recycling tyres
Miscellaneous	• Increases the durability of HMA by retarding oxidation	• Oxidants • Antioxidant • Silicones
Fuel resistance	• Improves the resistance of the HMA to fuel spillages	• FT Wax • Selected grades of EVA
Cut-backs	• Cut back penetration grade bitumen for constructing surfacing seals in South Africa to widen the window of acceptable binder viscosity	• Power paraffin

Modified binders are classified in TG1 as follows:

- **Application**
  - S: seal
  - A: asphalt and
  - C: crack sealant
- **Binder system**
  - If an emulsion is cold-applied, the letter C follows immediately after the letter indicating the type of application.
  - If the binder is hot-applied, no letter is used after the application indicator.
- **Modifier**
  - E: elastomer
  - P: plastomer
  - R: rubber crumb
  - H: hydrocarbon
- **Level of modification.** A numerical value which increases in relation to softening point values, but which is not necessarily indicative of improved overall performance characteristics.
- **Use of flux:** Should the binder application no permit the use of flux or cutter, the letter "t" should be shown in brackets after the classification.

Use of Polymers

Polymers are increasingly being used to modify bituminous binders, as the benefits become more evident.

For example, a classification of SC-E2(t) indicates that the binder is:

- S: Intended for a surfacing seal
- C: Is an emulsion
- E: The main modifier is an elastomer
- 2: The softening point is higher than an SC-E1
- (t): The use of a fluxing agent or cutter is prohibited

It is necessary that specialist pavement engineers have a sound knowledge of the range of modifiers available and their main application purpose.

8.1.4 Bitumen Emulsion

Bitumen emulsions are two-phase systems consisting of a dispersion of bitumen droplets in water containing an emulsifier. The proportion of bitumen:water is typically 60:40 or 65:35 by mass. The production of bitumen emulsion is illustrated in Figure 26. Emulsification of bitumen is a means of reducing the viscosity of a binder so it can coat the aggregate like paint during application. The common bitumen emulsion compositions are:

- **Cationic** emulsions, which have positively charged bitumen globules. Cationic emulsions break in a physical-chemical reaction, through the evaporation of the water phase and through mechanical actions, such as rolling. Three-quarters of the earth's crust comprises siliceous materials with negative charges that have an affinity for cationic emulsions. This includes acidic aggregates such as granite and quartzite, which are negatively charged and provide good adhesion to the positively charged bitumen in a cationic emulsion.
- **Anionic** emulsions, which have negatively charged bitumen globules. Anionic emulsions break predominantly when the bitumen particles agglomerate with the evaporation of the water and through mechanical action such as rolling.
- **Inverted** emulsions, which are distinct from normal oil in water cationic and anionic emulsions. With inverted emulsions, the water is dispersed in the binder phase. These emulsions are manufactured with cutback bitumens and have less than 20% water.
- **Non-ionic** emulsions, without charged bitumen globules. These emulsions do not have application in the roads industry, but are used in cosmetics.

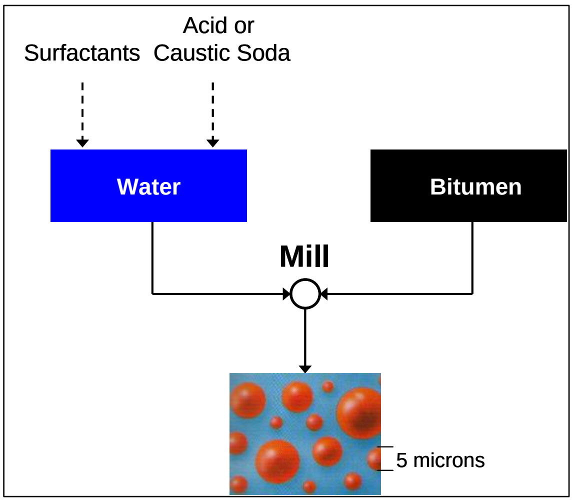


Figure 26. Manufacture of Bitumen Emulsion

Emulsions are available in the following grades, defined by the stability when in contact with aggregates:

- **Spray grade (rapid set, RS)**: These are the least stable emulsion as they include low emulsifier contents, resulting in weak repulsion between the bitumen globules. They are formulated for application by mechanical spray equipment for use in in chip seal construction, where no mixing with aggregate is required, or for tack coat applications.
- **Premix grade (medium set, MS)**: An emulsion formulated to be more stable than spray grade emulsion and suitable for mixing with medium or coarse graded aggregate when the percent aggregate passing the 0.075 mm sieve does not exceed 2%. Medium set emulsions allow binders with cutter to be emulsified and used to create a cold surfacing mix. They can be stored for some time before application.

- **Quick setting grade (QS)**: An emulsion specially formulated for use with microsurfacing seal types, where quick setting of the mixture is desired.
- **Stable mix grade (slow set, SS)**: These are the most stable emulsions due to high emulsifier contents. This allows sufficient time to be mixed in thick layers before breaking and compaction occurs, hence they are used for Bitumen Stabilized Materials BSMs. The emulsion is also formulated for mixing with very fine aggregates, sand and crusher dust, and are also used for slow-setting slurry seals and tack coats.

Modified emulsions are also available for specialised applications. These are three phase cationic emulsion systems where SBR latex is introduced as a third component in the normal bitumen and water two phase system. SBS emulsions are popular elsewhere in the world, and will be available in South Africa soon.

Most of the world uses cationic emulsions in the rapid, medium and slow set applications, due to the dominance of chemistry rather than climate in the behaviour of the emulsion during application. South Africa is an exception, using **cationic** emulsion for rapid and medium set applications, but **anionic** emulsions for slow set applications (BSMs) for 2 reasons: economics and climate (typically warm ambient temperatures in SA). A third reason sometimes cited is that cationic slow set emulsions include concentrations of hydrochloric acid, which is very corrosive for pumps and tanks, whereas the pH of anionic emulsions is high (basic), so they don't share this problem.

Most emulsions used in road application are oil-in-water dispersions. When the bitumen content is increased in an emulsion, an inverted emulsion is created, where the water is in the dispersed phase and the bitumen in the continuous phase. MSP 3, used for rejuvenation of surfacings, is an example of such an emulsion.

8.1.5 Stone Precoating Fluids

A precoating fluid is a low viscosity bitumen-based binder containing petroleum cutters and a chemical adhesion agent. They are used to precoat surfacing aggregates to improve the adhesion of the aggregate to the bituminous binder. Stone precoating fluids are covered in more detail in Section 7.2.

8.2 Selection and Application

The selection of bituminous binders for specific applications is dictated by several factors. These include:

- The **materials** to be bound together.
- Prevailing **environmental conditions** of climate (both during construction and in service), topography and traffic loading.
- Position and function of the **layer**.
- **Costs**

There is no substitute for extensive experience and knowledge in the selection and application of an appropriate bituminous binder. The following comments only serve as a general guide.

An indication of typical applications of the various binders is given in Table 16. These applications should be viewed in conjunction with:

- **TRH3** for spray seals
- **TG1** for justification and selection criteria for the use of modified binders
- **Sabita Manual 26** for guidance on the selection of primes and precoating fluids

Table 16. Typical Applications of Bituminous Binders

Application	Binder Type
Binders for Priming	• MC-30 cutback bitumen • MC-70 cutback bitumen • Invert emulsion
Binders for spray seals	• 70/100 pen. bitumen • 150/200 pen. bitumen • MC-3 000 cutback bitumen • SBR hot modified bitumen S-E1 • SBS hot modified bitumen S-E1 • SBS hot modified bitumen S-E2 • SBR modified emulsion 65% SC-E1 • SBR modified emulsion 70% SC-E2 • Bitumen rubber S-R1 • Cationic spray grade emulsions (60%, 65% or 70%) • Anionic emulsion (60%)
Binders for stabilization	• Anionic or cationic stable mix emulsion (60%) • 150/200 pen. bitumen • 70/100 pen. bitumen
Binders for hot mix asphalt	• 50/70 pen. bitumen • 35/50 pen. bitumen • SBR modified bitumen A-E1 • SBS modified bitumen A-E1 • SBS modified bitumen A-E2 • EVA modified bitumen A-P1 • Hydrocarbon (FT Wax) • Natural hydrocarbon modified • Bitumen rubber
Crack sealants	• Hot applied elastomer modified bitumen C-E1 • Emulsion elastomer modified bitumen CC-E1 • Hot applied bitumen rubber C-R1

9. PAVEMENT LAYERS: BITUMEN STABILIZED

A bitumen stabilized material (BSM) is a pavement material treated with either bitumen emulsion or foamed bitumen. The production of bitumen emulsion is illustrated in Figure 26 and foamed bitumen in Figure 27. The bitumen in a BSM is dispersed selectively amongst the finer particles, leaving the coarser particles uncoated. The method of dispersion differs for bitumen emulsion and foamed bitumen:

- **Bitumen emulsion treatment**: Tiny charged bitumen droplets (< 5 µm), suspended in water, are attracted to the smallest material particles with the highest surface area and charge concentration. Bitumen emulsions are discussed further in Section 8.1.4.
- **Foamed bitumen treatment**: Bitumen bubbles "burst" into tiny splinters when they come in contact with the material. These splinters have only sufficient heat energy to warm and adhere to the smallest of material particles (< 0.075 mm at 15 °C).

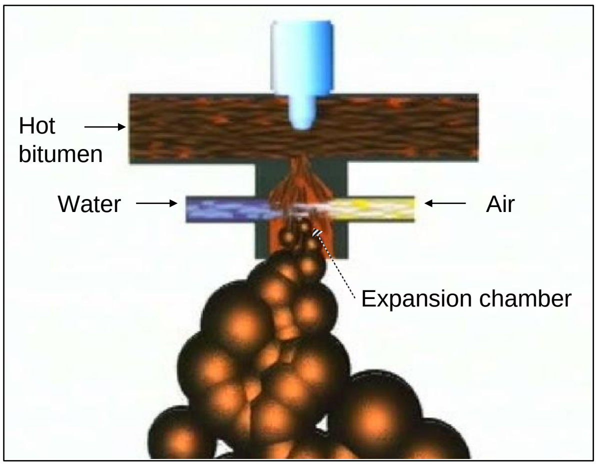


Figure 27. Foamed Bitumen Production in Expansion Chamber

This pattern of bitumen dispersion makes a BSM a "non-continuously bound material" when compacted to form a pavement layer. BSMs behave differently from all other pavement materials and should not be confused with continuously-bound materials such as cement stabilized material or asphalt. Their behaviour is more granular in nature but with significantly improved shear properties. The improved shear is a result of an increase in the cohesion due to the dispersed bitumen droplets (or splinters) forming localised bonds between material particles. The development of these bonds is different for bitumen emulsion and foamed bitumen:

- **Foamed bitumen:** The bonds develop when the material is mechanically compacted. Compaction energy presses the tiny bitumen particles (that are dispersed amongst the fines in the mastic of the material) against the adjacent material and, being a "sticky substance", the bitumen adheres, thereby forming localised bonds between the respective particles.
- **Bitumen emulsion:** The bonds develop when the emulsion breaks, leaving the bitumen adhering to the adjacent particles. This normally occurs after the material has been compacted. Since bitumen emulsion is attracted primarily to the finer fractions of the material, a bitumen-rich mastic is achieved.

There is little (if any) connection between the bitumen particles. The resulting bonds therefore remain isolated and hence the term "non-continuous" bonding. Furthermore, since the coarser particles are not coated with bitumen, the change in the angle of internal friction of the parent material is small.

As a consequence of this non-continuous bonding phenomenon, BSMs are less prone to fatigue cracking. The two fundamental failure mechanisms of BSMs are:

- **Permanent deformation.** This is the accumulation of shear deformation as a result of repeated loading and is dependent on the material's shear properties and densification achieved. Resistance to permanent deformation is enhanced by:

Nature of BSMs

BSMs are "non-continuously bound materials" when compacted in a pavement layer. They should not be confused with continuously-bound materials such as cement stabilized materials or asphalt. Their behaviour is more granular in nature but with significantly improved shear properties and durability.

- Improved angularity, shape, hardness and roughness of the parent material
- Increased maximum particle size
- Improved density through compaction
- Reduced moisture content
- Limiting the amount of bitumen added (usually less than 3%) because higher bitumen contents encourage instability
- Addition of active filler, limited to a maximum of 1% because higher active filler contents introduce brittleness that encourages shrinkage and traffic associated cracking

Reference for BSMs

The Technical Guideline published by the Asphalt Academy titled "**Bitumen Stabilised Materials, A Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials",** TG2 Second Edition, May 2009 provides comprehensive guidelines for the selection of suitable materials and the design procedures for bitumen stabilization. This publication is available for download from www.asphaltacademy.co.za.

- **Moisture Susceptibility**. The partially-coated nature of the material makes moisture susceptibility an important consideration when evaluating material performance. Moisture susceptibility is the damage caused by exposure of a BSM to high moisture contents and the pore pressures caused by traffic loads. This results in loss of adhesion between the bitumen and the material particles. Moisture resistance is enhanced by:
  - Increased bitumen content, bearing in mind the cost implications
  - Adding an active filler, limited to 1% by mass of dry aggregate
  - Increasing the density through compaction
  - Achieving a smooth continuous grading, which assists in achieving high levels of density

9.1 Background and Usage

The materials that are normally stabilized with bitumen are granular materials, previously cement treated materials and layers of reclaimed asphalt (RA). In addition to improving the shear strength, bitumen stabilization significantly reduces the moisture susceptibility of the material in the constructed layer. These benefits are, however, costly and BSMs are therefore best suited to upper pavement layers where stresses from applied loads are highest and moisture ingress due to surfacing defects are most likely to occur. Furthermore, road pavements constructed with BSMs are environmentally friendly, durable and cost effective. BSMs are suited to both construction of new pavements and to pavement rehabilitation, especially where in situ recyclers are used. A BSM-foam pavement is illustrated in Figure 28.

Figure 28. BSM-Foam Base Layer on Crushed Stone Subbase

Worldwide, the state of road pavements is deteriorating, and the demand for rehabilitating road pavements far exceeds the demand for new roads. Over the past twenty years, in place recycling has become increasingly popular as the preferred procedure for addressing the rehabilitation backlog by reusing material in the existing pavement. Where an existing pavement is recycled, old seals or asphalt surfacing is usually mixed with the underlying layer and treated to form a new base or subbase layer. Bitumen stabilization enhances the properties of these recycled materials, providing service lives that exceed those achievable had virgin aggregates been used, all at a lower cost and consuming far less energy in the process.

BSMs have mainly been used on pavement rehabilitation projects where the existing pavement structure is sound and balanced, and distress is confined to the upper layers. This scenario is often encountered when the surfacing layer has aged and cracked, allowing water to enter the pavement and cause moisture-activated distress in the underlying granular materials. Such pavements are ideal for in situ recycling and the bitumen is added to restore, and often improve, the structural integrity, before a thin asphalt or chip seal surfacing layer is applied.

Due mainly to escalating costs, a recent trend is to use good quality material such as recycled asphalt (RA), virgin graded crushed stone and/or blends of RA and virgin aggregate, stabilized with bitumen as a substitute for an asphalt base. The treatment process is generally carried out in a specialised plant that allows the input materials to be carefully controlled. The treated material can then be placed in stockpile (for limited periods) or trucked to the road where it is usually placed by paver.

Being able to hold a BSM in stockpile, especially when treated with foamed bitumen, makes it an ideal material for labour intensive construction.

9.2 BSM Technology

The Technical Guideline published by the Asphalt Academy entitled "Bitumen Stabilised Materials, A Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials, TG2 Second Edition, May 2009" provides comprehensive guidelines for the selection of suitable materials and the design procedures for bitumen stabilization. This publication is available as a free download under the Bitumen Stabilisation section of the Asphalt Academy website www.asphaltacademy.co.za. All references to "TG2" in this chapter are to the Second Edition (2009). The Second Edition of TG2 is an update for SABITA GEMS and ETB Manuals (1993 and 1999) and the 2002 version of TG2, which is no longer available.

For BSM technology, the differentiation between "stabilization" and "modification" has been dropped since the focus is on the behaviour of the final product rather than on the constituents that make up the product.

The design approach to BSMs is the same for both bitumen emulsion and foamed bitumen treatment. Due primarily to the difference in moisture (fluid) regimes, the procedures used to manufacture specimens in the laboratory are different for bitumen emulsion and foamed bitumen. However, the tests that are carried out on the specimens are the same, regardless of the stabilizing agent.

BSMs are classified into three classes, essentially depending on the shear properties of the treated material:

- **BSM1:** Materials with high shear strength
- **BSM2:** Materials with moderately high shear strength
- **BSM3:** Materials with moderate shear strength

The shear properties of the treated material (and hence the relevant BSM classification) are dictated by several variables, the main ones being the quality of the parent material, the amount and type of bitumen and active filler that is added, the effectiveness of the treatment process, as well as the density and moisture content of the material.

Stabilizing materials with foamed bitumen requires the use of specialised equipment whereas stabilizing with bitumen emulsion can be undertaken using conventional construction plant.

9.3 Components of BSMs

9.3.1 Parent Material

The quality of the parent material is the primary determinant of shear strength after treatment. The source of material is largely dictated by the type of project and treatment method:

- Rehabilitation by **in situ recycling** reuses material from existing pavements. The quality of the material encountered in the recycled horizon therefore dictates the quality of BSM achievable.
- Pavement **upgrading** (additional layers) and **new construction** normally calls for new materials, or those recovered from existing pavements and placed in temporary stockpiles. The quality of these new materials can be controlled, thereby ensuring that the required class of BSM is achieved. Treatment is usually undertaken in a

specialised mixing plant, i.e., not treated in situ. This allows better process control, particularly the quality and consistency of input materials.

The material requirements for the different BSM classes are described in detail in TG2. The main points are summarised below.

9.3.2 Bitumen Stabilizing Agent

Both BSM-emulsion and BSM-foam require the use of penetration grade bitumens. The following additional requirements are relevant:

- **Bitumen emulsion:**
  - Bitumens with penetrations of 70 to 100 are generally used.
  - Slow set stable grade anionic emulsions are generally used in South Africa, although cationic emulsions are used in the rest of the world.
  - The breaking rate of the emulsion is important, and should be tested with representative aggregate, and filler, and at water contents and temperatures applicable to the project.
  - The bitumen emulsion and aggregate must be compatible. Some aggregates, such as quartzite, granite, sandstone, rhyolite, syenite and felsite may not be compatible with anionic emulsions.
- **Foamed bitumen:**
  - Bitumens with penetrations of 70 to 100 are generally used. Harder bitumens are avoided due to the poorer quality foam produced.
  - The expansion ratio and a half-life of the bitumen must exceed minimum values, which depend on the aggregate temperature. These properties determine the quality of foam produced.
  - The temperature of the bitumen and the injection of water to create the foam have significant effects on the quality of foam produced.

The quantity of residual bitumen from emulsion or foamed bitumen that is added to achieve a stabilized material is usually less than 3% (by mass) of dry material.

Foaming Characteristics

When foaming bitumen, the following foaming characteristics are important:

- The **expansion ratio** is the maximum volume of foam relative to the original volume.
- The **half-life** is the time the foam takes to collapse to half its maximum volume.

9.3.3 Active Filler

The most common active fillers used with BSMs are hydrated lime and cement. When cement is used, the cement content should not exceed 1%. The amount of hydrated lime added is also normally less than 1%, except where the parent material is relatively plastic (PI > 10). For such materials, the Initial Consumption of Lime (ICL) is normally adopted as the application rate and the lime is often added as a pre-treatment before adding the bitumen stabilizing agent (usually after a minimum delay of 4 hours).

Active filler is used in BSMs for the following reasons:

- **Supplement the** fines needed for bitumen dispersion
- Improve **adhesion** of the bitumen to the aggregate
- Improve **dispersion** of the bitumen in the mix
- Modify the **plasticity** of the natural materials (reduce PI)
- Increase the **stiffness** of the mix and rate of strength gain
- **Accelerate curing** of the compacted mix
- Control the **breaking time** of BSM-emulsion
- Improve the **workability** of BSM-emulsion
- Assist in the **dispersion** of foamed bitumen droplets

Note that the application rate of cement should never exceed that for bitumen (residual in the case of emulsion). Stated differently, the ratio of bitumen percentage to cement percentage must always be greater than 1. Where this ratio is less than one, the material should be considered a cement treated material and the applicable guidelines given in this document should then be followed. In addition, the merits of using such a mix should be queried since the benefits of adding bitumen will be suppressed by the continuous bonding of the cement.
The appropriate quantity and type of active filler is best determined by testing during mix design.

9.3.4 Moisture Content

The amount of moisture present in the parent material at the time of mixing plays a significant role in determining the quality of the product.

- **BSM-foam**. When stabilizing with foamed bitumen, the moisture content must approximate the "fluff point" of the material (between 70% and 80% of OMC) for effective mixing.
- **BSM-emulsion**. When stabilizing with bitumen emulsion, the moisture content must be sufficiently low to accommodate the amount of added fluid (emulsion) without exceeding the saturation limit, thereby allowing the material to be compacted to the required level of density.

9.4 BSM Mix Design

Three levels of mix design testing are used to determine an appropriate mix for a particular BSM. Levels 1 and 2 involve Indirect Tensile Strength (ITS) tests whilst Level 3 calls for triaxial tests. All tests are carried out at 25 °C. The level of testing required for a particular project is dictated by the design traffic (structural capacity requirements):

- **Level 1:** low volume roads with design traffic < 3 million standard axles (MESA)
- **Level 2:** medium volume roads with design traffic > 3 MESA
- **Level 3:** higher volume roads with design traffic > 6 MESA

A flow chart of the mix design procedure for all three mix design levels is given in Figure 29.

The preparation of BSM mixes in the laboratory is carried out using a pugmill mixer. BSM-foam mixes require a specialised laboratory unit to foam the bitumen prior to injecting into the pugmill mixer. Such specialised laboratory equipment must be capable of simulating field conditions.

9.4.1 Level 1 Mix Design

All mix designs start with Level 1, which uses ITS tests on 100 mm diameter specimens (63.5 mm in height) to indicate the optimum application of bitumen, the need for an active filler and which active filler (cement or lime) is more appropriate. Specimens are manufactured using either vibrating hammer or Marshall drop-hammer compaction, and cured in an oven at 40 °C until a constant mass is achieved (normally 72 hours). Half the specimens are then soaked in water for 24 hours before testing to determine the ITS_{WET} value. The ITS_{DRY} value is obtained from the unsoaked specimens.

9.4.2 Level 2 Mix Design

Level 2 uses ITS test results from 150 mm diameter, 95 mm high specimens to fine-tune the optimum application rate for both bitumen and active filler. Specimens are manufactured using vibrating hammer compaction and cured to approximate the equilibrium moisture content (± 50% of OMC). This is achieved by placing the specimens in an oven unsealed for 20 hours (BSM-foam) or 26 hours (BSM-emulsion) at 30 °C. Each specimen is then sealed in a loose-fitting plastic bag and returned to the oven for 48 hours at 40 °C. Half the specimens are then soaked in water for 24 hours before testing to determine the ITS_{SOAK} value. The other half of the specimens are tested without soaking to determine the ITS_{EQUIL} value. All tests are carried out at 25 °C. "EQUIL" refers to the expected equilibrium moisture content in the pavement layer.

To select the most appropriate bitumen application rate, four or five sets of 150 mm diameter specimens are usually prepared with bitumen application rates varying by 0.1% intervals on either side of the optimum indicated by the Level 1 ITS tests. The ITS_{EQUIL} and ITS_{SOAK} values obtained from these specimens are then plotted on a graph and used to indicate the minimum amount of bitumen addition required to meet the required BSM classification. This is illustrated in Figure 30 where the bitumen content would be that determined by the ITS_{SOAK}. The amount of active filler can also be fine-tuned in the same manner by manufacturing additional specimens using this bitumen application rate, but with various application rates of active filler, normally in the range of 0.5% to 1.0% at 0.1% intervals.

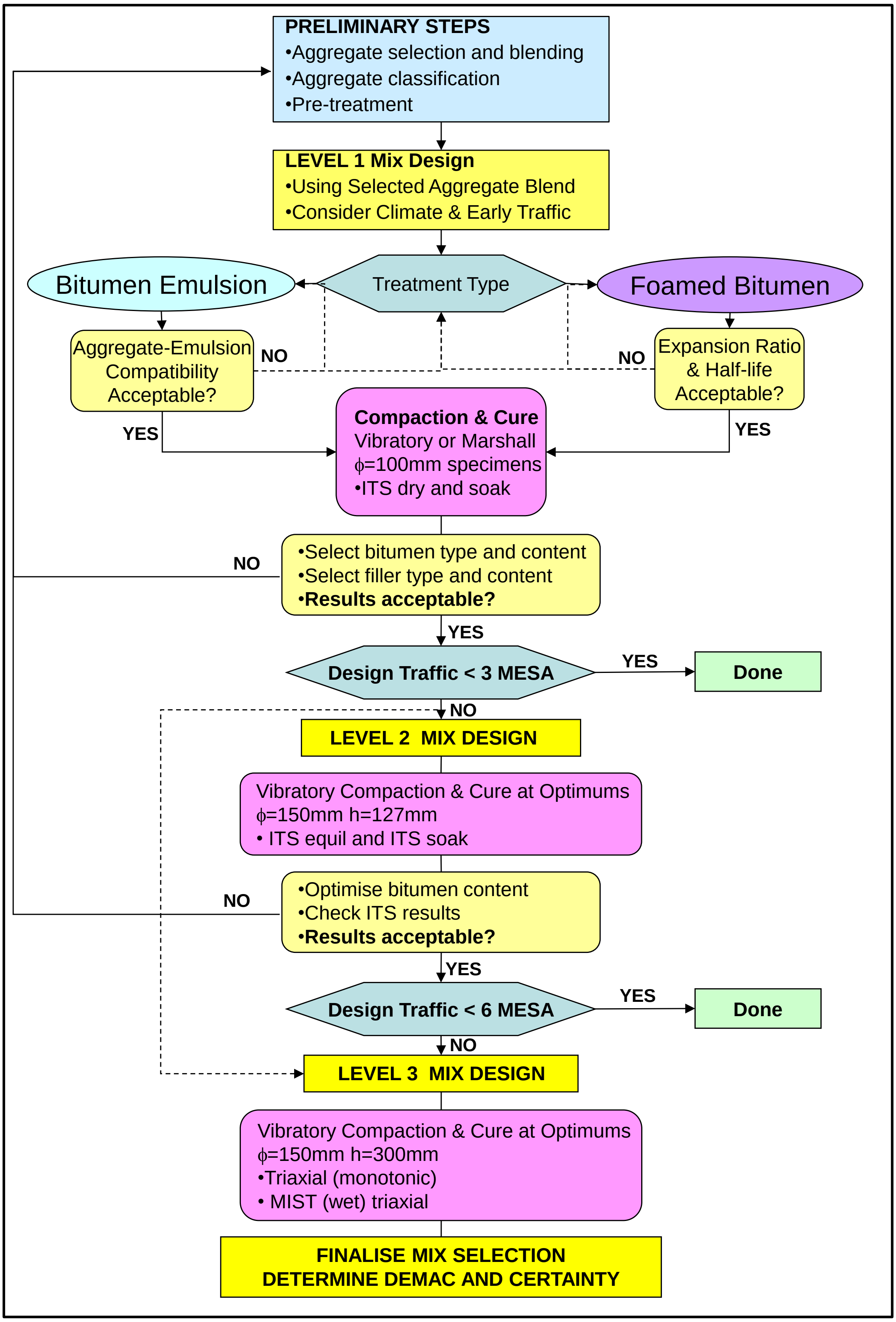


Figure 29. Mix Design Flow Chart for BSM Mixes

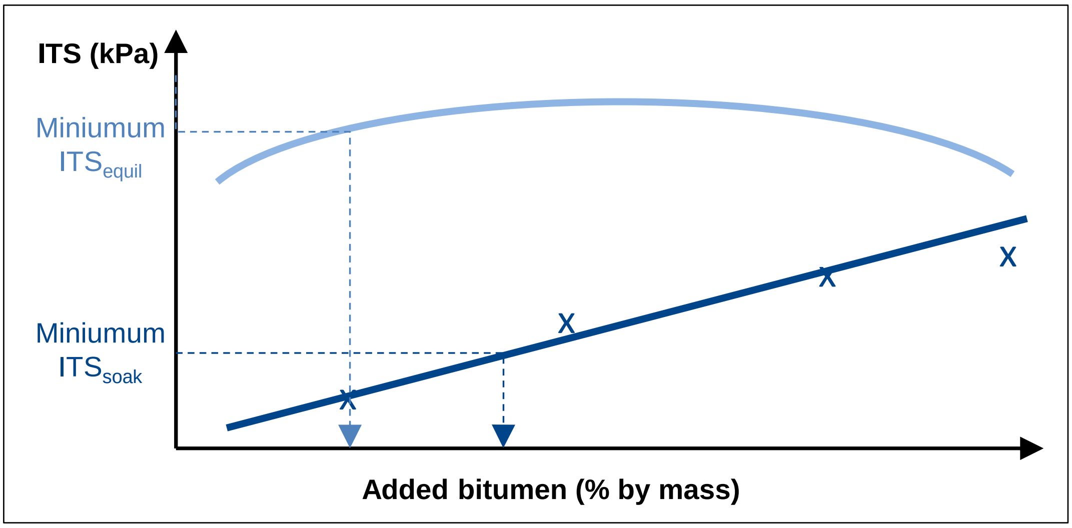


Figure 30. Typical ITS Test Results Showing Selection of Bitumen Application Rate

9.4.3 Level 3 Mix Design

Level 3 uses triaxial test results from 150 mm diameter, 300 mm high specimens to determine the shear properties, i.e., the cohesion and internal friction angle. Specimens are manufactured using vibrating hammer compaction and cured to approximate the equilibrium moisture content in the same manner as for Level 2 specimens. A simple triaxial test using relatively inexpensive equipment has been developed for this purpose. Specimens are tested at three different confining pressures to obtain the data required to plot the Mohr-Coulomb curve and obtain values for cohesion and internal friction angle. Additional specimens are required for the Moisture Induced Sensitivity Test (MIST) to determine the cohesion retained after subjecting the specimens to pressurised water induction. The MIST test is described in TG2.

Although Level 2 tests are often omitted when the Level 3 triaxial tests are undertaken, ITS tests on the larger 150 mm diameter specimens (Level 2) provide the best means for fine-tuning the application rates for bitumen and active filler, as well as the effectiveness of the active filler.

9.5 Where to use BSMs

BSMs are suitable for base layers on a wide range of pavements, from low volume roads to highways. They are also used in the subbase of heavily trafficked pavements where the base layer is constructed from asphalt. TG2 makes recommendations as to where the different BSM classes of materials can be used as a base layer for different traffic conditions:

- **BSM1** materials are suitable for pavements with a high structural capacity > 6 MESA.
- **BSM2** materials are suitable for pavements with a structural capacity less than 6 MESA.
- **BSM3** materials are only suitable for pavements with a structural capacity less than 1 MESA.

It is important to recognise that these recommendations are based on Long Term Pavement Performance (LTPP) data and are necessarily conservative.

The performance of a BSM layer in a pavement structure is determined by:

- Quality of the **parent material.**
- Proper **design procedures** for both mix design and pavement design.
- Competence of those responsible for **constructing** the layer.

Of primary importance in the construction process is proper mixing and achieving a high level of density throughout the thickness of the layer by adopting appropriate compaction procedures. The construction and quality management of BSMs is discussed in detail in Chapter 12: 2.7, 3.5 and 4.5 and Chapter 13: 5.

Construction of BSMs

The successful use of BSMs is heavily dependent on proper construction by competent construction crews. The construction and quality management of BSMs is discussed in detail in Chapter 12: 2.7 and 3.5

10. ASPHALT

This section addresses the utilisation and design of hot mix asphalt in the construction and maintenance of roads. The components used in the extensive range of asphalt surfacing and base mixes are covered. The engineering and functional properties required of the asphalt in the various positions where it is used in the structure of the road pavement are explained. The asphalt mix types which are most suited to these positions, or which have the most appropriate functional characteristics, are recommended.

Also addressed are the principles of asphalt mix design work that is carried out in the laboratory to optimise aggregate gradings and binder content, as well as the testing undertaken to evaluate the performance of the asphalt mix. Besides mix design in the laboratory, mention is made of the need to finally evaluate the mix in full-scale plant mixing and paving trials.

For asphalt to perform satisfactorily after it has been paved, it has to be compacted properly. This section concludes with information on the effects of compaction of the asphalt layer.

10.1 Composition of Asphalt

Asphalt mixes as used in surfacing and base mixes are generally composed of:

- Aggregates
- Filler
- Bituminous binders
- Reclaimed asphalt (RA) (sometimes)

These components are mixed together in proportions that ensure that the asphalt mix performs under the expected service conditions.

Asphalt Sections in SAPEM

This section is closely related to:

- Chapter 3, **Materials Testing**: Section 4.2
- Chapter 4, **Standards**: Section 4.2
- Chapter 10, **Pavement Design:** Section 7
- Chapter 12: **Construction Equipment and Method Guidelines:** Section 3.11
- Chapter 13: **Quality Management**, Section 6
- Chapter 14, **Post-Construction**: Section 4.1.

(i) Reclaimed Asphalt (RA)

The use of reclaimed asphalt (RA), illustrated in Figure 31, in hot mix asphalt is increasingly appropriate, both in economic and environmental terms. For example, it enables a reduction in:

- **Consumption of non-renewable resources**, like petroleum products, both fuel and bitumen, and aggregates.
- Use of **landfill space** for discarding asphalt removed from existing roads. Mixes incorporating RA are no longer regarded as inferior products, and are subjected to the same levels of quality management and performance expectations as those incorporating virgin materials only.

The process in which reclaimed asphalt (RA) is combined with new aggregate and new binder in a mixing plant to produce asphalt is included in this section.

Figure 31. Recycled Asphalt

10.2 Engineering Properties

Hot mix asphalt is designed to meet certain engineering properties, essential for the satisfactory performance of the surfacing and/or asphalt base layers. Three primary properties are:

- **Durability**
- Resistance to **cracking**
- Resistance to **permanent deformation**

Other important engineering properties that need to be considered are:

- Resistance to **shrinkage**
- **Flexibility**
- **Skid resistance** (for asphalt surfacings)
- **Permeability**
- **Stiffness** (elastic modulus)
- **Workability**

Each of these properties is discussed briefly below. The designer needs to strike a balance to deal with incompatibilities arising from opposing desired properties.

10.2.1 Durability

Durability of a hot mix asphalt layer is its ability to resist:

- **Hardening** of the bituminous binder due to
  - Oxidation
  - Loss of volatiles
  - Physical (steric) hardening
  - Loss of oily substances due to absorption of these into porous aggregates (exudative hardening)
- **Disintegration** of the aggregate
- **Stripping** of the bituminous binder from the aggregate
- **Action of traffic**

Durability of mixes can be improved by using:

- An appropriate bituminous binder in **relatively thick films**
- **Dense aggregate packing**, i.e., low air voids
- **Sound, durable aggregate**, resistant to stripping of binder films

10.2.2 Resistance to Cracking

Resistance to cracking is the ability of the layer to withstand tensile strains without fracture. This tensile strength is progressively reduced by repeated traffic and temperature related stresses, a process known as fatigue. Failure in tension (cracking), therefore, occurs when the applied stresses exceed the reduced tensile strength. Different types of asphalt mix exhibit varying resistance to this type of cracking.

In most cases, cracking occurs at low temperatures when the asphalt is brittle.

10.2.3 Resistance to Permanent Deformation

The ability of an asphalt layer to resist permanent or plastic deformation under the influence of traffic and elevated temperatures depends primarily on:

- Internal **frictional resistance**
- **Cohesion** (tensile strength)

Of these, frictional resistance is the dominant contributor to deformation resistance, especially at temperatures in excess of 40 °C which frequently occur in SA in summer. Aggregate gradings, angularity and roughness that produce optimal packing and provide adequate frictional resistance should be used where this kind of distress is a key consideration.

10.2.4 Resistance to Shrinkage

Shrinkage, which may lead to cracking, is typically caused by temperature-related volume changes as well as absorption of the binder into the aggregate and the resulting reduction of the volume of binder between the aggregate particles. It can be countered in much the same way as durability, particularly by dense aggregate packing and using less absorbent aggregate.

10.2.5 Flexibility

Flexibility is the ability of an asphalt layer to adapt to long-term non-uniform deformation of the pavement profile without cracking. Flexibility is promoted by:

- Higher **binder contents**
- **Flexible binder** type (softer grades or modified types)
- Appropriate **aggregate grading**

10.2.6 Skid Resistance

Skid resistant riding surfaces are especially important in high speed applications, particularly in areas where road surfaces are frequently moist. Skid resistance is improved by:

- Ensuring that adequate **air voids exist** in the mix to prevent the bitumen flushing of the surface.
- Selection of aggregates that have a rough **surface texture**, are resistant to polishing under the action of traffic and provide good micro-texture.
- An adequate amount of coarse aggregate remaining proud of the surface of the layer, providing **macro-texture.**

Examples of good and poor skid resistance are shown in Figure 32.

Asphalt with good skid resistance (but some cracks) | **New asphalt from the same area, with bleeding and low skid resistance**

Figure 32. Examples of Skid Resistance

10.2.7 Permeability

Permeability of an asphalt layer is a measure of the penetration of the mix by air, water and water vapour. Low permeability of an asphalt surfacing promotes long term durability and protects underlying layers from the ingress of water, which may lead to failure. Factors that reduce permeability are:

- **High binder contents** with adequate film thickness
- **Dense aggregate packing**
- **Dispersion** rather than inter-connection of air voids within the mix
- **Well compacted layers**

Where open textured and, hence, permeable surfacings are required to reduce noise, water spray and backsplash, these layers should always be placed on top of impermeable support layers.

Open Textured and Permeable Surfacing

Where open textured and, hence, permeable surfacings are required to reduce noise, water spray and backsplash, these layers should always be placed on top of impermeable support layers.

10.2.8 Stiffness

The stiffness of an asphalt layer determines the ability of the layer to carry and spread traffic loads to underlying layers. It is often expressed in terms of the elastic modulus of the material. Stiff asphalt is generally required for asphalt bases for effective load spreading to underlying layers. Less well supported surfacing layers may be better served by a lower stiffness asphalt, to avoid traffic induced cracking, provided the underlying support is still adequate to carry the traffic loads. The stiffness of asphalt is influenced by:

- Aggregate packing
- Binder content
- Flexibility (viscosity) of the binder
- Degree of compaction achieved during construction
- Temperature

10.2.9 Workability

Workability is the property of asphalt that facilitates good handling, spreading, compaction and uniformity of the layer under the prevailing conditions. This property is essential to achieve a uniformly acceptable product on the road. Variables that influence workability are:

- **Binder** content, viscosity and setting properties
- **Aggregate** grading, shape and type
- **Temperature** of the asphalt mix

For a given aggregate grading, workability is usually improved by:

- Increase in **binder content**
- Decrease in **binder viscosity**
- Less **angular aggregate**
- Limiting the **maximum aggregate size** to less than a third of the layer thickness
- **Construction controls** that ensure the mix is compacted at the proper temperature, normally by carrying this out quickly within an acceptable compaction time window

10.3 Mix Categories

It has been customary to categorise mixes in terms of a set of grading parameters. However, circumstances today increasingly require that volumetric, or spatial, concepts be considered. By so doing, a better understanding is developed of:

- How the components of the mix **pack together**
- How the packing **influences performance**

In addition, traditional gradings that have been in use for decades do not necessarily guarantee optimal designs today. This is especially in view of the change in aggregate shape over the years due to advances in crushing technology, and the increased heavy traffic loads that occur early in the life of the layer. Examination of the packing is often referred to as a "Level 1" design, to be followed by more complex testing and evaluation, to increase the confidence that the mix will perform as expected.

Examples for Recommended Skeleton Types

- **Thin layers**, serving primarily as water-tight seals on flexible, untreated pavements in low speed environments, such as residential streets, normally have sand-skeleton type mixes.
- For mixes on **high volume applications**, where anti-skid properties and resistance to permanent deformation (e.g., rutting) under elevated temperatures are key considerations, the preferred option is stone-skeleton type mixes.
- Between these two extreme conditions, a range of gradings for **general operating conditions** may be selected. **Continuous gradings** that ensure sand-skeletons are frequently selected for such general cases.

Two types of packing are generally in use:

- **Stone-skeleton mixes**: The spaces

between the coarser aggregate fractions are filled by the finer aggregate fractions, but do not push the coarser aggregates apart. Contact between the coarser aggregate fractions is thus assured. This situation results in the loads on the layer being carried predominantly by a matrix (or skeleton) of the coarser aggregate fraction. Stone-skeleton mixes are illustrated conceptually in Figure 33.

- **Sand-skeleton mixes**: In sand-skeleton mixes, the loads on the layer are mainly carried by the finer aggregate fraction, with the larger fractions providing bulk and replacing a proportion of the finer fraction. There is no meaningful contact between the individual larger aggregate particles. Sand-skeleton mixes are illustrated conceptually in Figure 34.

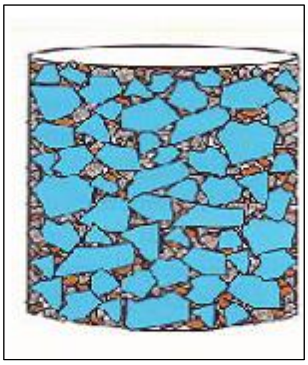


Figure 33. Stone-Skeleton Mix

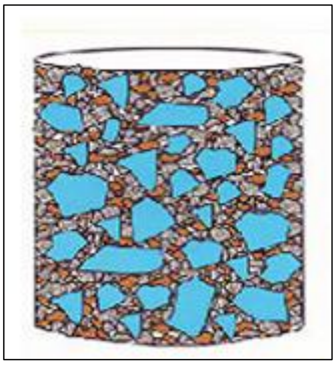


Figure 34. Sand-Skeleton Mix

A formal procedure of examining and achieving packing characteristics is given in Transport Research Circular Number E-C044: Bailey Method for Grading Selection in Hot mix Asphalt Mixture Design, October 2002. This method is known as the "Bailey Method".

Ternary diagrams, as used in Section 4.8 for granular materials, are also useful for classifying mixes according to the relative proportions of stones, sand and filler (Francken and Vanelstraete, 1993). An example is given in Figure 35.

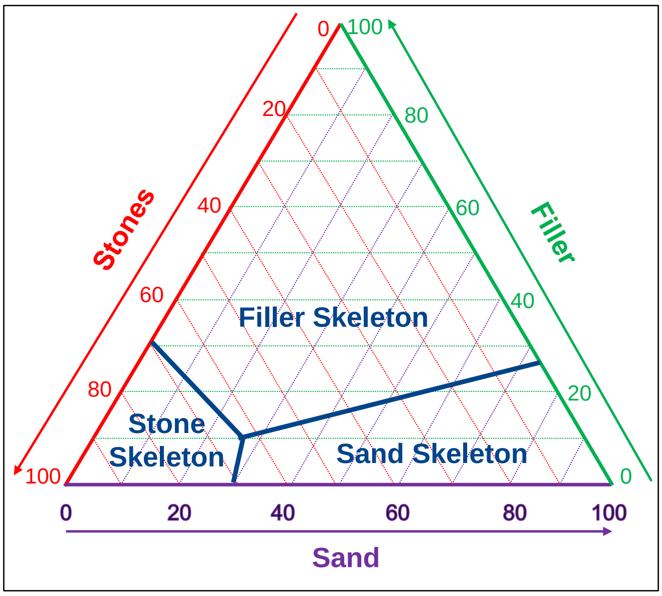


Figure 35. Ternary Diagram to Classify Asphalt Mixes According to Skeleton Type

Once a packing characteristic for a particular source of aggregates has been decided upon, and designs have been completed, mixes are described in terms of their gradings or particle size distributions. These gradings are then used in conjunction with other tests in quality assurance/quality control measures to ensure that design parameters are consistently met.

10.4 Grading Types

The gradings (or particle size distribution) of most asphalt mixes are generally classified into 5 classes, listed below and illustrated on a grading chart in Figure 36. Note that the exact meanings of these gradings differ in different parts of the world.

- Continuously graded, either continuous-coarse or continuous-fine
- Gap-graded
- Semi-gap graded
- Open-graded
- Semi-open graded

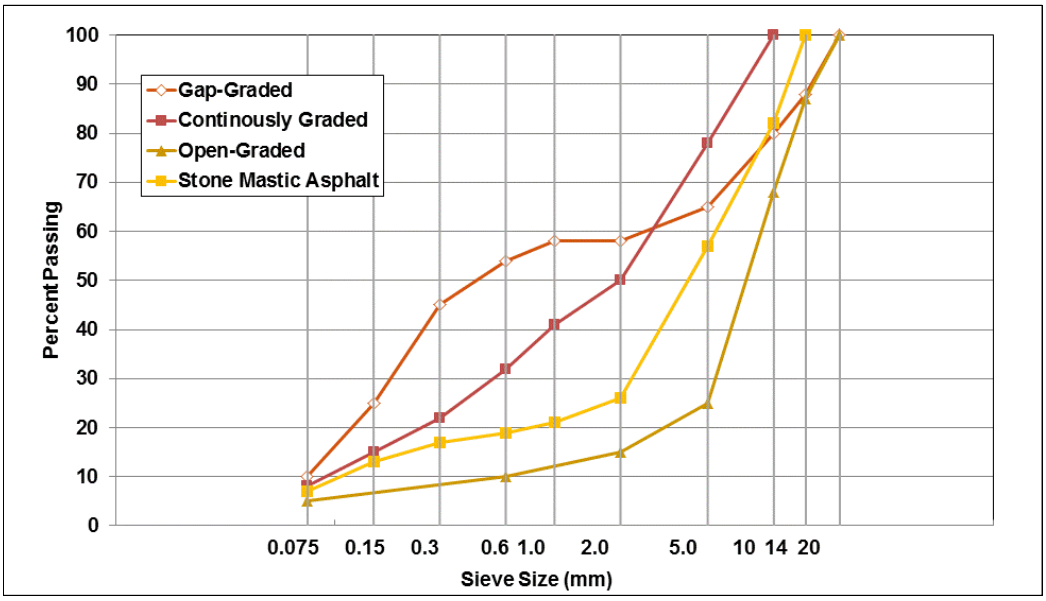


Figure 36. Representative Continuous, Gap and Open Graded Mixes

Stone mastic asphalt (SMA) and ultra-thin friction courses (UTFC) are regarded as special cases of gap graded asphalt. Semi-open graded mixes are less coarse than open-graded mixes, and the voids are largely filled with binder, typically a modified binder such as bitumen-rubber. General relationships between the various grading types and packing characteristics are given in Figure 37.

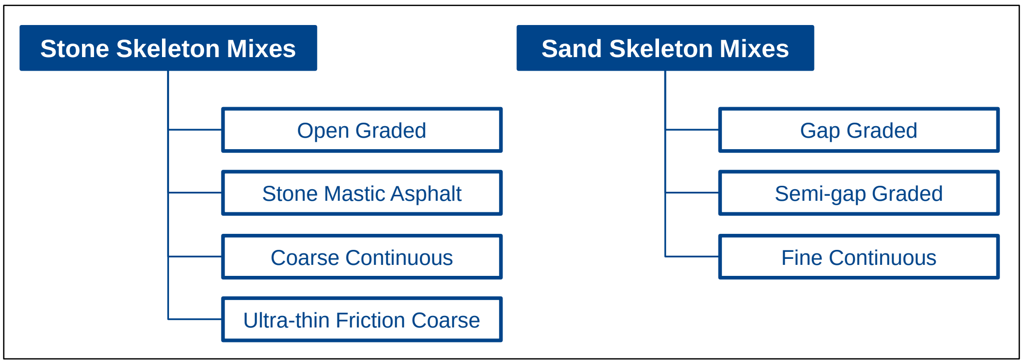


Figure 37. Relationships Between Grading Types and Packing Characteristics

10.5 Asphalt Surfacings

Asphalt surfacings provide the interface between the tyres of vehicles and the pavement, and are therefore one of the main structural layers of the pavement. They should meet the engineering properties covered in Section 10.2 and, in particular, they should be textured for adequate skid resistance. The aggregate packing adopted in asphalt surfacings could be either sand or stone-skeleton type, depending on the prime function of the layer. Examples of the recommended skeleton types for particular situations are given in the side box.

This section deals with the properties of the following asphalt surfacings:

- Gap-graded (AG)
- Continuously graded (AC)
- Semi-gap-graded (AS)
- Open graded (AO)
- Stone mastic asphalt (SMA)
- Semi-open graded asphalt (ASO)
- Ultra-thin friction courses (UTFC)

In general, the achievement of the desired properties of mixes depends on both the bituminous binder and the aggregate quality and packing. Consequently, the use of more costly modified binders should be considered on the basis of cost-effectiveness. Modified binders are discussed in Section 8.1.3.

In cases where RA is added to the wearing course mix, TRH21 makes recommendations on the maximum proportions of RA, ranging from 3% (SMA) to 18% for conventional mixes in wearing courses.

10.5.1 Choice of Mix Types

Asphalt surfacings, or wearing coarses, are divided into two broad categories in terms of the primary purpose served by such layers:

- **Structural** layers generally have a specified thickness of ≥ 30 mm and are designed to contribute measurably to the strength of the pavement and to provide adequate skid resistance for the prevailing conditions of traffic and climate.
- **Functional** layers have a specified thickness of < 30 mm, do not contribute significantly to pavement strength and can best be described as surface dressings that meet functional criteria, such as:
  - Suitable surface texture for **skid resistance**, **noise reduction** and surface water **drainage** given the traffic volumes, speed and prevailing climate
  - **Sealing** of the substratum against water penetration
  - Limited improvement of **riding quality**

Functional layers are used in two distinct applications:

- **Thin asphalt layers** for low speed and light traffic applications, mainly in residential areas. The design and construction of the thin asphalt layers for low speed, residential streets are covered fully in Sabita Manual 27: Guideline for Thin Layer Hot Mix Asphalt Wearing Courses on Residential Streets.
- **Ultra-thin friction courses (UTFC)** for high volume, often high speed, applications on major highways.

Table 17 compares the various categories of wearing courses and the primary expectations associated with each.

10.5.1.1 Ultra-Thin Friction Coarse (UTFC)

UTFC'S are hard wearing surfacing systems comprising fairly open graded stone-skeleton type mixes and a sprayed layer of binder applied with self-priming pavers to ensure an impermeable membrane at the base of the layer. Modified binders are generally employed in these layers.

UTFC's should preferably be accredited by Agrément South Africa (ASA) as being fit for purposes in accordance with the criteria outlined in their guidelines (see Section 14.1). Critical criteria assessed during the certification process and on-site quality management are:

- Aggregate polishing resistance
- Bond strength between surfacing and substrate (Torque bond value)
- Skid resistance

- Texture depth
- Resistance to moisture induced stripping
- Aggregate strength (ACV)
- Permanent deformation
- Durability (resistance to ravelling)

Where it is not feasible to apply an ASA certified product, the reader is referred to the Standard Specifications.

Table 17. Selection of Appropriate Asphalt Surfacing Mixes

Property	Category		
	Structural	Functional	
		Thin asphalt	UTFC
Thickness	≥30 mm	< 30 mm	< 30 mm [1]
Key properties	• Low permeability to protect the substratum, especially granular bases • Adequate skid resistance • Structural integrity	• Low permeability to protect the substrate • Compactibility, given the rapid cooling of thin layers • Surface texture for low speed (< 80 kph) skid resistance	• Low permeability to protect the substrate • Surface texture for skid resistance, noise reduction and surface drainage • Adhesion to substrate
Quality assurance	• Performance related testing	• Examination of functional properties, e.g., uniformity of surface texture, permeability and skid resistance	• Preferably Agrément South Africa accreditation

Note
1. Frequently less than 20 mm

10.5.1.2 Design Considerations

In determining the desired mix proportions, the designer should identify the key objectives of the design and strike a balance in targeting mix parameters to ensure the desired engineering properties are achieved.

During this process, determining the aggregate packing characteristics of the mix, i.e., a stone-skeleton or sand-skeleton type mix, are critical choices to be made for mix type selection. These choices ultimately determine the grading of the specific blend of aggregates used and typical grading types for various applications. Table 19 shows some grading types for various applications from the Interim Guidelines for the Design of Hot Mix Asphalt in South Africa (HMA, 2001).

10.5.1.3 Maximum Aggregate Size

A fundamental property of the grading of the mix is the nominal maximum aggregate size. This size should be selected with due consideration of the intended layer thickness. Except for UTFC's, it is generally accepted that the maximum aggregate size should be at most one third of the layer thickness to ensure compactibility and to counter segregation during paving. That means that for a 40 mm layer, the maximum aggregate size should not exceed 14 mm or for a 30 mm layer the maximum aggregate size should not exceed 10 mm.

Recommended minimum layer thicknesses for maximum aggregate sizes are indicated in Table 18 as per Sabita Manual 5: Guidelines for the Manufacture and Construction of Hot Mix Asphalt.

Table 18. Recommended Minimum Layer Thickness

Maximum Aggregate Size (mm)	Minimum Layer Thickness (mm)	Preferred Layer Thickness (mm)	Applications
7.1	20	25 – 30	Asphalt surfacings on residential streets
10	30	35 – 40	Conventional surfacings
14	45	50 – 60	Thick surfacings and bases
20	80	90 – 100	Bases
28	100	110 – 125	Typically LAMBs (large aggregate mixes for bases)

Table 19. Grading Types for Various Applications

Layer	Type of Grading and Binder	Typical Application	Performance Rating (1 = Poor, 5 = Good)				
			Rut Resistance	Durability/ Fatigue Resistance	Skid Resistance	Impermeability to Water	Noise Reduction
Conventional thickness (30 – 40 mm)	Continuous, 50/70 pen	Surfacing/overlay	3	3	3	3	3
	Continuous, bitumen-rubber	Flexible surfacings/overlays	3	4	3	2	4
	Continuous, SBS modified	Flexible surfacings/overlays	4	4	3	3	3
	Continuous, SBR modified	Flexible surfacings/overlays	3	4	3	3	3
	Continuous, EVA modified	Rut-resistant surfacing	4	3	3	3	3
	SMA, 50/70 pen	Rut-resistant surfacing	4	4	4	3	4
	SMA, modified	Rut-resistant surfacing	5	4	4	3	4
	Open-graded, 50/70 pen	Drainage/noise reduction	4	2	4	N/A	5
	Open-graded, modified	Drainage/noise reduction	5	3	4	N/A	5
	Semi-Gap, 50/70 pen	Flexible surfacing/overlay	2	3	4[1]	4	4
	Semi-Gap, modified	Flexible surfacing/overlay	2	4	4[1]	4	4
	Semi-open, bitumen-rubber	Flexible surfacing/overlay	4	5	4	2	4
	Gap-graded, 50/70 pen	Flexible surfacing/overlay	2	4	3	5	3
Thin (< 30 mm)	SMA, 50/70 pen	Functional layer	5	3	4	4[2]	4
	SMA, modified		5	4	4	4[2]	4
	Open graded, 50/70 pen		5	3	4	N/A	5
	Open graded, modified		5	3	4	N/A	5
	UTFC		5	4	4	4[3]	5
	Fine continuous for residential streets		–	4	4	4	–

Notes
1. With rolled-in chips.
2. Impermeable support layer or membrane required.
3. A continuously applied bituminous membrane is usually a standard component of this type of surfacing.

10.5.2 Constituent Materials

10.5.2.1 Aggregates Used In Asphalt Surfacing Mixes

Several factors should be taken into account with aggregates used in asphalt surfacings are covered in Chapter 3: 4.2.2, including:

- **General requirements**, including cautionary notes on aggregate types as well as factors to take into account when using rolled in chips with continuously graded surfacing mixes
- **Grading**, including gradings for rolled in chips
- **Physical properties**
- **Chemical properties**, especially those that influence adhesion of the binder to the stone particles

10.5.2.2 Filler

The filler, i.e., the material substantially passing the 0.075 mm sieve, is incorporated in asphalt to:

- Act as a **binder extender** to stiffen the mastic and, hence, the mix
- Act as a **void filling material**
- Improve the **bond** between the binder and the aggregate (in some cases)

The various fillers in general use are shown in Table 20 with comments on their use.

Table 20. Fillers In Common Use

Filler Type/Origin	Comments and Characteristics
Hydrated lime	• Active filler • Improves adhesion of bitumen to aggregate • Improves mix durability by retarding oxidative hardening of the binder • Low bulk density, high surface area
Portland cement	• Active filler
Bag house fines **Cyclone dust**	• Variable characteristics requiring greater control • Some source types may affect the durability of the mix and render mixes sensitive to binder content
Fly ash	• Improves mix compactibility • Low bulk density • Variability may require greater control

In addition to evaluating the properties of fillers, the designer should also be mindful of the relative costs of various fillers as well as their availability, storage potential and relative effects on the stiffness of the binder. Criteria that fillers should meet with regard to percentage passing 0.075 mm sieve size, as well as bulk density and void content are given in Chapter 4: 4.2.3. Recommendations are also given for the chemical properties of fillers.

The following list gives some important issues which must be considered when designing fillers for asphalt mixes:

- Excessive quantities of active filler may cause an **undesirable increase in the viscosity** of the hot mastic that causes difficulties during the compaction of the mix. As a general rule, it is recommended that the filler-binder ratio should not exceed 1:2. For thin layer asphalt layers, i.e., less than 30 mm thickness, a maximum ratio of 1:1.5 is preferred.
- The **compactibility of stone-skeleton mixes**, e.g., SMA, is less affected by high filler-binder ratios as the compaction and stability properties are determine mainly by stone-to-stone contact.
- The proportion of **hydrated lime should not exceed 2%** by mass of dry aggregate. Quantities in excess of this tend to produce dry and brittle asphalt mixes. Small quantities of lime, i.e., 1 – 1.5% by mass of dry aggregate, can improve the resistance of the mix to stripping of the asphalt from the aggregate.
- Fillers have a wide range of **bulk densities**. If the filler used in a mix is substituted by another type of different bulk density, this difference must be taken into account to ensure that the volume concentration remains the same. Thereby changes in workability, stiffness of the mix and the optimum binder content are prevented.

10.5.2.3 Reclaimed Asphalt (RA)

In South Africa, reclaimed asphalt (RA) is generally obtained from milling, but also from the excavation of existing asphalt pavements or stockpiles of discarded or surplus asphalt. It is then crushed and screened to achieve a reasonably well graded, free flowing, and consistent product. See Chapter 8: 4.3 for further discussion on RA.

As with virgin aggregates, RA should be free of foreign, deleterious material such as unbound granular base, broken concrete and crumbed rubber. Asphalt modified with bitumen rubber is thus not suitable for recycling.

Recycling Asphalt with Bitumen Rubber

Asphalt modified with bitumen rubber is not suitable for recycling because of the presence of the crumbed rubber, which on heating results in unacceptable levels of air pollution.

The following list gives some important issues which must be considered when using RA:

- Over the past fifty years a wide range of asphalt mixes have been in use, both in terms of aggregate composition and binder types, particularly as a result of increased use of modified binders. More recently, SMA's were introduced. Ultra-thin friction courses (UTFC's) have increasingly replaced surfacing seals as a means of applying a suitable texture to an existing asphalt surfacing. The **variable composition of RA** arising from these circumstances clearly has to be carefully considered when contemplating the recycling of these materials.
- **Asphalt containing coal tar**, or incorporating coal tar prime coats, normally identified by its pungent odour, should not be used, in the interests of health and environmental considerations.

(i) Fractioning

TRH21: Hot Mix Recycled Asphalt recommends that reclaimed asphalt is crushed and fractioned, depending on the proposed RA content of the final mix. The maximum size is generally 28 mm and the fractioning is into 3 sizes: 28 – 14 mm, 14 – 7.1 mm and minus 7.1 mm for mixes containing in excess of 15% RA.

(ii) Moisture

Excessive moisture in the RA adds significantly to the cost of production, as a result of increased heating required to reduce the moisture content. This additional heating may also cause an imbalance in the production stream. It is generally accepted that the moisture content of the final mix containing RA should not exceed 0.5% by mass. Stockpiling techniques and procedures should therefore be in place to limit the moisture content of the RA fed into the mixing plant.

10.5.2.4 Bituminous Binders

The bituminous binder should be selected with due consideration of the aggregate packing (grading) of the mix, in conjunction with traffic and environmental conditions. Bituminous binders and modifiers in general use in wearing course asphalt are set out in Table 21.

Bitumen may also contain additives, which improve the properties of bitumen for specific applications, whilst not forming an integral part of the binder. Additives in general use are given in Table 22.

There is an increasing tendency to use modified binders in asphalt surfacings to cope with higher intensity loading, arising from increases in traffic volumes, axle loads and tyre pressures. The onus is on the designer to ensure that the use of these more costly binders, in conjunction with suitable aggregate packing, offer cost-effective solutions. Modified binders are discussed in Section 8.1.3.

Table 21. Bituminous Binders for Asphalt Surfacings

Class	Category	Grade/type	Applications
Conventional binders (complying with SANS 4001-BT1)	Penetration grade road bitumen	35/50 pen	• High traffic situations where high stiffness is required. • Generally not appropriate for situations of yielding support layers and low temperatures.
		50/70 pen	• Gap, semi-gap, continuous and open-graded asphalt for typical applications in most climatic zones.
		70/100 pen	• For improved flexibility of asphalt on more flexible pavements. • Especially appropriate for very thin layers on residential streets where extending the compaction window under conditions of rapid cooling may be a critical consideration.
Modified binders (complying with TG1)	Elastomer	Styrene-butadiene-rubber (SBR) latex	• Improved flexibility and resistance to fracture. • Increased stiffness at elevated temperatures. Lower stiffness at low service temperatures.
		Styrene-butadiene-styrene (SBS)	
		Bitumen-rubber	
	Plastomer	Ethyl-vinyl-acetate (EVA)	• Improved resistance to permanent deformation.
	Natural hydrocarbons	Gilsonite Durasphalt	• Stiffening of the bitumen and hence the stiffness modulus of the asphalt layer.
	Aliphatic synthetic wax	FT Wax	• Primarily for lowering the mixing and laying temperatures. • Also has a beneficial effect on the resistance to permanent deformation. • More resistant to fuel spillage than conventional binders.

Table 22. Bitumen Additives

Additive	Type	General purpose
Natural or synthetic fibres	Natural rock wool Polypropylene Polyester Fibreglass Mineral cellulose	Reduced risk of drain-down of binder especially in open-graded asphalt and SMA. Improved tensile strength and cohesion of the asphalt.
Anti-stripping agents	Amines Lime	Minimises stripping of binder from aggregate

10.6 Asphalt Base

Asphalt bases are normally employed at the higher end of the traffic loading spectrum, i.e., ≥ 3 million equivalent single axle loads (MESA) on important roads (Category A and B). In these situations they are cost-effective. By their very nature they form a dominant strength component of such pavement types. They are designed to meet three main requirements:

- **Distribution of the loads carried to underlying layers**. For this purpose it should be relatively stiff material, i.e., with a high stiffness modulus.
- **Resistance to permanent deformation**, particularly under conditions of elevated temperatures and high volumes of slow moving heavy traffic.
- **Durability**, i.e., properties should be maintained at a suitable level over an extended period of time.

It may also be appropriate to use asphalt bases on shorter lengths of roads, or lanes carrying less than 3 MESA during reconstruction or rehabilitation works in confined areas, to limit the depth of excavation and disruption to traffic flow. Asphalt bases may also be considered where construction has to be fast-tracked, or in situations where moisture conditions rule against unbound granular bases.

When designing asphalt bases, the designer should still be mindful of the resistance of the layer to fatigue. However, this is a lesser consideration for asphalt base layers, given the material's high stiffness, and hence low strains, which are further reduced by the conventional practice of providing a stiff support to the layer, often in the form of a cementitious subbase layer.

As is the case with wearing course asphalt, reclaimed asphalt (RA) may be incorporated in the base mixes. TRH21 recommends a maximum RA content 27% for asphalt bases.

10.6.1 Mix Types

Stone-skeleton packing is preferred for asphalt bases in view of the prime layer requirements. Larger maximum aggregate sizes are selected and gradings generally are:

- **Coarse continuous**, e.g., dense bitumen macadam (DBM)
- **SMA type** grading with significantly larger maximum aggregate sizes (> 25 mm). Typically Large Aggregate Mixes for Bases (LAMBs) as described in Sabita Manual 13, "The Design, Construction and Use of Large Aggregate Mixes for Bases".

HiMA Mixes

High modulus asphalt (HiMA) mixes are essentially sand-skeleton types, containing a high proportion of very hard (15–25 pen.) binder yielding very high stiffness moduli. These mixes have been used with success in Europe under conditions of extremely heavy traffic loading on highways and airports.

An exception is high modulus asphalt (HiMA), which are essentially sand-skeleton types, containing a high proportion of very hard (15 – 25 pen) binder yielding very high stiffness moduli.

When larger aggregate sizes are employed, the designer should consider the workability and compactibility of the mix to ensure uniformity and adequate compaction of the finished layer. Segregation can become a major factor with increasing aggregate size.

10.6.1.1 Maximum Aggregate Size

A fundamental property of the grading of the mix is the nominal maximum aggregate size. This size should be selected with due consideration of:

- Intended total **layer thickness**
- **Paving depth** where the layer is constructed in successive passes of the paver

To ensure compactibility and to counter segregation during paving, it is generally accepted that the maximum aggregate size should be at most one third of the layer thickness. Recommended minimum layer thicknesses for maximum aggregate sizes are indicated in Table 23 from Sabita Manual 5.

Table 23. Recommended Base Layer Thickness

Maximum Aggregate Size (mm)	Minimum Layer Thickness (mm)	Preferred Layer Thickness (mm)
14	45	50 – 60
20	80	90 – 100
28	100	110 – 125

10.6.1.2 Grading of Aggregate Bases

Grading guidelines for asphalt bases are given in Chapter 4: 4.2.4. Asphalt mix properties depend on a number of factors other than grading. Therefore, an insufficiency in a particular engineering property does not necessarily imply that the asphalt mix type cannot be designed to accommodate this inadequacy. Table 24 compares the three mix types by rating their engineering properties.

Mixes with Large Aggregates

The workability and compactibility of mixes with large aggregates must be considered to ensure adequate compaction can be achieved. The large aggregates can also cause segregation in the mix. To counter this, the maximum aggregate size should be at most one third of the layer thickness.

Table 24. Ratings of Asphalt Mixes in Terms of Engineering Properties

Engineering Property	Rating (1= good and 5 = poor)		
	Grading		
	Continuous	Semi-gap	LAMBS
Durability	1 – 2	2	3
Tensile strength	1	2	2
Fatigue resistance	4	2	4
Deformation resistance	1 – 2	2 – 3	1
Impermeability	2 – 3	2	4
Stiffness	1 – 2	2	1
Shrinkage	1	2	1
Workability	2	2	4

10.6.2 Constituent Materials

10.6.2.1 Aggregate Used in Asphalt Base Mixes

As with asphalt surfacing mixes, several factors should be taken into account with aggregates used in asphalt bases. These are covered in Chapter 4: 4.2.4 and include:

- General requirements
- Grading
- Physical properties

10.6.2.2 Filler

The general requirements listed in Section 10.5.2.2 also apply.

10.6.2.3 Reclaimed Asphalt

The general requirements, and those pertaining to fractioning and control of moisture, as outlined in Section 10.5.2.3, also apply to bases.

10.6.2.4 Bituminous Binders

The bituminous binder for asphalt bases should be selected with due consideration of the aggregate packing (grading) of the mix, in conjunction with traffic and environmental conditions. Bituminous binders in general use in asphalt bases are set out in Table 25.

Table 25. Bituminous Binders for Asphalt Bases

Category of Conventional Binder[1]	Grade/type	Applications
Penetration grade road bitumen	35/50 pen	• High traffic situations where high stiffness is required. May not be appropriate for low temperature applications.
	50/70 pen	• Semi-gap and continuously graded asphalt for typical applications in most climatic zones.
Elastomer	Styrene-butadiene-styrene (SBS) Bitumen-rubber	• Improved flexibility and resistance to fracture. • Increased stiffness at elevated temperatures. • Lower stiffness at low service temperatures.
Plastomer	Ethyl-vinyl-acetate (EVA)	• Improved resistance to permanent deformation.
Natural hydrocarbons	Gilsonite Durasphalt	• Stiffening of the bitumen and hence the stiffness modulus of the asphalt layer.
Aliphatic synthetic wax	FT Wax	• Primarily for lowering the mixing and laying temperatures. • Beneficial effect on the resistance to permanent deformation.

Note
1. Complying with SANS 4001–BT1

10.7 Asphalt Mix Properties

It is important for the designer to understand that no clear quantitative relationships between test results and actual performance have been established to date. However, experience with the performance of mixes guides the design of the mix. The evaluation of some test results are based on ranges of values, rather than fixed criteria.

10.7.1 Structural Layers

Asphalt mixes for surfacings and bases that form significant structural components of the pavement are designed to meet performance properties listed in Section 10.2. Three major performance properties that need to be tested and evaluated are the asphalt's resistance to:

- Permanent deformation
- Fatigue cracking
- Moisture damage

Other performance related tests for specific mix types also in general use are:

- Indirect tensile test
- Stiffness modulus
- Resilient modulus
- Dynamic creep
- Cantabro abrasion test
- Schellenburg drainage test
- Water permeability
- Axial loading test

The tests are discussed in Chapter 3: 4.2.5. Current design criteria are somewhat variable and it is recommended that the designer consult TRH8, HMA and Sabita Manual 24 to establish criteria relevant to the specific project.

10.7.2 Asphalt Seals (Functional Layers)

Thin surfacings, i.e., those of specified thickness of less than 30 mm, on roads carrying light traffic in residential areas, do not form a significant component of the structure of the pavement. They should be designed principally for:

- Durability
- Low permeability
- Workability (ease of compaction)
- Flexibility
- Surface texture associated with low speeds

In such cases, a volumetric design approach is normally adopted. Sand-skeleton mixes are more conducive to meeting the required criteria. The designer is referred to Sabita Manual 27, "Guideline for Thin Layer Hot Mix Asphalt Wearing Courses on Residential Streets" for further guidance.

10.8 Principles of Asphalt Mix Design

10.8.1 HMA Mix Design in the Laboratory

The asphalt design method described in TRH8 has been used in South Africa for well over a decade. This method is based on the Marshall design method, but includes additional information and criteria on component evaluation. Changes in the road building industry, such as increases in traffic loadings and the use of new asphalt mixes which are not catered for, prompted the development of a new HMA design method published in 2001, "Interim Guidelines for the Design of Hot Mix Asphalt in South Africa" (HMA, 2001). TRH8 and the Interim Guidelines for the Design of Hot Mix Asphalt in South Africa should be used together when carrying out HMA mix design work.

Guidelines in This Manual

The guidelines provided in this manual are not intended to serve as a design manual, but to rather outline the methodology and most important considerations to be made during the design stage.

NEW South African HMA Design Method

Sabita is compiling a new "**Asphalt Mix Design Manual for South Africa**". The purpose of this manual is to establish a common base for the design of asphalt mixes in South Africa, and to advance the move towards performance-related specifications for the design of asphalt pavement materials (see Chapter 3: 4.2.6). Significant developments in asphalt technology have taken place since the publication of the 2001 HMA guidelines, and therefore the existing design methods need updating, particularly in the light of the following developments:

- The revision of the South African Pavement Design Method (SAPDM), which allows for direct linkages between asphalt mix design, structural design and field performance in terms of resilient response and damage evolution (see Chapter 10: 7.1).
- The increasing use of mix types that cannot be classified as conventional Hot-Mix Asphalt (HMA) and that require alternative design methods, including warm mix, cold mix, mixes with significant proportions of reclaimed asphalt, stone mastic asphalt and Enrobé à Module Élevé (EME) asphalt.
- International and local advances in asphalt technology.
- Increase in volume of heavy vehicles on South Africa's roads.
- The need for roadway infrastructure for bus rapid transit systems.
- A demand for higher performance mixes, often leading to more sensitive mix designs.
- A need to review the current national compliance criteria for asphalt layers in contract specifications.

This new guideline will supersede the 2001 HMA guideline and TRH8. Most of the basics and principles will essentially remain the same as given in those guidelines and SAPEM, but the details of the design method will be updated. The new manual will be released by Sabita through the Society for Asphalt Technology (SAT) in late 2014 and 2015. Contact Sabita, www.sabita.co.za.

Note that the guidelines provided in this manual are not intended to serve as a design manual, but rather to outline the methodology and most important considerations to be made during the design stage.

The Interim Guidelines provide useful guidelines for the design process that can be roughly divided into in four phases:

- Preliminary considerations leading to **mix selection** and rating of design objectives
- **Component evaluation**: aggregate, filler and binder
- **Volumetric design,** leading to the grading selection and an optimum binder content
- **Performance testing**

Figure 38 illustrates the laboratory asphalt mix design procedure. As can be seen in this figure, a phased approach is adopted.

- **Phase 1: Evaluate the Design Situation.** Traffic, pavement, climate, construction, material availability, geometry and the environment.
- **Phase 2: Design Objectives**. This evaluation enables the most appropriate mix type to be selected.
- **Phase 3: Evaluate the Materials.** This includes the aggregates, filler, and binder that will be used in the mix.
- **Phase 4: Volumetric Design Process**.
  - Usually more than one aggregate grading is included in the laboratory design so that the packing properties of the specific aggregates can be optimised. Samples of the various aggregate fractions and filler are prepared and heated to the required compaction temperature.
  - Heated binder is then added and thoroughly mixed together with the aggregate and filler.
  - Briquette specimens are prepared by compacting each of the mixed samples in a steel mould.
  - Once cooled, the briquette is extruded from the mould and is subjected to various tests, to determine its compaction and volumetric properties. These properties are used to select the mix's optimum binder content.
  - Other performance related tests, to determine properties such fatigue and rut resistance, may be performed on other laboratory prepared samples, as a further means of evaluating the suitability of the mix.
- **Phase 5: Performance Testing**.

Form D3 in TMH10, "Manual for the Completion of As-built Materials Data Sheets" is useful as a means of collating the trends of the various test parameters. An example of Form D3 is given in Figure 39, which shows results obtained on the design of a AC Medium continuously graded mix. Note that

Rules of Thumb for Binder Content Selection

To select a target binder content from the "optimum" determined with the standard Marshall compaction, the following rough guidelines, based on the paving thickness relative to the 64 mm Marshall specimen height have been shown to be useful:

- For every 10 mm paving thickness less than 64 mm, add 0.1% additional bitumen to the target binder content.
- For every 10 mm paving thickness greater than 64 mm, reduce the design/target binder content by 0.1%.

Form D3 is normally presented on one A3 page. It can be seen from the plots of the Marshall Stability and Flow results, as well as from the plot of the Indirect Tensile Strength, that in this case, peak values are achieved around 5.5% binder. The results of void content determinations tend to match this binder content, and an **optimum** binder content of 5.4%, which corresponds to a mid-range void content of 4.5%, is selected.

10.8.2 Full-Scale Plant Mix and Paving Trials

Once the laboratory mix design has been completed, it is necessary to verify the asphalt's properties by manufacturing the mix in a full-scale asphalt mixing plant using the proportions of components established in the laboratory mix design. Often some "shift" between the test results of the laboratory produced mix and that produced in the mixing plant are found, and adjustments are made to achieve the desired mix properties. The final stage of accepting the mix is a paving trial where the mix's workability and compactibility are verified. Further details on asphalt trials are covered in Chapter 12: 4.1. Quality management for asphalt construction is given in Chapter 13: 6.

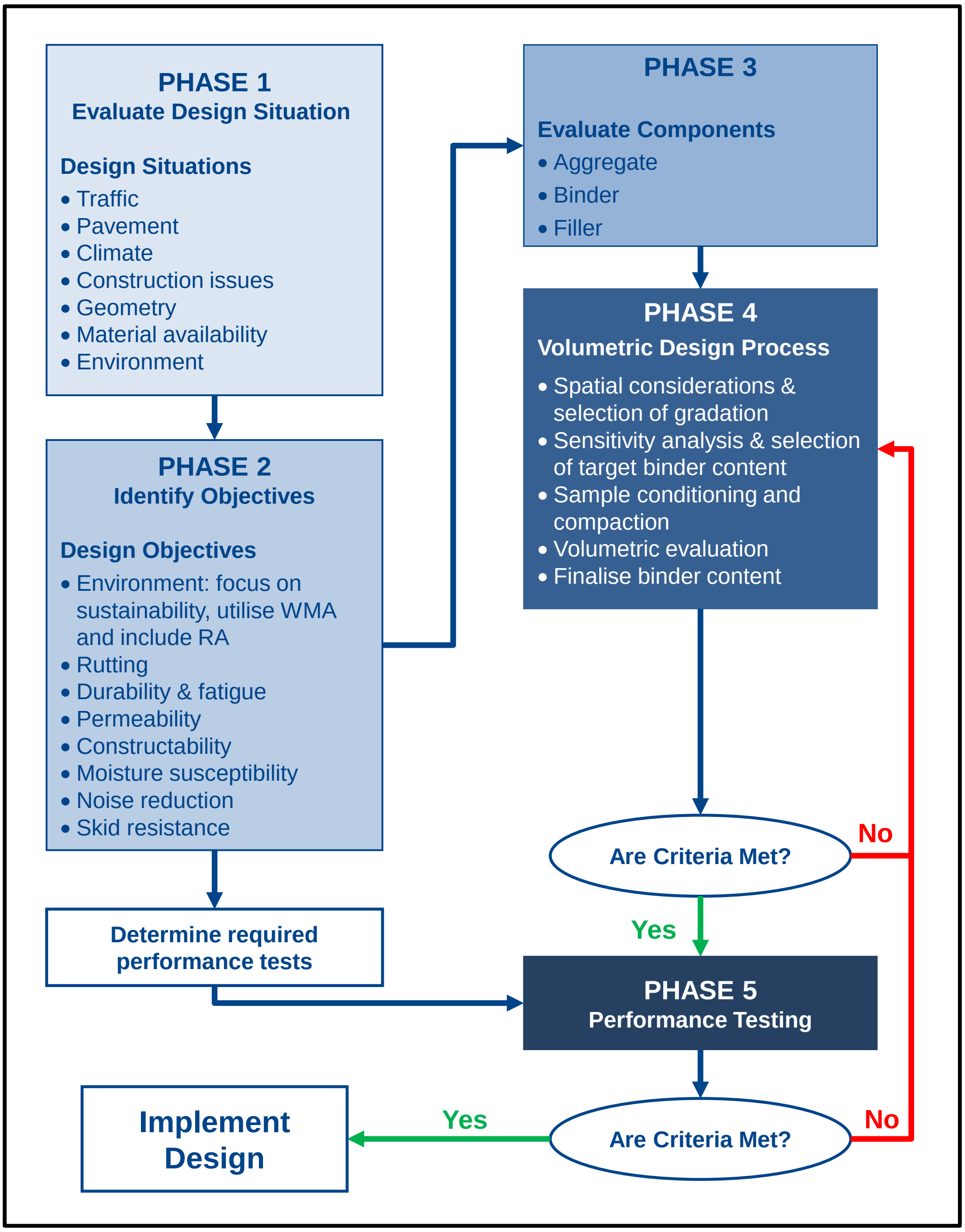


Figure 38. Asphalt Mix Design Procedure

		Prepared by: Voorberei deur:	**ASPHALT MIX DESIGN**
		Date / Datum:	**ASFALT MENGSELONTWERP**

Aggregates / Aggregate

Sample / Monster No		Nom. Size / Grootte	Type and Source / Tipe en Bron
1	1	9.5mm	Tillite ex Scottburgh Crushers
2	2	6.7mm	Tillite ex Scottburgh Crushers
3	3	Fine Crusher Dust	Tillite ex Scottburgh Crushers
4	4	Course Dust	Tillite ex Scottburgh Crushers
5	5		
6	6		
7	7		
8	8	Lime	Lime Distrubutors

NOTE : Test Methods Refer To T.M.H. 1 [ ]. SABS [ ]
NOTA : Toetsmetodes Verwys Na T.M.H. 1 [ ]. SABS [ ]

Sieve Analysis / Sifanalise [B 4 } - % Passing Sieves / % Deur Siwwe

Sample No. / Monster No.		1	2	3	4	5	6	7	8	**Design / Ontwerp** Mix / Mengsel	Spec. / Spes.		
% In Mix / Mengsel		14.0	30.0	49.0	6.0				1.0				
Sieve Size (mm) Sifgrootte (mm)	37.5	100	100	100	100				100	100	100	-	100
	26.5	100	100	100	100				100	100	100	-	100
	19.0	100	100	100	100				100	100	100	-	100
	13.2	100	100	100	100				100	100	100	-	100
	9.5	93	100	100	100				100	99	82	-	100
	6.7	2	76	100	96				100	79	64	-	85
	4.75	1	22	95	84				100	59	54	-	75
	2.36	1	7	73	49				100	42	35	-	50
	1.18	1	5	51	26				99	29	27	-	42
	0.600	1	4	35	16				98	20	18	-	32
	0.300	1	4	24	11				97	15	11	-	23
	0.150	1	3	17	7				96	11	7	-	16
	0.075	0.3	2	12	5				95	7.8	4	-	10
BRD	[B 14 + 15]	2 652	2 668	2 656	2 656				2 595	2 658	N/A	-	N/A
Sand eqv. / ekw .	[B 19]			50.0	66.2					52	50	-	Min
Water absorp.	[B 14 + 15]	0.5	0.4	0.9	0.9					0.7	1.0	-	Max
ACV / AVW	[B 1]	11.5											
10 % FACT / FAVV	[B 1]	330.0											
Flak. Index / Plath. Index	[B 3]	16.3											
PSV / KPW	[848]	54.0											
Methylene Blue	[ ]			0.40	0.50								
Fine Agg. Angularity	[ ]			44.8	47.0								

Filler / Vulstof

	Type / soort	% in mix / meng
1	HYDRATED LIME	1.0%
2		
3		

Notes / Notas

Sifgrootte volgens log skaal Sieve size to log scale

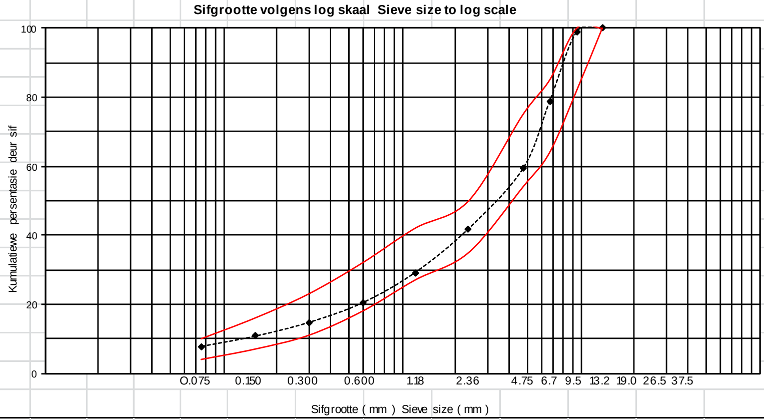


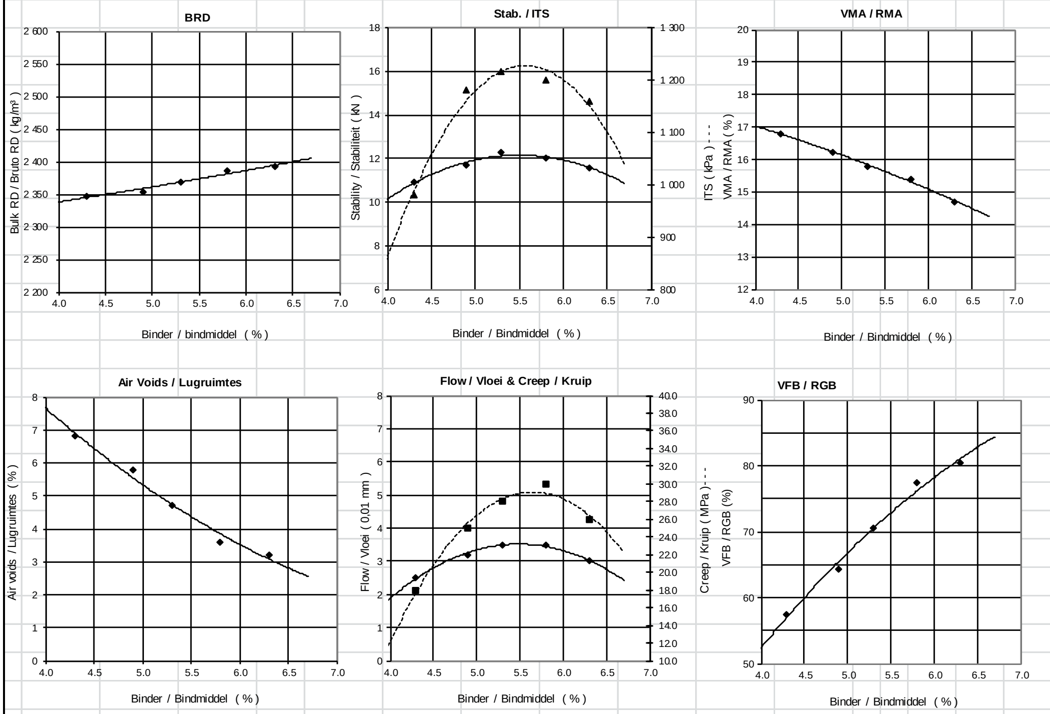
Figure 39. Example Mix Design

		Form Vorm	
Contract / Kontrak :		**D 3**	
Route / Roete :			
Section / Gedeelte :		(03 / 1993)	
Layer / Laag :	SURFACING	Sheet no. / Vel no.	
Mix / Mengsel :	AC MEDIUM 9.5 Max	1 of / van 1	

Binder / Bindmiddel

Type / Tipe	60/70 Pen Bit	Source / Bron	Engen
Pen @ 25°C (0,1mm)	65	Visc. @ 60 °C (Pa.s)	134
Soft. / Sagw. Pt (°C)	48	Visc. @ 165 °C (Pa.s)	0.3
RD	1.023	Hot Flow/Warmvloei (mm)	
Temp. (Mix / Meng) (°C)	150	Temp. (Comp. / Verdig.)	135

Design curves / Ontwerpkurwes



Design / Ontwerp							Specification	Proposal / Voorstel	
Binder / Bindmiddel	%	4.3	4.9	5.3	5.8	6.3	N/A	VIM	Binder
Max. / Maks. TRD (Rice)	kg/m³	2 518	2 499	2 487	2 475	2 470	N/A	RIM	Bindmiddel
BRD (Marshall)	kg/m³	2 347	2 354	2 369	2 386	2 392	N/A	%	%
Voids in Mix / Ruimtes in Mengsel	%	6.8	5.8	4.7	3.6	3.2	3.0 - 6.0	**4.5**	**5.4**
Stab. (Marshall)	kN	10.9	11.7	12.3	12.0	11.6	8.0 - 18.0		
Flow / Vloei (Marshall)	mm	2.5	3.2	3.5	3.5	3.0	2.0 - 6.0		
Stab. / Flow Ratio / Stab. / Vloei Verh.		4.4	3.7	3.5	3.4	3.9	2.5 Min		
ITS	kN	980	1 180	1 215	1 200	1 160	1000 Min		
Immersion / Dompel Ind.	%	87.0	91.4	97.5	90.2	88.0	75 Min		Binder
VMA / RMA	%	16.8	16.2	15.8	15.4	14.7	15 Min	Lane	Bindmiddel
VFB / RGB	%	57.5	64.3	70.5	77.4	80.4	65 - 75	Laan	%
Creep / Kruip [Dyn. / Stat.] *	Mpa	18.0	25.0	28.0	30.0	26.0	20 Min	**All**	**5.4**
Air Perm. / Lugdeurlaatbaarheid (	cm³			0.08			$1x10^{-8}$ Max		
Resillient Modules	Mpa			3 726			3000.0		
Filler Binder Ratio		1.4	1.3	1.2	1.1	1.0	1.0 - 1.5		
Absorp. (Binder / Bindmiddel)	%	0.312	0.379	0.433	0.552	0.795	0.5 Max		
Film thickness / Film dikte	µm	7.0	7.7	7.4	8.2	8.5	5.5 - 8.0		

* Delete if not applicable / * Haal deur indien nie van toepassing

Figure 39. Example Mix Design (continued)

10.9 Compaction

Achieving adequate compaction is critically important to ensure satisfactory performance of an asphalt layer. Compaction accomplishes two important goals:

- Develops **strength and stability** through aggregate lock-up
- Partial closing of **air and water passages**

Good compaction, therefore, ensures:

- Satisfactory structural and functional **performance**
- **Durability**
- Protection of underlying layers against the **ingress of water**
- Prevention of **oxidative hardening** (ageing) of the binder

Poor or inadequate compaction may result in the following adverse effects:

- Excessive **permeability and oxidative hardening,** leading to premature ageing, which may lead to premature fracture and ravelling of the asphalt.
- **Rutting** in the wheel tracks as a result of further compaction of the layer under traffic.
- Insufficient **long term traffic compaction** and kneading due to premature ageing, permeability and brittle distress.

Chapter 12: 3.11 covers construction aspects of hot mix asphalt. Important construction aspects include the priming of the substrate, as well as the application of a tack coat. The priming of the granular base or subbase is necessary to bind the surface, provide protection against scuffing by construction traffic and to assist in preventing the ingress of rain water. The main purpose of the prime is to penetrate into the granular layer. It is not designed to provide adequate adhesion between the base or subbase and the asphalt layer, a tack coat is applied for this purpose.

The application of a tack coat is always recommended, either on a primed surface, or between layers of asphalt. The tack coat ensures a good bond between the base or subbase, as well as between asphalt layers; the lack of proper bonding allows slippage during compaction of the asphalt layer and during the pavement's service life, resulting in shoving, particularly in areas where traffic brakes or accelerates, such as at intersections. Prime and tack coats are discussed in Section 7.

Further guidance on the use of tack coats is provided in Sabita Manual 5.

Criteria for, and process control and quality management, for the compaction of asphalt surfacing and base layers are covered in Chapter 3: 4.2, Chapter 12: 3.11 and Chapter 13: 6.

11. SPRAY SEALS AND MICRO-SURFACINGS

The main functions of a surfacing are to:

- Provide a **waterproof cover** to the underlying pavement structure.
- Provide a **safe all-weather**, dust free riding surface with adequate skid resistance.
- **Protect the underlying layer** from the abrasive forces of traffic and the environment.

Appropriately selected and constructed spray seal types have been successfully applied over decades to address these primary needs. Micro surfacings may, when appropriate, be utilised to improve the riding quality by smoothing out small irregularities, or to fill traffic induced ruts. This improves safety by reinstating the surface cross-drainage profile.

Since most spray seals and micro-surfacings are relatively thin, they have no load distribution properties. As such, it is important to remember that any non-conforming or distressed areas in the underlying substrate layers must be corrected or repaired prior to placing a new surfacing. Failure to do so will result in the distress manifesting on the road surface very quickly.

The selection of the most appropriate surfacing type should be based on the most cost-effective option able to accommodate the expected traffic, the associated horizontal and vertical induced stresses, as well as the requirements listed above. There may, however, be other factors that dictate a relatively more expensive option, such as limited construction capacity or particular expected climatic conditions at the time of construction. It is thus important that all constraints or risks be identified and assessed prior to making the final selection of seal type.

The construction of seals is discussed in Chapter 12: 3.10, and the associated quality management in Chapter 13: 7.

11.1 Spray Seals

Approximately 80% of South Africa's surfaced roads are sealed with a spray seal, either as the initial surfacing or as a reseal. In its simplest form, a spray seal consists of a coat of bituminous binder sprayed onto the road surface, which is then immediately covered with a layer of aggregate, rolled and broomed, to ensure close contact and thus good adhesion between the aggregate and the binder film.

There are a number of seal types available, each having specific advantages and/or disadvantages. It is thus important to understand the limitations of each seal type in order to select and design the most appropriate one. The various seal types commonly constructed are illustrated schematically in Figure 40. These figures are from TRH3: Design and Construction of Surfacing Seals.

The selection of the most appropriate spray seal type to ensure adequate long-term performance is influenced by a number of factors, such as:

- **Pavement structure** and condition
- Existing **substrate**
- **Traffic spectrum**
- **Traffic movement**, e.g., lateral forces at intersections
- Road **geometry**
- Design **speeds**
- Available **materials**
- **Pre-treatment** and repair constraints
- **Construction** and supervision capacity
- **Maintenance** capacity
- Physical and social **environment**
- **Climatic conditions** at the time of construction

References for Seals

- TRH3: **Design and Construction of Surfacing Seals**. Available for download on www.nra.co.za.
- For seals using modified binders: TG1: **The Use of Modified Bituminous Binders in Road Construction**. Published by the Asphalt Academy. Available for download on www.asphaltacademy.co.za.
- Further **useful information** is available on the Australian website: www.arrb.org.au/sealing/index. This site was established to assist in the dissemination of good, as well as innovative, international practice in the field of spray seals.
- For slurry seals, SABITA Manual 28: **A Guide on the Design and Use of Slurry Surfacings**. Available for download at www.sabita.co.za.

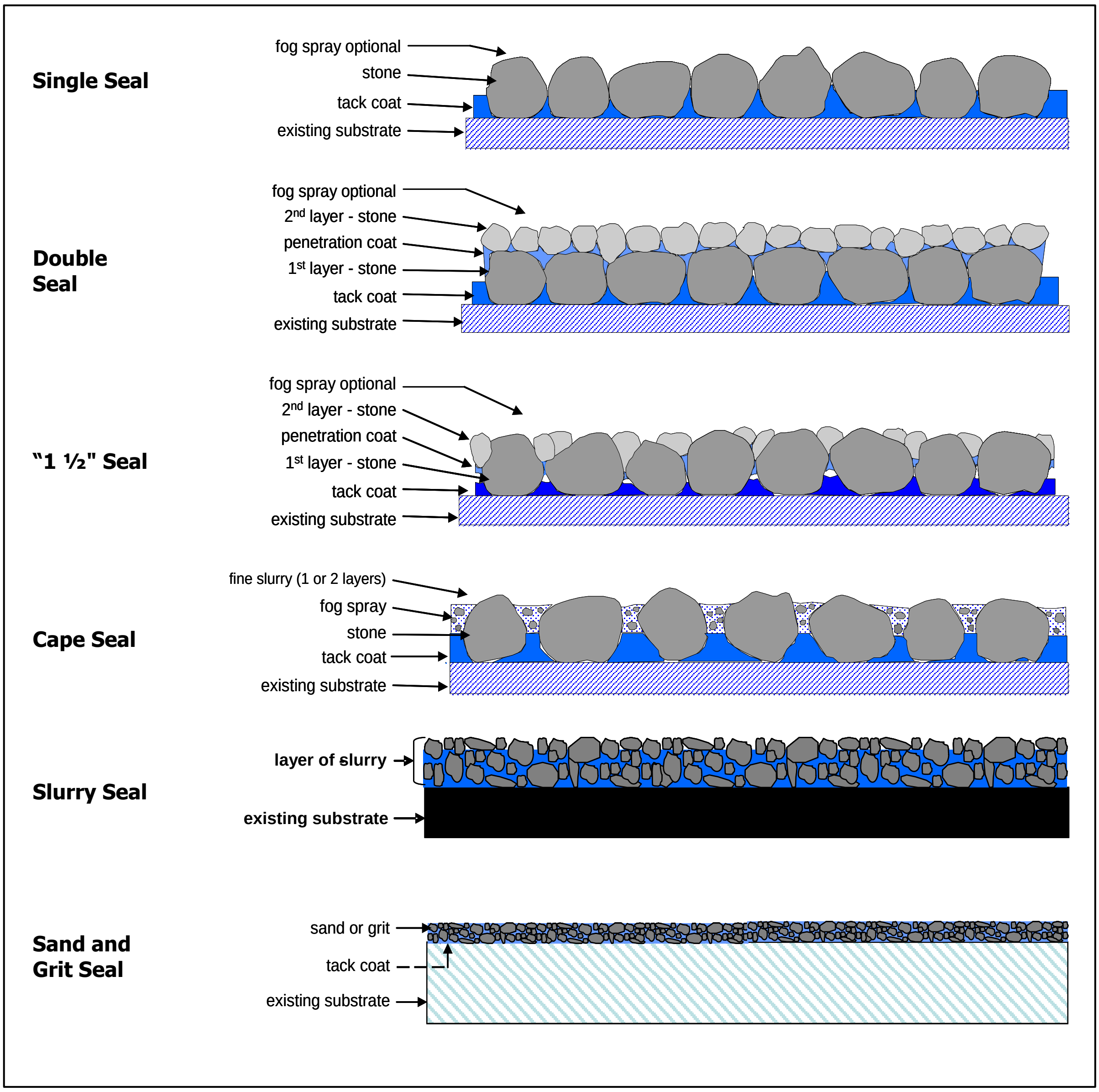


Figure 40. Schematic Illustration of Seal Types

The introduction of various modified binders into the industry over the last decade or two has significantly extended the upper traffic spectrums that were traditionally associated with surface seals. It is, however, important to note that the use of modified binders create other constraints and risks that need to be addressed. Due to the increased viscosity characteristics of modified binders, they generally have significantly reduced aggregate "wetting" characteristics. This could greatly reduce the ability of the binder to retain the aggregate chippings, especially in the following conditions:

- **Cold and/or wet weather** experienced soon after construction and early opening to traffic, particularly with single seals.
- **Significant delay** between spraying of the binder and application of the aggregate chips and subsequent rolling, resulting in weak binder and aggregate bonding.

In addition, the increased stiffness of modified binders compared to conventional binders increases the risk of entrapping moisture that may be present in the underlying substrate layers, as a result of unsealed cracks or a permeable gravel or asphalt base. This entrapment of moisture has been proven to strip the bitumen from the underlying asphalt layer, causing an unstable base layer or even stripping of the seal above any cracks. It is thus important that, if for any reason, a modified binder is required, every effort must be made to mitigate all of the potential risks. Extensive information and guidance on the use of modified binders is available in TG1: The Use of Modified Bituminous Binders in Road Construction.

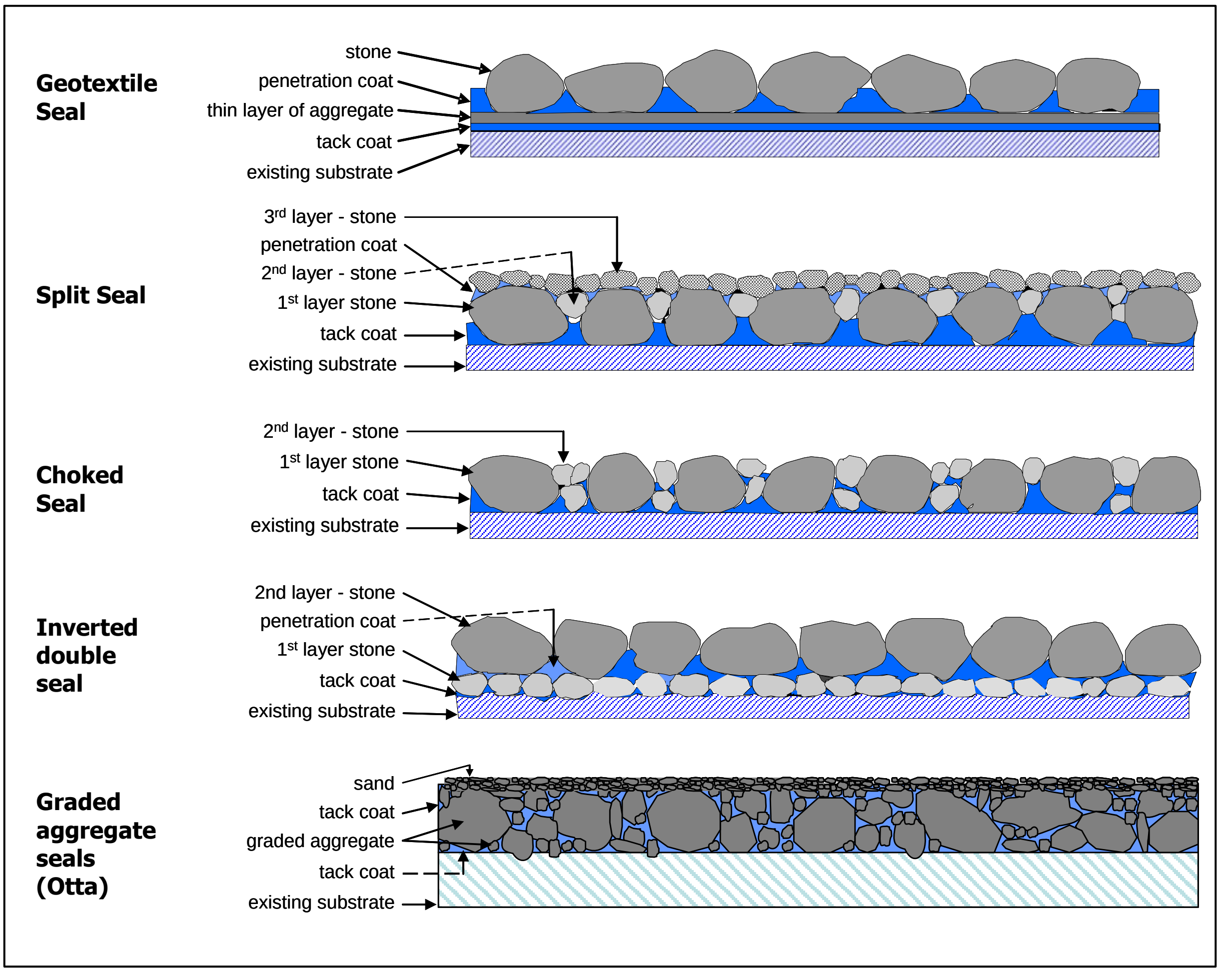


Figure 40. Schematic Illustrations of Various Seals (continued)

Before 1998, the design of spray seals was undertaken in accordance with the method and requirements of the specific provincial road authority for which the project was being undertaken. These different design methods, which were invariably empirically based, often resulted in significant differences in calculated binder application rates for similar application conditions. Since then, significant progress has been made in the development of a unified rational seal design method, which is applicable nationally, and also covers the addition of numerous generic modified binders.

TRH3 contains extensive guidelines for design. It is important to note that TRH3 focuses only on spray seals for routine applications. The design of spray seals for unique situations, or where exceptionally high traffic is expected, requires specialist knowledge and experience.

It is also important that a distinction is made between an initial surfacing on a new road and a reseal on an existing road, as different needs or requirements may need to be addressed. The recommended process for selecting the most appropriate seal is as follows:

- Obtain all **relevant information** necessary from a pre-design investigation.
- If necessary, divide the road into **uniform sections** of similar existing condition and required characteristics.
- Identify **appropriate surfacings** for each situation.
- Compare **initial and life cycle costs**.
- Compare the **influence of other factors**, such as available materials.
- **Final selection** of seal type.

TRH3 contains detailed guidance on the selection of the most appropriate seal type for a particular circumstance. In addition, further useful information is available on the Australian website: www.arrb.org.au/sealing/index. This site was established to assist in the dissemination of good, as well as innovative, international practice in the field of spray seals.

11.2 Slurry Seals/Microsurfacings

Slurry seals or microsurfacings are defined as a cold batched and applied mixture consisting essentially of the following:

- Bituminous emulsion
- Crusher sand
- Cement or lime
- Water
- Additives, such as polymer modifiers and/or bitumen rubber additives

> **Testing of Component Materials**
> Information on the appropriate standards and testing of the component materials is covered in Chapter 3: 4.4.2 and Chapter 4: 4.4.2.

Slurry seals have been extensively used for decades in the following applications:

- New construction
  - Cape seals
  - Thin overlays (< 6 mm)
  - Slurry Bound Macadam
- Maintenance
  - Texture treatments (1 mm to 3 mm)
  - Cape seals at intersections
  - Slurry seal overlays (< 6 mm)
  - Edgebreak repairs
  - Surfacing repairs
  - Cover for geotextile patches

As a result of global environmental concerns, the use of cold applied micro-surfacings is becoming more common for the following applications:

- **Medium overlays**: 6 mm to 8 mm
- **Thick overlays:** 8 mm to 15 mm
- **Rut filling:** < 30 mm

An example of the application of a slurry is given in Figure 41. The slurry is coloured brown, which indicates the emulsion has not yet broken.

Figure 41. Application of a Slurry

The selection of the most appropriate cold applied product is highly dependent on the specific application to be addressed. In addition, whether the product is applied by machine or labour-enhanced methods also influences the selection. Over and above the normal issues surrounding slurries, such as available aggregate type, grading and maximum particle size, recent advances in bitumen emulsion technology has resulted in numerous options now being available to address particular application conditions and/or constraints.

Due to the risk of settlement and/or segregation, slurries and microsurfacings should not be placed in single layer thicknesses exceeding 1.5 times the maximum aggregate size.

Slurry Layer Thickness

Due to the risk of settlement and/or segregation, slurries and microsurfacings should not be placed in single layer thicknesses exceeding 1.5 times the maximum aggregate size.

11.2.1 Design Methodologies

The design of a slurry mixture should be such that:

- It has **creamy consistency,** allowing it to be easily applied by hand or mechanical spreaders.
- The hardened product has **sufficient binder** to not ravel, but **stable** enough to carry the applied wheel loads without bleeding or deformation.

To date, there has not been a single unified approach to the design of slurries and microsurfacings in South Africa. There are however a number of semi-empirical methods used by practitioners based largely on personal experience and developed to address specific applications. For example, a different design approach may be used for a texture treatment slurry than for a medium microsurfacing.

There are three common slurry design methods:

- **Method A**
  - Marshall briquettes prepared at various binder contents with 70/100 penetration grade bitumen.
  - Specimens compacted 30 blows/side at 130 – 135 °C.
  - Determination of void contents, film thickness, Marshall Stability and Flow.
- **Method B**
  - Slurry prepared at various binder contents and allowed to air dry.
  - Material placed in oven at 135 °C.
  - Marshall briquettes compacted at 75 blows/side at 135 °C.
  - Determination of void content, Marshall Stability and Flow, Indirect Textile Strength and Creep Modulus.
- **Method C** (Practical Site Method)
  - Slurry mixes prepared at various emulsion contents (180 – 240 ℓ/m^3).
  - Slurry mixes poured into Marshall briquette moulds to a thickness of approximately 15 mm.
  - Mould placed in oven heated to 60 °C to allow water evaporation.
  - Samples compacted with a Marshall hammer for 150 blows on one side only.

The design is decided by ensuring minimum void contents and film thickness, and optimising the stability and flow. The decision process is an engineering optimisation, and relies heavily on experience.

The ideal binder content results in:

- **No plastic deformation** at the edges between the hammer and the side of the mould.
  - **Dark grey to black coloured specimens,** which are slightly pliable when broken in two.

Note that when the Marshall approach is used, the optimum binder content is established on a compacted sample. However, all slurry seal calculations are based on uncompacted material. For example, if the optimum bitumen content is determined as being 8% at a density of 2415 kg/m^3, then the quantity of residual binder would be 193.2 ℓ/m^3 compacted. If the material was loose and a bulking factor of 1.4 was applied, the quantity of residual binder in 1 m^3 would be 138 ℓ, i.e., 193.2 ÷ 1.4. This would result in 230 ℓ of 60% emulsion per m^3 of loose aggregate.

Table 26 may be used as a guide for the most appropriate binder content for slurries for various applications. The most appropriate emulsion content is highly dependent on the aggregate grading (particularly the fines content), aggregate type, and aggregate relative density.

Further methods for the design of these products are available from ASTM as follows:

- ASTM D39010-98: Standard Practice for the Design, Testing and Construction of **Slurry Seals.**
- ASTM D6372-99a: Standard Practice for the Design, Testing and Construction of **Micro-Surfacings.**

For detailed information on the selection and design of these products it is recommended that the guidelines and processes contained in SABITA Manual 28, "A Guide on the Design and Use of Slurry Surfacings" are followed.

Table 26. Guidelines for Appropriate Slurry Binder Contents

Application	Aggregate Grading from the Standard Specifications	Emulsion Type	Emulsion Content (ℓ/m³)
Slurry texture treatment	Fine (fine) Texture	Anionic Stable 60 Cationic Stable 60	190 – 210
Thin slurry (4 – 6 mm)	Fine (fine)	Anionic Stable 60 Cationic Stable 60	250 – 260
Medium slurry (6 – 8 mm)	Fine (medium)	Anionic Stable 60 Cationic Stable 60	230 – 250
Coarse slurry (8 – 12 mm)	Fine (coarse)	Anionic Stable 60 Cationic Stable 60	210 – 230
Slurry Cape Seal	Fine (medium 1st layer) Fine (fine 2nd layer)	Anionic Stable 60 Cationic Stable 60	220 – 240
Microsurfacing overlay (6 – 8 mm)	Fine (medium)	Modified	180 – 190
Microsurfacing overly (8 – 12 mm)	Fine (coarse)	Modified	170 – 180
Microsurfacing rut filling (8 – 30 mm)	Fine (coarse)	Modified	160 – 170

12. CONCRETE

Concrete is one of the most commonly used manufactured construction and building materials. There are a number of reasons for this:

- The **raw materials** required to make cement and concrete are fairly abundant in most countries and concrete, with a few exceptions, is produced locally.
- Concrete has infinite variability in terms of the **strengths, densities and colours** that can be produced. As it is produced in a wet, plastic form, it can be moulded into virtually any shape.
- Concrete can also be **designed to last** as long as is required.
- In transport and pavement engineering, concrete can be used in both **structures and services** and in the **pavement**.

This section contains an overview of the uses of concrete in roads, especially concrete pavements, and the requirements and considerations for the design and assessment of such mixes. "Fulton's Concrete Technology" (2009) and "Concrete Road Construction" (2009) should be consulted for more information and guidance. The reader is also referred to Chapter 3: 5.1 and Chapter 4: 5.1 for Materials Testing and Standards for concrete mix constituents, Chapter 10: 8 for Pavement Design, Chapter 12: 3.12 for construction, and Chapter 13: 8 for quality management.

References for Concrete

- **Fulton's Concrete Technology**. 2009. 9th Edition, Cement & Concrete Institute.
- **Concrete Road Construction**. 2009. Cement and Concrete Institute (C & CI).
- **A Guide to the Common Properties of Concrete.** 2009. Cement and Concrete Institute (C & CI).

12.1 Concrete for Structures

As concrete is such a versatile and durable material, it finds wide application in structures, other infrastructure and furniture in pavements. Concrete can be used in wet form to cast structures on site, or in precast form, manufactured either on or off site.

- **In situ concrete**. Concrete can be cast in situ to construct bridges, culverts, side drains, kerbs, manholes, median and other barriers as well as foundation bases for lighting masts or road signs. In the case of bridges, in situ concrete is usually used for the construction of abutments, support columns and decks. There are a large number of techniques for constructing bridge decks including in situ, launched and balanced cantilever. An example of an incremental in situ bridge construction is shown in Figure 42.
- **Precast concrete.** Concrete can also be used in precast form to provide stormwater pipes, culverts, kerbing, median and other temporary or permanent barriers, bridge beams and bridge decks. These can either be purchased from a commercial supplier or cast in a precast yard on site. A bridge construction using precast concrete is shown in Figure 43.

Figure 42. In Situ Incremental Bridge Construction


Figure 43. Precast Concrete

12.1.1 Material Usage, Mix Design and Application

Fulton's (2009) provides a detailed reference document covering the following six broad categories of concrete technology:

- **Materials** for concrete
- **Properties** of fresh concrete
- **Properties** of hardened concrete
- **Design** of concrete mixes
- **Production** of concrete
- Special **techniques and applications**

Guidance is provided on material choice and mix proportioning to provide required properties such as strength, durability and stiffness, and, on how to manufacture, transport and place concrete to provide good durable structures. An additional publication, "A Guide to the Common Properties of Concrete", C & CI (2009) provides further information.

Concrete Pavements in SAPEM

Various aspects of concrete pavements are discussed in:

- Chapter 2: **Pavement Composition and Behaviour**, Section 2.4
- Chapter 3: **Materials Testing**, Section 5.1
- Chapter 4: **Standards**, Section 5.1
- Chapter 10: **Pavement Design**, Section 8
- Chapter 12: **Construction Equipment and Method Guidelines**, Section 3.12
- Chapter 13: **Quality Management**, Section 8
- Chapter 14: **Post Construction**, Section 4.2

12.2 Concrete for Pavements

12.2.1 Requirements for Concrete Pavements

The function of a concrete pavement, as with any pavement, is to support traffic loadings and to meet the required performance requirements of that road over its design life. To achieve this, the pavement must:

- **Enable the stresses** in the subgrade caused by traffic loads to be maintained at a level that the subgrade can sustain, without the development of cumulative permanent deformation, and without the development of elastic strains in the subgrade of a magnitude that causes deterioration within the pavement.
- **Accommodate stresses** developed within the concrete pavement slab, caused by drying shrinkage and differential temperature and moisture conditions within the concrete base.
- Designed and constructed so that the effects of **subgrade moisture variation,** which can cause subgrade volume and strength changes, are limited.

Dowels in Plain Jointed Pavements

It is recommended that dowels are always placed in plain jointed pavements to reduce faulting. Some existing jointed pavements are being retrofitted with dowels to improve their performance.

12.2.2 Concrete Pavement Types

In South Africa and overseas, a number of concrete pavement types have been constructed, and are discussed below. The following types, which differ only by the crack control criteria, are the most common concrete road pavements in South Africa.

- **Jointed unreinforced** (plain) concrete
- **Continuously reinforced** concrete (CRCP) pavement

- **Ultra-thin** concrete pavement (UTCP)

(i) Jointed Unreinforced (Plain) Concrete Plan Pavement

Plain concrete pavements, illustrated in Figure 44, contain no reinforcement except in special situations such as bridge approach slabs, small irregular areas with curved or angled edges or mismatched joints. Tie bars, placed to hold longitudinal joints closed, and dowel bars, used to transfer loads across transverse contraction joints, are not regarded as reinforcement. Transverse contraction joints are placed at relatively short intervals. A large proportion of the existing concrete roads in South Africa are of this type. Some of these pavements contain dowels and others do not. Current research and performance indicate that the inclusion of dowels in jointed plain pavement is cost-effective due to improved performance of the pavement and a reduced incidence of faulting. Dowel bar installation during construction is shown in Figure 45.

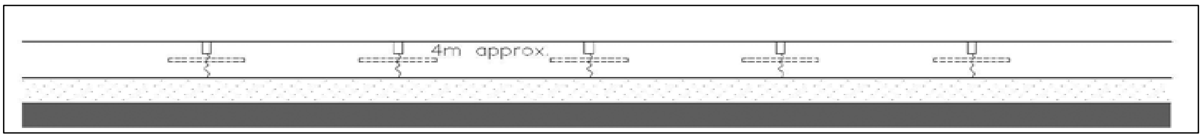


Figure 44. Jointed Unreinforced (Plain) Concrete Pavement, with or without Dowels

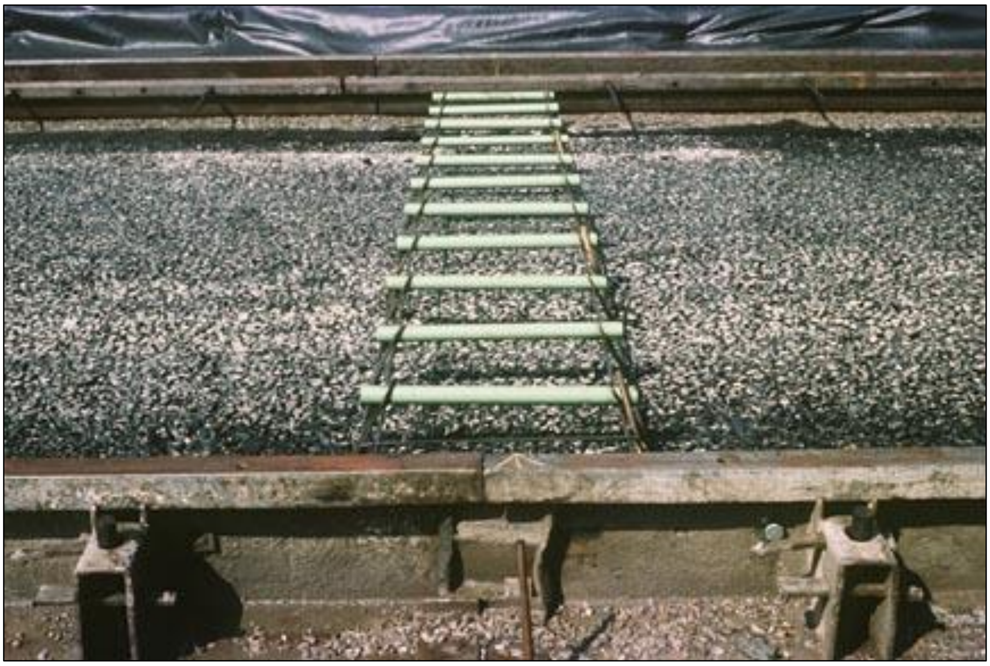


Figure 45. Installation of Dowel Bars

(ii) Continuously Reinforced Concrete (CRC) Pavement

The main feature of a continuously reinforced concrete road pavement, illustrated in Figure 46 and Figure 47, is that the longitudinal reinforcement is continuous for the entire length of the pavement; the need for transverse control joints is consequently effectively eliminated. As shrinkage develops in the pavement, fine randomly spaced cracks about 1 mm wide develop. These cracks are prevented from opening by reinforcement and subbase friction. Experience has shown that the preferred crack spacings are in the range 1 to 2 meters. To obtain this crack spacing, the necessary longitudinal steel percentage is between 0.5 and 0.7% of gross cross sectional area. Even with this higher amount of steel, the purpose of the reinforcement is still to control cracking. The principal benefit of this pavement type is the elimination of transverse contraction joints and their associated maintenance. Disruption costs attributable to maintenance operations are therefore minimised. For heavily trafficked, limited access roads this can be an important factor in pavement type selection as one of the reasons for having to close a trafficked lane, even if temporarily, is eliminated. Current experience has shown that although this pavement type contains more reinforcement than a jointed reinforced pavement, the absence of dowels and of the need to form and seal joints minimises the construction cost premium. This pavement type is being increasingly used in South Africa for the construction of overlays and inlays of existing distressed asphalt and concrete pavements. The thickness of CRC pavements is generally slightly less than that of jointed pavements, and is in the order of 180 to 230 mm.

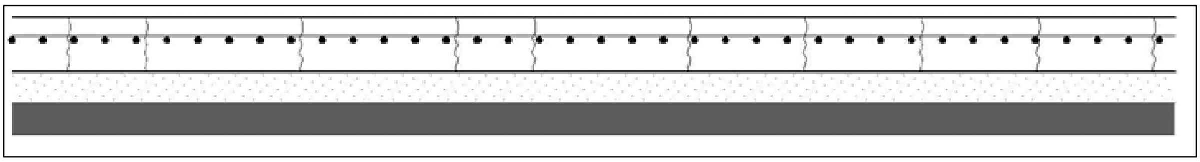


Figure 46. Continuously Reinforced Concrete Pavement

Figure 47. Constructing a Continuously Reinforced Concrete Pavement

(iii) Prestressed Concrete Pavement

Although the concept of prestressed concrete pavement was developed in the 1920s, no significant further development occurred until the late 1940s. The claimed benefits are: few if any joints; no cracking under normal loads; control of thermally induced warping stresses; reduction of thicknesses; flexibility; and use of the tension stress zone. The thickness can be significantly less than that of a jointed pavement but the cost is often a lot higher. As a result of this reduced thickness, particular care should be taken with the supporting layers to reduce excessive deflections.

(iv) Steel Fibre Reinforced Jointed Concrete Pavement

In a steel fibre reinforced jointed concrete pavement, illustrated in Figure 48, a small quantity of short steel fibres is distributed throughout the concrete. These steel fibres produce a significant increase in the flexural strength and fatigue resistance of the concrete, and therefore, a reduction in pavement depth. Trial sections of steel fibre reinforced concrete road pavement have been constructed in Gauteng and their performance is being monitored. They are considered a good alternative for unusual shapes such as roundabouts, where an even joint distribution is difficult. The reduction in depth possible in this type of pavement needs to be balanced against the increased cost of the concrete.

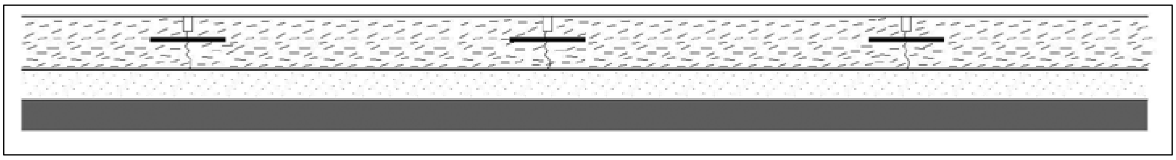


Figure 48. Fibre Reinforced Jointed Concrete Pavement with or without Dowels

(v) Ultra-Thin Concrete Pavement (UTCP)

Two types of ultra-thin concrete pavements are currently used in South Africa for rehabilitation. On heavily trafficked roads, ultra-thin continuously reinforced concrete pavements are used. For low-volume township streets, ultra-thin reinforced concrete pavements are constructed.

- **UTCRCP for Heavy Duty Pavements.** The main features of an ultra-thin continuously reinforced concrete road pavement, illustrated in Figure 49, are:
  - Concrete layer is very **thin**, in the order of 50 to 70 mm.
  - Very **high strength** cement paste is used.
  - Strengths are in excess of **80 MPa**.
  - Both **steel and plastic fibres** are used. The longitudinal steel percentage is of the order of 3.0 %.

  This type of pavement has been extensively researched using the Heavy Vehicle Simulator (HVS) and has carried up to 90 million Equivalent Standard Axles (E80's) (Kannemeyer et al, 2008). Due to constructability issues and the risk of buckling, the recommended thickness tends to be closer to 70 mm, and lower strength concrete is used.

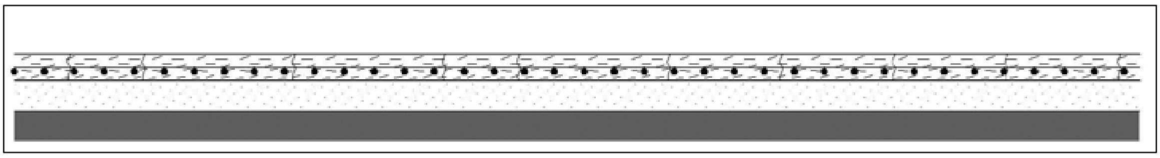


Figure 49. Ultra-Thin Continuously Reinforced Concrete Pavement

UTCRCP for Heavy Duty Pavements

The benefit of using UTCRCP is that, due to the concrete and reinforcement, it is a long lasting layer requiring little maintenance. In addition, it can be placed as an overlay. Because of the thickness, only a thin layer of the existing pavement needs to be milled off, which is advantageous for vertical clearance under bridges and overpasses, and to utilise the existing strength in the pavement.

In South Africa, UTCRCP has been constructed on several projects:

- **Heidelberg Traffic Control Center** on the N3 in 2007, where the HVS testing was done.
- **N1 near Cape Town** in 2010. On this project, extreme temperatures exceeding 40 °C were experienced, which caused buckling. This has been resolved by providing edge restraint.
- **N2 Tongaat Toll Plaza** in 2011. The pavement at the plaza is wide to accommodate the many lanes, and was constructed without joints. This resulted in buckling longitudinally.
- **N12 near Benoni and Boksburg,** completed in 2012. This project is approximately 19 km on a heavily trafficked road with two lanes in each direction. The UTCRCP is overlaid with an asphalt UTFC for noise and skid resistance.

The UTCRCP was 50 mm thick for all these projects. The main challenge with UTCRCP is the attention to detail and careful quality control during construction, especially at day joints. To mitigate some of these problems, a thickness of 60 to 70 mm is now recommended.

- **UTRCP for light duty pavements.** The main features of an ultra-thin reinforced concrete road pavement (Figure 50) for this application are:
  - Concrete layer is very thin, in the order of 60 mm.
  - Conventional concrete is used.
  - Longitudinal steel percentage is in the order of 0.3%. No fibres are used.

  This type of pavement has been extensively researched at the Roodekrans test site using actual truck traffic, and carried up to 1 million Equivalent Standard Axles (du Plessis, et al, 2009; Steyn et al, 2005).

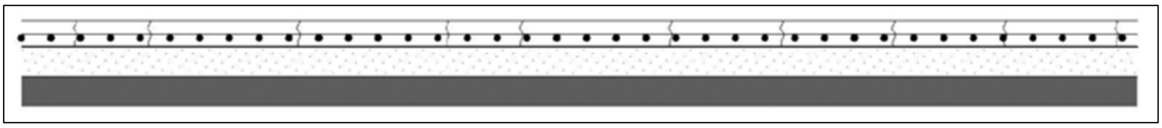


Figure 50. Ultra-Thin Reinforced Concrete Pavement

12.3 Requirements for Paving Concrete

12.3.1 Fresh Concrete

The properties of the fresh concrete influence the surface finish and the riding quality of the completed pavement. The primary properties of fresh concrete for paving include:

- **Place ability**. The fresh concrete must be able to be placed with the equipment utilised. This may include placing by either slipform or side form equipment, or even hand placing.
- **Flow ability**. The ability of the concrete to flow under vibration is assessed in a pan mixer by forming a mound of concrete on one side of the pan. A rectangular steel float is held on its side against the heap of concrete into which a small poker vibrator is inserted. Flow ability is indicated by the amount of concrete flowing around the ends of the trowel.
- **Work ability**. The concrete must be workable enough to ensure full compaction under vibration, but compacted concrete must remain sufficiently rigid to resist flow on steep grades and cross falls. The slump test (shown in Figure 52) is used as an indicator of workability by pushing the standard tamping rod through the compacted concrete in the slump mould. The ease with which this can be done indicates workability or stoniness of the mix. Also, by observing the shape of the slumped concrete after the slump mould has been removed, shows the workability. If the concrete in contact with the base plate retains its diameter, poor workability is indicated. The degree of spreading of this zone is an indication of workability.
- **Consistency.** This is the stiffness or sloppiness of the fresh concrete and is measured with the slump test for the normal range of consistence. Establishment of the optimum consistence of the concrete mix is critical, as this has a major impact on the final surface finish and riding quality of the pavement.
- **Cohesiveness**. Cohesiveness is the tendency of fresh concrete to resist segregation, and is assessed by tapping the base plate used in the slump test after the slump has been measured, while the concrete is still in position. A cohesive mix settles gradually without the moulded concrete falling apart.
- **Bleeding**. Bleeding is the separation of water from the water/cement paste. Water is seen on the surface of freshly placed concrete, illustrated in Figure 53. Although bleeding or settlement reduces the net water:cement ratio in the concrete it has the following disadvantages:
  - Water trapped under large aggregate particles, tiebars and dowels, causes zones of weakness in the concrete when it hardens.
  - Water trapped under horizontal reinforcing bars reduces the bond between concrete and steel.
  - If the settlement of the solid particles is obstructed, e.g., by horizontal steel, especially if it is near the top of the concrete, cracks may develop in the concrete before it hardens.
  - Severe bleeding can adversely affect the abrasion resistance of the surface concrete. This may result in premature loss of texture in service.

  The propensity for bleeding to occur is assessed by compacting the concrete in a cube mould to slightly below the top surface and observing the degree of bleeding. Concrete that bleeds very little is, however, vulnerable to plastic shrinkage cracking when placed in hot, dry, windy conditions.
- **Dowel and tiebar insertability**. Where dowels and tiebars have to be inserted in the pavement, as illustrated in Figure 51, this should be possible without undue effort and without disturbing the uniform distribution of coarse aggregate in the mix. This is important to ensure good riding quality. A simple assessment is to take a short length of bar or a piece of coarse aggregate and attempt to push it into compacted concrete in a cube mould while it is still being vibrated on a vibrating table. It should be possible to push the bar or stone into the concrete with minimal effort. A mortar plug should not be left above the bar or stone.

Figure 51. Inserting Dowel Bars

- **Texturability**. During compaction, sufficient mortar should be worked to the surface to obtain the required pavement texture. A tined texture requires a thicker layer of mortar than a broomed texture (See Chapter 12: 3.12.9). The use of a spring-steel tine or broom bristles on the surface of a compacted and floated concrete cube gives an indication of whether sufficient mortar is available. This mortar layer should, however, not be excessive because this increases the risk of plastic shrinkage, drying shrinkage cracking and may promote surface wear.

Figure 52. Slump Test

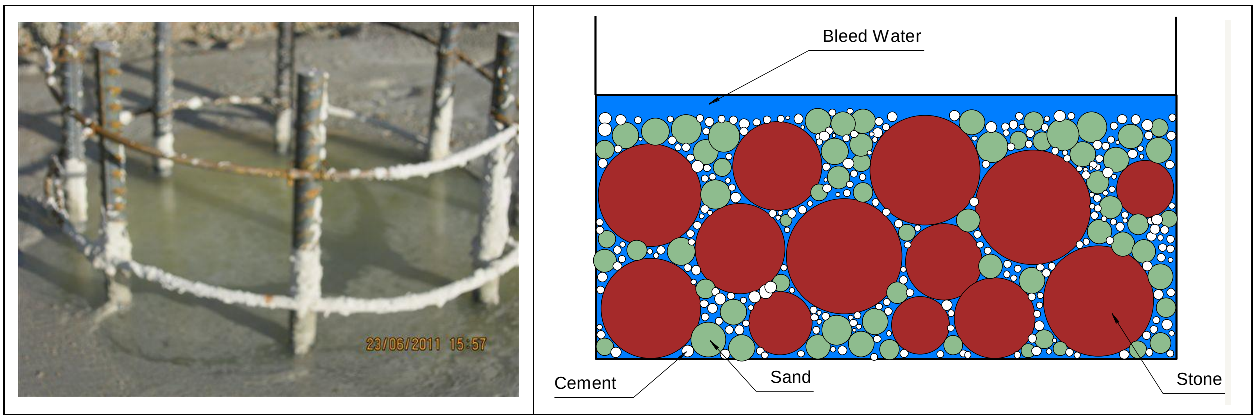


Figure 53. Bleeding

12.3.2 Hardened Concrete

Several properties of hardened concrete are important, including saw ability, strength and shrinkage.

(i) Saw Ability

The saw-ability of concrete at early ages is affected by the strength gain of the concrete and the type of aggregate. The earlier the contraction joints are sawn, the lower the risk of random cracking. Where it is necessary to assess the effect of various binder types, admixtures or aggregate types, the following method of assessment can be used.

- A number of **concrete cubes** are cast for testing at various ages, one cube for each of 8, 10, 14, 16, 18, 20, and 24 hours. One or two extra cubes are cast for sawing.
- At each of the allotted times, a **cube is crushed** to determine the compressive strength and a **cut** is made, using a diamond saw, to a depth of 25 to 40 mm in one of the cubes set aside for sawing. The amount of ravelling and plucking is observed. The test method is SANS 3001– CO11 (see Chapter 3: 5.1.8.2).
- The process is continued until a cut with an **acceptable degree of ravelling** is obtained.
- This assessment can also be carried out with the cubes stored at **different temperatures**.

An indication of the effect of aggregate type on saw cutting is given in Table 27.

Table 27. Effect of Aggregate Type on Saw Cutting

Aggregate Type	Strength when Acceptable Cut Achieved
Granite, Quartzite	3 – 5 MPa
Dolerite, Andesite	4 – 6 MPa
Felsite	> 8 MPa

(ii) Strength

Most specifications require that the compressive and flexural strengths are determined at three water:cement (W/C) ratios. Compressive and flexural strength tests are done in accordance with SANS 3001-CO11 and CO12, respectively (see Chapter 3: 5.1.8.2). With increased cement strengths, and with the use of continuously graded mixes and high-quality aggregates, the maximum W/C ratio and minimum cement content together normally result in strengths significantly higher than required. Compressive strengths up to 50 MPa and flexural strengths of up to 5.5 MPa have been obtained with the maximum W/C ratio of 0.53. Consequently, unless weak friable coarse aggregates or pebbles are used, or where the coarse aggregate is significantly fractured, it may not be necessary to test mixes at three W/C ratios.

Where low flexural strengths are obtained, i.e., less than 4.5 MPa, the broken faces of the beams should be examined for the degree of aggregate pluck-out and aggregate fracture. Should there be a large proportion of either, the W/C ratio may have to be decreased or an alternative aggregate used.

Concrete for concrete pavements should possess adequate strength to ensure

cncPAve

The cncPave design program (see Chapter 10: 8) uses average values as input parameters. The typical mean 28 day flexural strength used is 4.5 MPa if the Standard Specification is used.

a hard, durable, skid-resistant surface and to accommodate the tensile stresses resulting from shrinkage, warping and loading. For minor roads, this requirement is satisfied by specifying a target flexural-strength or modulus of rupture, generally third-point loading of not less than 4.0 MPa at 28 days. Generally, a characteristic 28-day compressive strength of 30 MPa satisfies the flexural-strength requirement, unless alluvial pebbles are used. With certain coarse aggregates and careful mix design, flexural strengths of 4.5 MPa and higher may be achieved.

High flexural strengths are beneficial. However, there is little advantage in increasing the characteristic compressive strength of the concrete mix above 30 MPa to achieve an increase in flexural strength. This is because an increase in compressive strength does not produce a proportionate increase in flexural strength. Excessively high compressive strengths are also disadvantageous, considering mix costs and the potential for construction problems associated with the use of "rich" mixes.

For major roads, concrete should meet the requirements of the Standard Specifications, which has specific strength requirements. Refer also to Chapter 4: 5.1 for Standards.

(iii) Shrinkage

Load transfer at joints and cracks depends on the crack width, and therefore, shrinkage of concrete should be limited as far as possible. The test methods to determine the shrinkage of concrete and the associated standards requirements are addressed in Chapter 3: 5.1.8.2 and Chapter 4: 5.1.9.2. It should be noted that the test (SANS 6085) is carried out at a temperature and relative humidity unlikely to be encountered in practice. The results obtained are therefore only an indication of the shrinkage in a concrete pavement.

Some authorities are of the opinion that this parameter is not relevant for dowelled pavements. There is also some thought that the limits specified (0.040%) may be too stringent, and could be relaxed. Where the mixes contain steel fibres such as in UTCRC pavements, the requirements for testing are different, and a toughness test is used (Fulton, 2009).

12.4 Design of Concrete Mixes for Pavements

Concrete is in essence a mixture of cement paste (cement and water) and aggregates (sand and stone). The aggregates reduce the amount of paste needed and affect certain properties of the concrete, such as the workability, consistency and shrinkage properties. The strength of a concrete mix is largely dependent on the cement:water ratio; the lower the water content, the stronger the concrete. Concrete mix design is all about proportioning the ingredients to attain the desired strength, consistency, workability and the other properties required by the designer. There are several methods of proportioning a suitable concrete. Fulton's Concrete Technology, Ninth Edition, Chapter 11 (2009) describes the method used by C & CI.

Mixes for use in concrete roads should be designed by an approved concrete testing laboratory. Where this is not possible or practical, the Cement and Concrete Institute should be approached for advice on materials and mix proportions.

Design of Concrete Mixes

Mixes for use in concrete roads should be designed by an approved concrete testing laboratory. Where this is not possible or practical, The Concrete Institute should be approached for advice on materials and mix proportions.
www.theconcreteinstitute.org.za

12.4.1 Information Required for Mix Design

The following information is required to design an appropriate concrete mix:

- **How the concrete is to be placed**, i.e., by hand or by machine (side form or slipform). See Chapter 12: 2.9 and 3.12 for construction details.
- **Grades and cross falls** present
- Most **appropriate consistence** to suit the particular mode of placing.
- Whether **dowel bars or tiebars** are to be inserted.
- **Type of texturing** to be applied. See Chapter 12: 2.9.6 and 3.12.9 for texturing details.
- **Climatic conditions,** which could affect plastic cracking, strength gain at the time of texturing and saw cutting.
- Choice and costs of **available materials**.

12.4.2 Constituent Materials

12.4.2.1 Aggregates

As stated in Chapter 4, Standards: 5.1.1, aggregates for use in concrete roads and streets need to comply with SANS 1083. Also, the fine aggregate should possess an acid insolubility of at least 40% for skid resistance. This requirement is generally satisfied when quartzitic sand is used. Calcareous sands, such as dolomite, may be acceptable when blended with at least 40% of suitable quartzitic sand. Some authorities have additional requirements for aggregates for concrete pavements, including (details of these tests are included in Chapter 3: 5.1.1):

- **Drying shrinkage** of the fine aggregate
- 10% Fine Aggregate Crushing test (**FACT**)
- Aggregate Crushing Value (**ACV**)
- **Flakiness Index**
- Proof that the aggregate is not **deleteriously alkali reactive**

The fineness modulus (Chapter 3: 2.3.3) of the fine aggregate should not deviate from the approved material by more than 0.20. If this occurs, the mix may have to be redesigned. Variations in fineness modulus have a significant effect on the workability of concrete, which creates problems when constructing concrete pavements, particularly with slipform paving.

The nominal maximum size of coarse aggregate in concrete pavements should be limited to one quarter of the pavement thickness. In practice, the nominal aggregate sizes suitable for pavement thicknesses are given in Table 28.

Nominal Aggregate Sizes

Specifications, e.g., SABS 1083, give specific requirements for a nominal aggregate size. For example, a 28 mm nominal size concrete aggregate requires:

- 100% passing the screen one size up, i.e., 37.5 mm
- 85% to 100% passing 28 mm
- 0 to 50% passing the 20 mm
- 0 – 25% passing the 14 mm
- 0 to 5% passing the 10 mm

Table 28. Suitable Layer Thicknesses for Aggregate Size

Nominal Aggregate Size	Suitable Layer Thickness
20 mm	100 – 150 mm
28 mm	150 – 175 mm
37.5 mm	> 175 mm

Nominal 37.5 mm aggregate should be combined with a smaller size, e.g., 20 mm, 14 mm, or 10 mm, in accordance with the recommendations of an approved concrete testing laboratory. In special circumstances, nominal 20 mm or 25.0 mm aggregate may also require blending with a smaller aggregate. The Standard Specification allows the blending of up to four sizes of stone and any number of sands. Natural or manufactured material is permitted for use as both coarse and fine aggregate.

Materials which are fractured, have impaired bond due to coatings, or are smooth-textured and/or rounded, such as alluvial pebbles, may reduce the flexural strength of concrete. Washing of the aggregate may remove coatings. Alternatively, crushing of the aggregate to expose clean faces, especially in the case of alluvial pebbles, may improve the flexural strength. Very hard aggregates, or those which slowly develop bonds with the cement paste, e.g., felsite, may delay the time at which saw cutting can be done. Such aggregates should preferably be avoided.

Aggregates that Delay Sawing

Very hard aggregates, or those which slowly develop a bond with the cement paste, e.g., felsite, may delay the time at which saw cutting can be done. Such aggregates should preferably be avoided.

12.4.2.2 Cement

Cement should comply with the requirements of SANS 50197. Cement extenders (ground granulated blast furnace slag or fly ash) should comply with the requirements of SANS 1491. The use of different cement types has marked effects on properties such as early strength gain and durability. See Chapter 4: 5.1.2 for the different types of cement and extenders.

The choice of appropriate cement type depends on the type of pavement and the environment in which it is constructed. In pavements with sawn joints, concrete must achieve a certain strength to enable sawing of the joints.

With mixes of low early strength, the time taken to reach this strength is increased. The longer the period between casting and saw cutting, the greater the possible moisture loss from the concrete, and the higher the risk of shrinkage cracks occurring before the joints are cut. Cement types, cement and extender blends and/or cement contents promoting sufficient early strength, should therefore be chosen.

Standard Specifications for Concrete

The Standard Specification covers cement and admixtures separately, but they should be investigated together to check for combined effects.

The following are specified:

- **W/C ratio** of the mix shall not be greater than 0.53.
- **Cement content** of the mix shall be no less 320 kg/m^3.
- **Air content**, if air entrainment is used, shall be between 2% and 4% and should not vary by more than 1% from the desired value.

In general, the use of cements with an extender, i.e., fly ash (FA) or ground granulated blast furnace slag (GGBS), with contents higher than 15% may result in increased savings in concrete costs due to a reduction in water and therefore cement content. However, there may be an adverse effect on some concrete properties, the most important being the low early-age strength of the concrete. Also, the greater the extender content, the greater the vulnerability of the concrete to poor curing. This may result in reduced abrasion resistance and durability. The lower early-age strength may also adversely affect the crack width and spacing in continuously reinforced concrete pavements.

The use of high extender contents generally retards the setting of the concrete, allowing bleeding over a longer period of time and delaying finishing operations. The adverse effects of extender content are often worsened in cold weather, whereas the use of extenders may be advantageous in very hot weather.

12.4.2.3 Mixing Water

The mixing water should be clean potable water, or other water free from injurious amounts of substances that may impair the strength, the setting time, the durability of the concrete, or the strength and durability of the reinforcement, tie bars or dowels. Should the quality of the water be in doubt, the water should be tested in accordance with SANS 51008. For example, sugar detrimentally retards the setting of concrete while chlorides are notorious for enhancing corrosion of steel reinforcement.

The generally acceptable limits necessary to ensure that the water used is not detrimental to the concrete are given in Table 29.

12.4.2.4 Chemical Admixtures

Chemical admixtures are used to improve the properties of the plastic concrete, or to effect savings. The use of admixtures is based on an evaluation of their effects on specific materials and combinations of materials, including air content and strength development, particularly within the first 24 hours after concrete placing. This is because certain admixtures retard the setting and strength development of the concrete, thus delaying joint sawing and increasing the risk of random cracking. The most common admixtures, and the reason for their use, are:

- **Air entrainers** improve workability and reduce bleeding, and can also be used in freeze-thaw conditions. They are beneficial in concrete placed by a slipform paver. However, as the air content is increased, the strength of the hardened concrete may have to be increased to achieve the required strengths.
- **Water reduces or plasticizers** improve workability or reduce the water content of the concrete, and thereby the cement content. However, it is often possible to reduce the cement to the minimum allowed, by the optimum choice of aggregates and good proportioning, without using admixtures. A disadvantage of water reducers or plasticizers is that many have a retarding effect on the concrete, thus affecting the early-age strength gain.
- **Retarders** may be necessary in very hot weather, or for long haul routes. The effect on early strength gain must be considered.

Admixtures Containing Chorides

Admixtures containing any form of chloride should not be permitted in concrete pavements with reinforcement, tiebars or dowels.

Chemical admixtures and air-entraining admixtures should conform to the requirements given in Chapters 3: 5.1.5 and 4: 5.1.4. Admixtures containing any form of chloride should not be permitted in concrete pavements having any reinforcement, tiebars or dowels.

Air Entrainment

Air entrainers are surface active chemicals that promote the formation of small air bubbles (mostly < 0.3 mm) in the concrete mix.

Table 29. Water Quality Classification Code

Property	Pure Water (AR)	Clean Water (Rain)	Treated Water (Municipal)	Silty (muddy) water with low salt content	Highly mineralised chloride, sulphate water (brackish)	Waste, brack, sewage marsh, sea, etc. water
	Water Type					
	H0	**H1**	**H2**	**H3**	**H4**	**H5**
pH[1] (SANS M113)	7.0	5.7 -7.9	4.5 - 8.5	4.5 - 8.5	9.0	–
Dissolved Solids[1] (ppm) (maximum) (SANS 213)	0	1000	1500	3000	–	–
Total hardness[1] (maximum) (SANS 215)	None	None	Temporary	Temporary	Permanent	–
Suspended matter (ppm) (maximum) (SANS 1049)	0	2000	2000	5000	–	–
Electrical conductivity (Ms/m) (maximum) (SANS 1057)	0	200	200	500	–	–
Sulphates, SO_4 (ppm) (maximum) (SANS 212	0	200	300	500	1000	–
Chloride, Cl (ppm) (maximum) (SANS 202)	0	500	1000	3000	5000	–
Alkali Carbonates, CO_3 & Bicarbonate, HCO_3 (ppm) (maximum) (SANS 241)	0	500	1000	1000	2000	–
Sugar (SANS 833)	Negative	Negative	Negative	Negative	Negative	–
	Suitability of Water					
	HO	**H1**	**H2**	**H3**	**H4**	**H5**
Untreated layer works	N/A	✓	✓	✓	✓	Investigate effect on quality of material
Chemically treated layer works	N/A	✓	✓	✓	Investigate effect on quality of material	×
Concrete – mass	N/A	✓	✓	✓	Investigate effect on structural strength	×
Concrete prestressed	N/A	✓	✓	×	×	×
Slurry emulsion	N/A	✓	✓	×	×	×
Soil/gravel tests	N/A	✓	✓	×	×	×
Chemical or control tests	N/A	✓	✓	×	×	×

Notes

1. A primary property. The quality of the water is that quality where all three of the primary properties are within the limits.

12.4.2.5 Steel and Fibreglass Fibres

Mixes containing steel fibres, such as in UTCRC pavements, are assessed with a toughness test. (Fulton, 1999).

12.4.3 Mix Proportioning

A number of aspects need to be considered in designing a concrete mix for paving.

12.4.3.1 Cement and Admixtures

Often the choice of cementitious material is made on economic grounds, thus minimizing the options available to the designer. The decision to use a chemical admixture for water reduction, retardation or air entrainment largely depends on the observed properties of the aggregates selected and the anticipated climatic conditions. The designer is often forced to use the minimum cement content, as the tender was based on this value. This often results in the use of a concrete mix that is far from ideal for paving.

For durability reasons, and to reduce shrinkage as far as possible, the water:cement ratio should be less than 0.53. Also, the total cementitious content, including extenders, should not be less than 320 kg/m^3. These two requirements may result in strengths significantly higher than those required by the specifications. The air content, if air entrainment is used, shall be between 2% and 4% and should not vary by more than 1% from the desired value.

The Standard Specification covers cement types and admixtures separately and does not cover the possible combined effects. It is important, therefore, that the combined effect of cement and extender type and admixture is assessed during the concrete mix design.

12.4.3.2 Coarse Aggregate

In the selection of proportioning of aggregate, some contractors put too much emphasis on the economic considerations. Good workability is equally important in meeting the special requirements of concrete for pavements. Selecting a high stone content may have economic advantages as it usually lowers the water requirement and hence the cement content. However, it can have adverse effects on workability, promote segregation and prevent easy insertion of bars. A compromise is usually necessary.

Changing the ratios of the two or more sizes of stone alters the packing capacity of the stone, and hence affects the required sand content. The stone particle shape also has a marked influence on packing capacity. Larger stone sizes are beneficial to load transfer at contraction joints which have cracked through. Smaller stone sizes contribute to better flexural strengths.

The majority of concretes for pavements in South Africa are placed using gap-graded mixes. Experience has shown that when proportioning gap-graded mixes, better results are obtained when one or two successive stone sizes are omitted. For example, a nominal 13.0 mm stone, which is often graded, blends well with a nominal 37.5 mm stone. A 20.0 mm stone is too similar in size to particles often contained in nominal 37.5 mm stone. European practice is to use continuously graded aggregates, and blending of five or more aggregates is not uncommon. Continuously graded mixes give better workability and permit easier insertion of bars.

Continously Graded Concrete Mixes

Continuously graded mixes give better workability and permit easier insertion of bars.

12.4.3.3 Fine Aggregate

In proportioning concrete mixes incorporating coarse sands, the stone content is reduced. This is preferred to the use of fine sands, which increase the stone content. However, coarse sands often lack fines and may give rise to bleeding. They may also tear during the tining or texturing process.

Where a large "gap" exists in the overall grading, stone interlock may occur when fine sands are used. Fine sands usually provide good cohesion, which helps prevent segregation, but may increase the water requirement, particularly if the dust content is high. High dust contents may also make air entrainment difficult. For slipform paving it is preferable that the mix contain sufficient minus 0.300 mm material. When this fraction is lacking, air entrainment is beneficial.

12.4.3.4 Place Ability

The stone-to-sand ratio should be such that the correct amount of mortar is worked up to the surface by the placing equipment to suit the intended method of texturing.

The insertion of bars is not only facilitated by good grading but also by good particle shape and a favourable stone:sand ratio. For paving concrete into which bars will be inserted, it is recommended that the specified flakiness index values are met, to avoid placing problems. Where the insertion of bars is difficult, it may be desirable to increase the sand content of the mix. To prevent other problems, such as slush build-up and increased edge slump, the voids in the coarse aggregate can be increased. This may necessitate varying the ratios of the stone sizes, usually by increasing the proportion of the smaller sizes. Alternatively, a drier consistence may be used.

It is recommended that the proportions selected are such that the variations in the properties of the individual ingredients do not drastically affect the properties of the concrete. This is necessary to ensure that the concrete is always placeable without disruption, and to ensure the best possible riding quality.

Conventional methods, as described in Fulton (2009), can be adopted, but these have been found inadequate for determining the stone content or the blend proportions for different stone sizes and sands for pavement concrete. Often a 60:40 blend of large to small stones is used. It is sometimes, however, necessary to use other ratios, e.g., 70:30 or 50:50. In some cases, it is necessary, due to relative particle sizes and shape of the aggregate, to use a 40:60 blend.

Experienced practitioners have found the graphical method of Rothfuchs (TRH8) convenient to obtain the necessary proportions of each aggregate ingredient in the mix, provided the required overall grading is known. The method is applicable to any number of stone sizes or sands. If continuously graded aggregates are used, then the grading envelope of the American Concrete Paving Association (C & CI Perrie and Rossman, 2009) in Figure 54 is suitable. Unless experience dictates otherwise, a good starting point is to select a grading in the middle of the envelope. A suggested envelope for gap-graded mixes, developed from data on previous mixes, which had proved to be successful in South Africa, is also included in Figure 54. A drawback of this method is that it is based on grading only, and does not consider particle shape and its effect on packing capacity. It, therefore, remains essential that laboratory trial mixes are made, and the concrete evaluated.

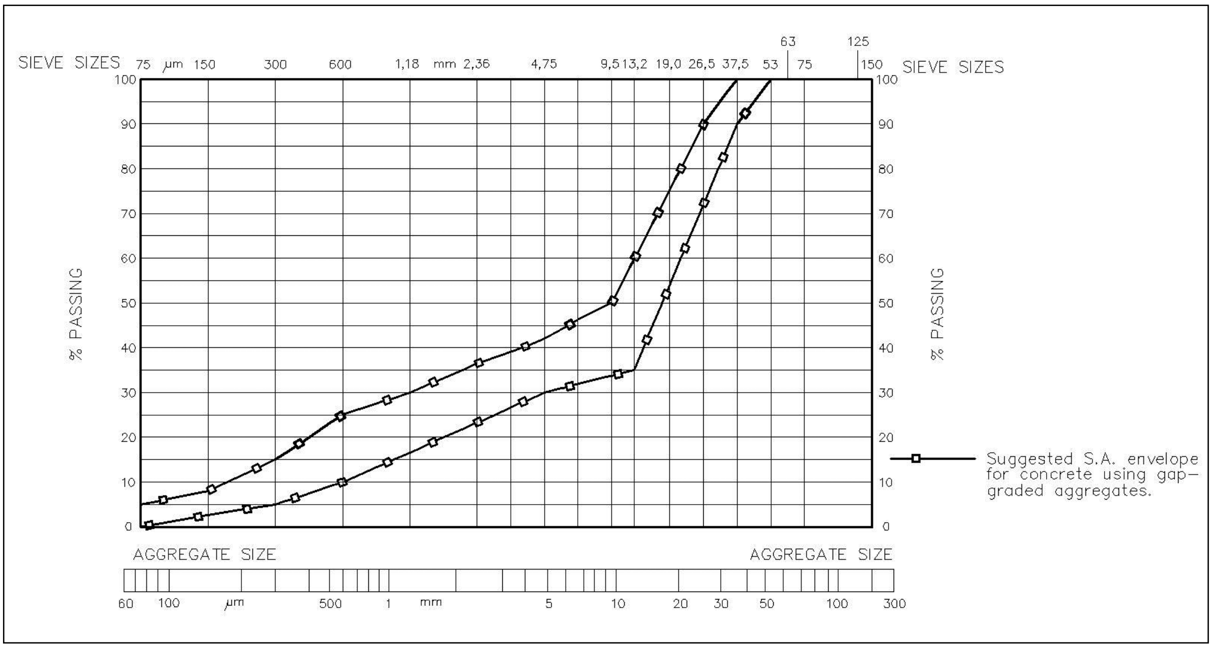


Figure 54. American Concrete Paving Association Suggested Grading Envelope

An alternative approach is suggested in the "Integrated Materials and Construction Practices for Concrete Pavements: A State-of-the-Practice Manual" issued by the FHWA (2007).

12.4.3.5 Laboratory Trials

Once theoretical mix proportions are determined, it is necessary to carry out a series of trial mixes in the laboratory, to verify and to adjust the proportioning if necessary.

After satisfactory concrete is produced, it is possible to proceed with evaluation of strength at various W/C ratios and ages, to determine relationships between flexural and compressive strengths. These relationships are used later on site for control purposes. If the most suitable values for slump and Vebe are not known, these are also established.

A number of simple techniques (see Section 12.3) are used to assess each trial mix with the requirements for paving concrete, the relevant specifications and to compare two or more mixes. However, experience is necessary to determine what adjustments should be made, if necessary.

The workability of the selected mixes for field trials should be such that the concrete can be properly placed with the available equipment. The consistence when controlled by the slump test should be within 10 mm of the required value or, with the Vebe test, within 4 Vebe seconds of the required value. The Vebe test equipment is shown in Figure 55, and is discussed in Chapter 3: 5.1.8.1.

Figure 55. Vebe Test Equipment

12.4.3.6 Site Trials

Prior to the full-scale site trial, it is advantageous to the contractor to carry out limited trials to ascertain that the designed concrete mix is suitable; the batching plant has been set up satisfactorily; and the placing equipment is functioning correctly and can handle the concrete. Paving trials should preferably be done under the worst conditions likely to be encountered during construction, e.g., at a location where grade and cross-fall are most severe. Concrete paving trial sections are discussed in Chapter 12, 4.6.

Site Trials
Paving trials should preferably be done under the worst conditions likely to be encountered during construction, e.g., at a location where grade and cross-fall are most severe.

From the site trial, the concrete mix as manufactured and placed under site conditions is assessed against the requirements such as workability, consistency, cohesiveness, bleeding potential, dowel insertability (if appropriate), texture ability, saw ability and shrinkage under site conditions and strength.

At this stage, it is likely that minor adjustments to the mix are made to ensure a workable mix, compliant with the requirements, is achieved.

13. BLOCK PAVING

Concrete block paving, also known as segmented concrete paving, comprises of small individual shaped blocks laid shoulder to shoulder on a sand bedding layer. Sand is placed in the network of closely spaced joints between the blocks to fill the gaps and to enhance interlock between the blocks. This system of blocks, jointing sand and bedding layer provides a durable wearing course. The system is supported by a subbase and selected layers, of which the thickness depends on the applied loading and subgrade strength. The block pavement system is illustrated in Figure 56.

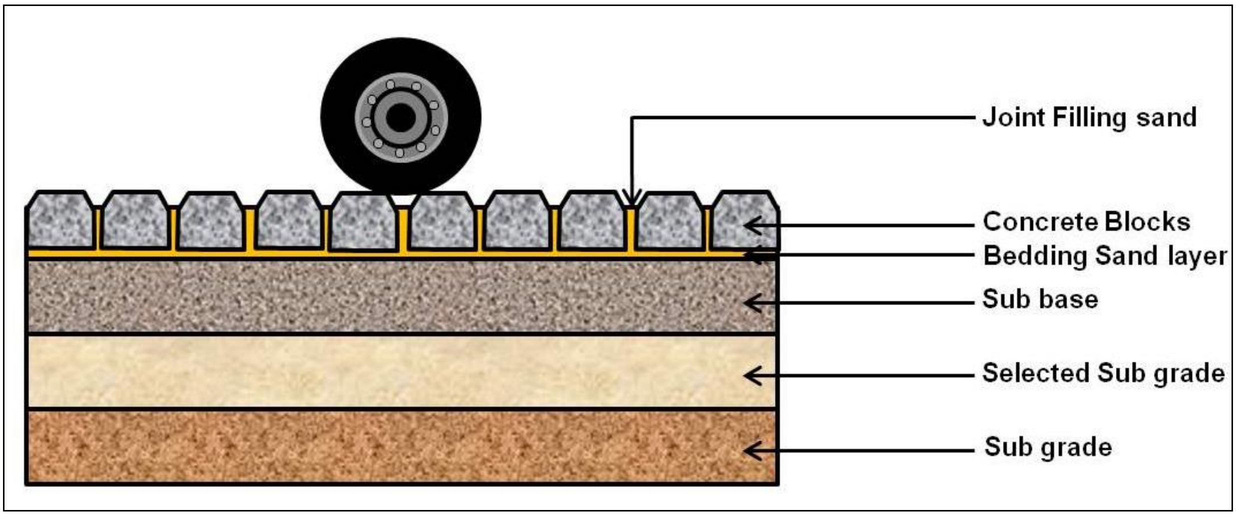


Figure 56. Block Pavement System

Block paving using precast paving blocks has been successfully used in South Africa and in many other parts of the world in different applications over the past 30 years. Applications range from non-trafficked areas, such as footpaths and civic areas, to heavily trafficked streets and large industrial stacking areas. In areas where relatively high traffic speeds are expected, block paving should be used with caution unless proper construction practice and maintenance of the joints in the early stages of its life are guaranteed.

This section deals with the wearing course elements of the block pavement, i.e., the paving blocks, jointing sand that fills the gaps, and bedding sand layer on which the blocks are laid. The quality of the support layers are designed and constructed in the same manner as a conventional pavement structure and are dealt with in other sections of this guideline. Pavement design for concrete blocks is including in Chapter 10: 9.

The paving blocks act as a structural layer, rather than merely providing a wearing course, if the block paving is properly constructed, ensuring interlock or lock-up between individual blocks. There are several aspects of the block paving wearing course layer with a significant effect on the performance of the layer. It is thus essential that all these aspects receive the necessary attention and control during both manufacturing of the blocks and construction of the layer, to obtain the maximum structural strength.

There are various specifications and guideline documents published to assist in the successful construction of block paving. The following publications are most widely used in South Africa:

- **The current specification** in South Africa for the requirements for concrete paving blocks is SANS 1058 – Concrete Paving Blocks.
- **The current specification** in South Africa for the construction of concrete block paving is SANS 1200 MJ – Standard Specification for Civil Engineering Construction: Segmented Paving.
- **The Standard Specifications** also have specifications for block paving, which need to be complied with on COLTO based projects. These specifications may be different to those of SANS and CMA.
- Cement & Concrete Institute (C & CI) issued a technical note "**The Manufacture of Concrete Paving Blocks**" that covers the basic principles of block paving and aims to assist manufacturers to

Block Paving in High Speed Traffic Areas

In areas where relatively high traffic speeds are expected, block paving should be used with caution unless proper construction practice and maintenance of the joints in the early stages of its life can be guaranteed.

produce a durable and consistent product. This note can be downloaded from www.theconcreteinstitute.org.za.

- CSIR published the **Urban Transport Guideline (UTG2):** Structural Design of Segmental Block Pavements for Southern Africa.
- The **Concrete Manufacturers Association** (CMA) has four books on **Concrete Block Paving**. These books can be downloaded from www.cma.org.za. The books are titled:
  - Book 1: Introduction
  - Book 2: Design Aspects
  - Book 3: Specification and Installation
  - Book 4: Site Management and Laying
- The **CMA** also has guideline brochures on several aspects of **concrete block paving,** which can be downloaded from www.cma.org.za.
  - A Step-by-Step Guide to Perfect Paving
  - Good Earthworks Practice
  - Drainage of Concrete Block Paving
  - Technical Note for Steep Slopes
  - Efflorescence is only a Temporary Problem
  - An Introduction to Permeable Concrete Block Paving
- Other very useful guides on various aspects of concrete block paving are publications by the **Concrete Masonry Association of Australia** (CMAA), which can be downloaded from www.cmaa.com.au.

13.1 Paving Blocks

Paving blocks are the main element of the system. It is thus essential that the block strengths, shape and dimensions are tightly controlled, to ensure a constructed layer of adequate strength.

Pavement Design for Block Pavements

The pavement design for block pavements is included in Chapter 10: Section 9.

13.1.1 Block Strength

The required strength of concrete block pavers is significantly higher than the strength of concrete bricks, which have compressive strengths in the order of 7 MPa. The strength of the blocks should be sufficient to ensure that the blocks have adequate resistance to withstand the wedging effect between adjacent blocks during traffic loading, and the abrasion effect of the tyres during traffic movements. It is thus essential that the strength of the individual blocks is controlled to ensure proper performance.

The specification currently used is SANS 1058, referred to in the Standard Specifications, which requires the following for the blocks:

- Minimum wet **compressive strengths** of:
  - 25 MPa for paving blocks for light traffic conditions
  - 35 MPa for heavy traffic conditions, with wheel loads higher than 30 kN
- Minimum **tensile splitting strength** of:
  - 2.2 MPa for light traffic conditions
  - 2.8 MPa for heavy traffic conditions
- The average and individual **mass loss of blocks** shall not exceed 12 g and 15 g respectively during the abrasion resistance tests. This test is discussed in Chapter 3: 5.2.1.
- The average and individual **water absorption** measured in the blocks shall not exceed 6.5% and 8.0% respectively. This test is discussed in Chapter 3: 5.2.1.

Although the specifications require a minimum strength for the blocks, research has shown that compressive and tensile strengths significantly higher than the minimum specified have little structural advantage to the block paving layer.

13.1.2 Block Shape

There are three different types of shapes of concrete paving blocks manufactured in South Africa. The difference between the types is the effective interlock achieved between adjacent blocks in a laying pattern. Clay bricks are not used as paving blocks anymore, as they are generally only rectangular in shape. The multi-sided paving blocks have a greater surface contact area with adjacent blocks, greatly improving the load transfer capabilities. The different shape types, which refer to the plan shape of the blocks, and whether interlocking or non-interlocking, are as follows:

- **Type S-A**: The shape of these blocks allows geometrical interlock between all vertical or side faces of adjacent blocks. When keyed together, this block type resists joint opening by their plan geometry. The shape of the blocks and a typical paved area are shown in Figure 57.
- **Type S-B**: The shape of these blocks allows geometrical interlock between some faces of adjacent blocks. When keyed together, this block type resists the joint opening parallel to the longitudinal axes of the blocks. The shape of the blocks and a typical paved area is shown are illustrated in Figure 58.
- **Type S-C**: The shape of this block type does not allow geometrical interlock between adjacent blocks. When keyed together these blocks rely on their dimensional accuracy and the laying accuracy to develop interlock between adjacent blocks. The shape of the blocks is illustrated in Figure 59.

Paving Block Shape Types

Type S-A blocks develop the best resistance to both vertical and horizontal creep and are therefore recommended for areas where heavy traffic conditions are experienced, such as industrial areas and major roads. The shape types S-B and S-C have significantly lower resistance to creep, and are therefore generally only used for aesthetic reasons in paved areas or in lightly trafficked parking areas and streets.

Type S-A blocks develop the best resistance to both vertical and horizontal creep and are therefore recommended for areas where heavy traffic conditions are experienced, such as industrial areas and major roads. Shape types S-B and S-C have significantly lower resistance to creep and are therefore generally only used for aesthetic reasons in paved areas, or in lightly trafficked parking areas and streets.

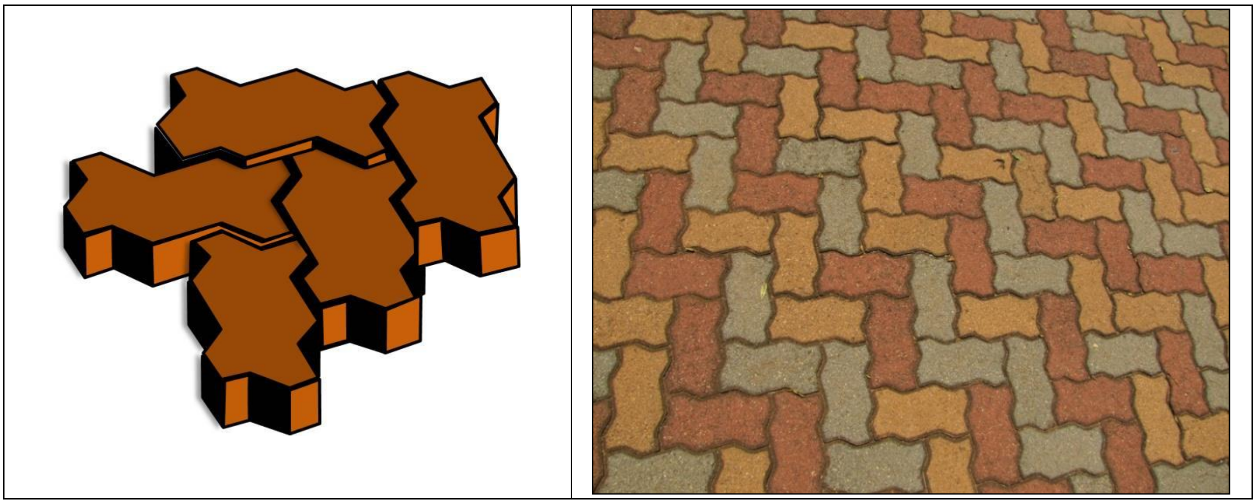


Figure 57. Type S-A Paving Blocks

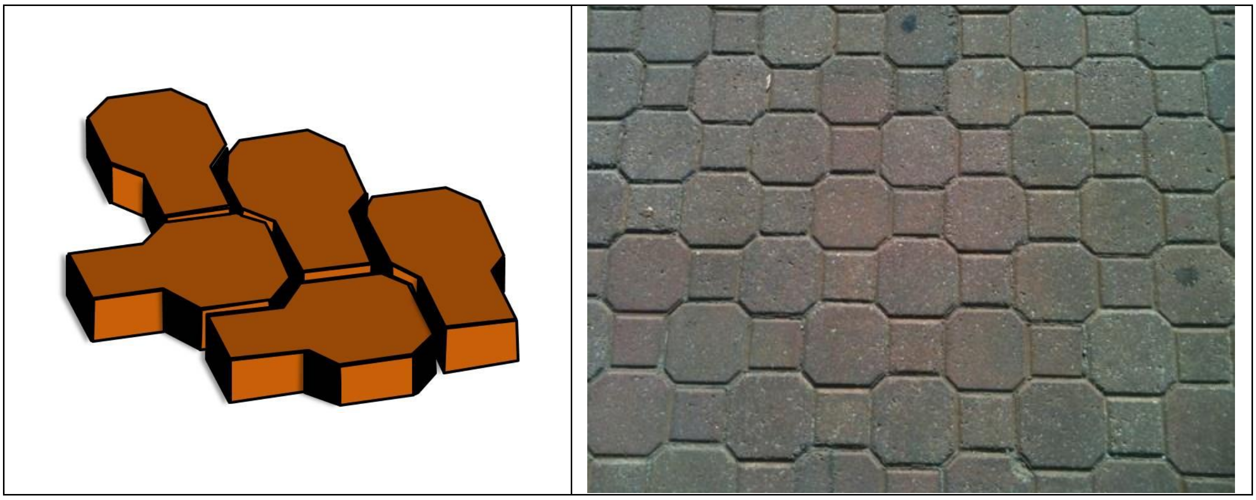


Figure 58. Type S-B Paving Blocks

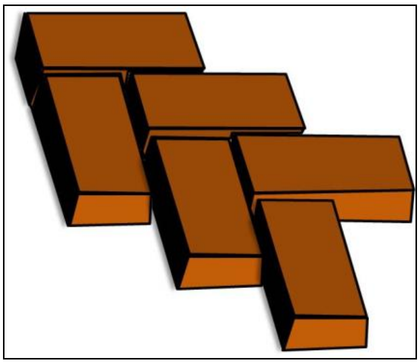


Figure 59. Type S-C Paving Blocks

13.1.3 Thickness

Paving Block Thickness
The most common thickness used in heavily trafficked areas is 80 mm.

Block pavers of variable thickness are available, but 60 mm, 80 mm or 100 mm thicknesses are generally used, depending on the traffic conditions. An increase in the thickness of the blocks results in a reduction in the rutting deformation, surface deflections and subgrade stresses. This is attributed to the larger contact area between the blocks, resulting in improved interlock or lock-up of the paved layer. Research has shown that a change in block thickness from 60 mm to 80 mm has a more significant effect on the performance of the layer than a change from 80 mm to 100 mm. The most common thickness used in heavily trafficked areas is 80 mm.

13.1.4 Dimensional Tolerance

It is very important for block pavers to be of similar dimensions to ensure good interlock between the pavers and to provide a smooth surface on the constructed layer. The required tolerances on individual blocks as specified in SANS 1058 are ± 2 mm on plan dimensions and ± 3 mm on thickness.

The specification also requires that the thickness of the block, as measured at any point along the perimeter, not vary by more than 2%. This specification is necessary to ensure an acceptable riding quality of the paved layer.

13.2 Laying Patterns

The laying pattern is an important contributor to the performance of the block paved layer. The efficiency of a block layer is the degree to which blocks must rotate to achieve interlock. "Mechanisms of Paver Interlock" by Schackel and Lim (2003) provides useful information on the effect that the shape of the block, as well as the laying pattern, has on the performance of the block paving layer.

There are numerous patterns for the laying of paving blocks. The following patterns are the most common, and are illustrated in Figure 60:

- **Stretcher** pattern
- **Herringbone** pattern
- **Basket weave** pattern

The function of the laying pattern is to resist the effects of vehicular traffic loads, whether the vehicles are travelling in a straight line or turning, which disturbs the laid blocks, leading to a loss of interlock and thus serviceability of the paved area.

Paved areas with laying patterns other than the herringbone pattern are generally disturbed by traffic loading and movement during the early stage of trafficking. This disturbance leads to irregular gaps between rows of pavers, which in turn leads to a loss of jointing sand, resulting in excessive movements between blocks. An example is shown in Figure 61. The excessive movements between individual blocks results in spalling of the contact points between blocks. Another benefit of the herringbone laying pattern is that it can be laid around bends and corners without interrupting the laying pattern.

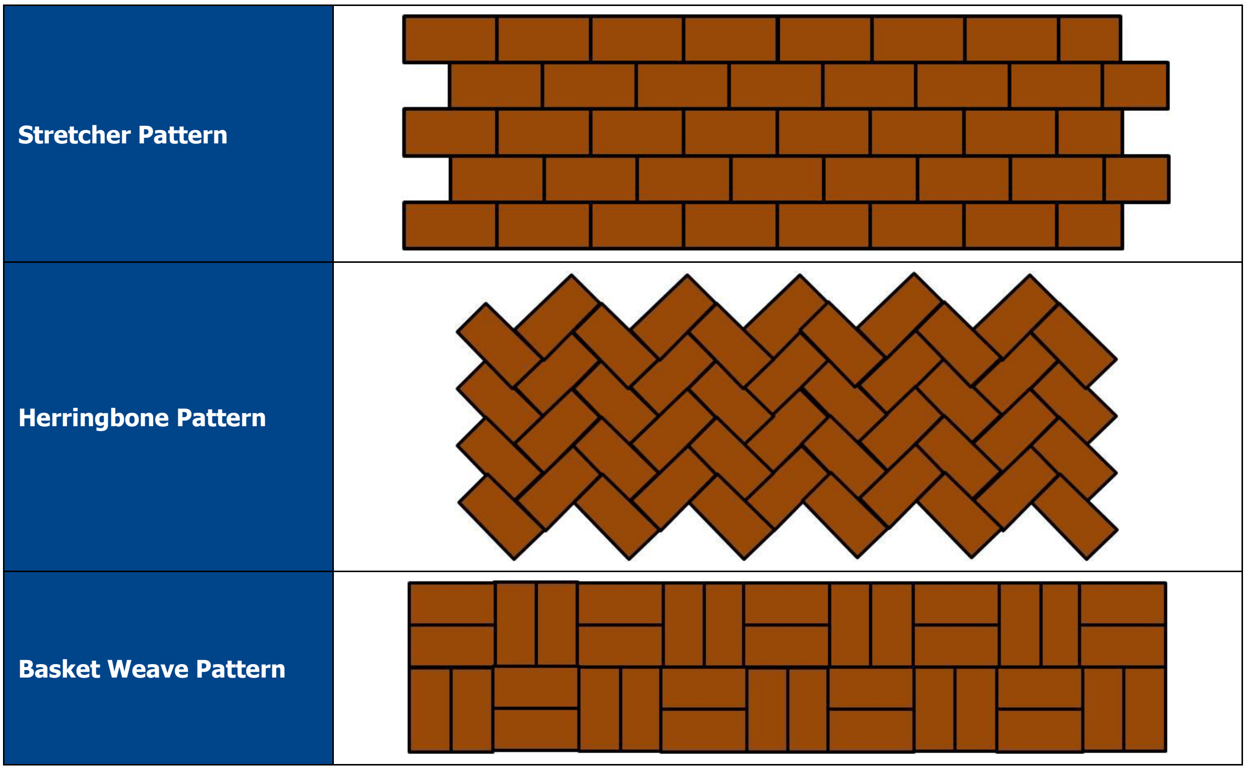


Figure 60. Common Laying Patterns of Paving Blocks

Figure 61. Excessive Openings in an Area Paved with Type S-C Blocks

13.2.1 Edge Restraint

Edge restraint is required along the edges of a block paved area. It is very important to retain the bedding sand, and to ensure that the paving blocks at the edge of the paved area do not creep outwards or rotate under load, causing opening of the joints and loss of the interlock between the paving blocks. The integrity of the paved layer is lost if the joints open.

Varies types of edge restraint are used, including concrete drains, concrete kerbs and edge beams. The edge restraints should have sufficient stability to withstand occasional vehicle impacts. They can also be used to separate areas of different laying patterns, or as kicker blocks where steep grades are encountered.

13.3 Joint Filling Material

Filling the joint or gap between individual pavers is crucial to the performance of a concrete block paving layer as block pavers develop very little, or no, strength with open joints. A block paving layer develops its structural capacity by the wedging action between individual pavers during construction and under traffic.

The small gaps or joints, nominally 2 – 5 mm wide, between the block pavers are filled with a fine continuously graded dry sand, which conforms to the grading specified in SANS 1200 and included in Table 30. The use of cement in the joint filling material is not recommended.

Table 30. Grading of Joint Filling Material

Nominal Sieve Size (mm)	% Passing
2	100
1	90 – 100
0.600	60 – 90
0.300	30 – 60
0.150	15 – 30
0.075	5 – 10

Cement as a Joint Filling Material

The use of cement in the joint filling material is not recommended.

Maintaining the jointing sand protects the structural integrity of the paving layer. If sand is lost, it is essential to refill the joints, especially when windy conditions and/or fast moving traffic are experienced.

The cumulative compactive effect of traffic causes the blocks to bed further into the bedding sand layer, and displaces some of the jointing sand upwards. Vehicle movements remove some of the joint filling sand, which becomes part of the detritus on the surface of the paved layer. This detritus eventually forms an upper plug over the jointing sand and assists in sealing the block paving layer.

13.4 Bedding Sand Layer

The bedding sand has a crucial influence on the performance of a block paving layer. The following three factors are the main contributors to the uniformity of the layer and should be controlled during construction:

- **Thickness** of the sand layer
- **Grading** and **angularity** of the sand
- **Moisture content** of the sand during construction and in service

The thickness of the bedding sand layer is between 25 to 30 mm in a loose condition. Most failures in block paving occur because the bedding sand layer exceeds the recommended limits. Bedding sand thicker than 30 mm can result in differential compaction under the blocks, leading to loss of interlock between the blocks. The sand should conform to the grading given in Table 31 (SANS 1200).

Thickness of Bedding Sand Layer

The thickness of the bedding sand layer is between 25 to 30 mm in a loose condition. Most failures in block paving occur because the bedding sand layer exceeds the recommended limits. Bedding sand thicker than 30 mm can result in differential compaction under the blocks, leading to loss of interlock between the blocks.

Table 31. Grading of Bedding Sand

Nominal Sieve size (mm)	% Passing
10	100
5	95 – 100
2	80 – 100
1	50 – 85
0.600	25 – 60
0.300	10 – 30
0.150	5 – 15
0.075	0 – 10

In addition, the following material requirements should be noted:

- Materials, such as clean graded **crushed quarry fines** and good quality **concrete sands** perform well, provided the paving blocks are constructed correctly.
- Bedding sand shall be free from **deleterious substances,** be **non-plastic** and **permeable**. The silt and clay contents should be less than 20%.

- **Single-sized, gap-graded** or material containing an excessive amount of fines, reduces performance. The proportion of silt and clay material smaller than 0.075 mm should be less than 15%
- **River sand** should be used with caution as the angle of shearing resistance is generally low due to the rounded shape of the particles.
- Use of **cement-bound material** is not recommended.

The material should have a uniform moisture content of 4 to 8% when placed. Saturated material should not be used.

14. PROPRIETARY AND CERTIFIED PRODUCTS

Proprietary products are manufactured and distributed by a company with exclusive rights to the product. These products are usually protected by trademark, patent or copyright. They may be standard products, in the sense that they meet the requirements of local or foreign standards, or they may be non-standard or innovative, in that there are no applicable standards.

'Standard' refers to products meeting the requirements of specifications, codes of practice and standard methods. In South Africa, reference is sometimes made to SABS 'approved' or 'certified' products. This reference is applicable to products manufactured by companies participating in, and meeting the requirements of, the SABS's Mark Scheme, which is applicable to SABS 'standard' products.

Certified non-standard or innovative products are usually those that have been subjected to some form of independent review, which confirms that they either meet certain minimum claimed or specified requirements. In South Africa, the organisation responsible for carrying out such assessments is Agrément South Africa. Agrément is a member of the World Federation of Technical Assessment Organizations (WFTAO), which currently comprises 27 similar organisations worldwide (www.wftao.com). However, not all of the WFTAO members assess road products, some limit themselves mainly to building related products.

Certification of Proprietary Properties

It is important to note that the certification of such products indicates only that they have conformed to certain controlled criteria. It does not necessarily guarantee that they will perform satisfactorily with all materials under all conditions.

14.1 Agrément South Africa's Certification Process

The Agrément certification process is managed by the agency or secretariat of Agrément South Africa. In all cases, external specialists, recognised as experts in their respective fields, are appointed by the Agency to carry out the assessments.

The certification process is divided into the following stages:

- **Assessment of applicant's data**. The applicant's documentation is thoroughly studied, and missing data identified.
- **Assessment of production control**. The applicant's documented quality system, as applicable to manufacture, installation or application of the product, is scrutinised.
- **Laboratory testing.** Relevant material properties required to confirm that performance requirements are met and to ensure that properties of materials at various stages of the manufacturing and application process are available for quality control purposes, are established. Should these not be submitted by the applicant, the assessment process ensures that these properties are determined.
- **System installation.** Implementation of the applicant's proposed quality system for the installation or application of the product is assessed. The applicant's ability to offer technical support to prospective specifiers and clients, as well as to ensure that adequately trained staff are involved in the manufacture and installation/application is also assessed.
- **System performance trial (if required).** Where no established track record of performance is available, the performance of the product in use in the field is monitored. This is the case with new products.
- **Certification.** On successful completion of the assessment, an interim certificate, if a performance trial is required, or final certificate is granted to the applicant by the Board of Agrément South Africa.
- **Monitoring.** All certificate holders are subject to ongoing quality assurance monitoring and periodic certificate reviews. Reports of product failures are followed up and appropriate action in the form of adjustments to formulations, manufacturing and installation processes insisted upon, where necessary. Non-compliance with the conditions of certification could lead to suspension of the certificate, or its cancellation.

A graphical representation of the assessment process is given in Figure 62.

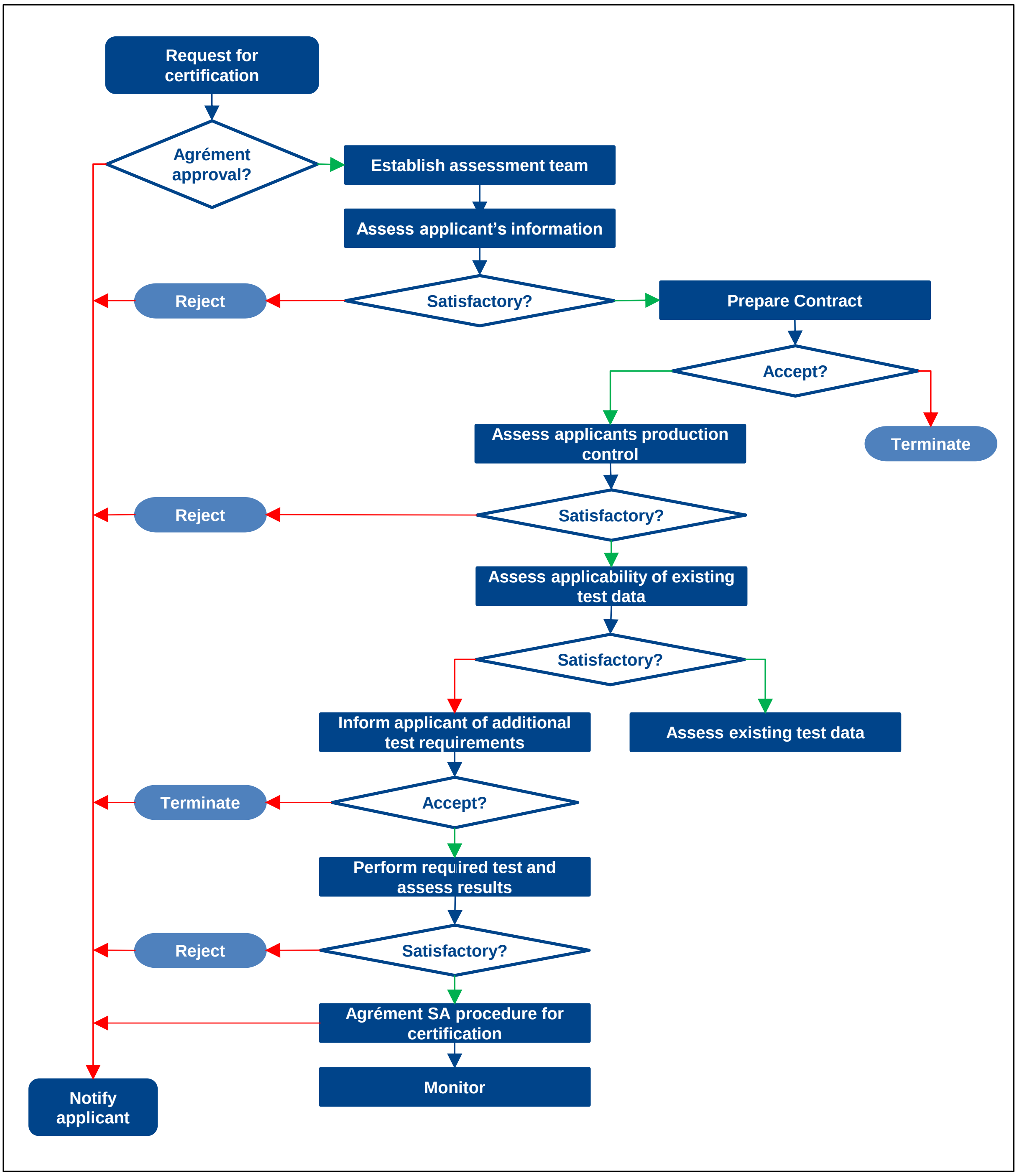


Figure 62. Assessment Process for Agrément Certification

14.2 Materials Utilisation and Design

The purpose of Agrément certification is to provide reassurance that the product has been assessed by an independent organisation, and is fit-for-purpose and/or meets certain requirements. An essential component of the Agrément certification process is quality assurance. It is important, before specifying or using an Agrément product, to seek confirmation from Agrément South Africa that the certificate is currently active. There are many reasons why certificates become inactive, are suspended or cancelled, and most of these reasons are likely to impact negatively on the product performance. The status of a certificate is verified on Agrément South Africa's website at www.agrement.co.za, or by contacting the organisation on (012) 841 3708.

It is essential that the product is used as intended. It is for this reason that the certificate includes the following information:

- **Uses** for which the product is intended
- **Description** of the product
- **Conditions** of certification
- **Scope** of the assessment
- **Performance requirement** that has been met or, if applicable, that must be met after any performance trial period
- Details of how the product must be **correctly applied** on site and maintained, where applicable.

Over and above this, the actual assessment reports, on which approval is based, are made available to the certificate holder upon completion of the assessment. At their discretion these are available to design engineers or other interested parties.

14.3 Range of Road Construction Products

Road construction products belonging to the following categories are currently certificated by Agrément South Africa:

- Bridge deck expansion joints
- Concrete additives
- Thin bituminous surfacing systems
- Soil compaction aids
- Bituminous road primers

Criteria have already been established for dust palliatives and cold-mix asphalt, but to date (November 2012) such products are yet to be certified. Criteria for product assessments are included in the various guideline documents, which may be downloaded free of charge from Agrément South Africa's website (www.agrement.co.za). Criteria for other products can be drawn up, should the need for such be expressed by role players in industry. Details on the kinds of tests and standards used for proprietary products are included in Chapter 3: 6.3.

14.4 Guidance in the Use of Certificated Products

The following guidance is offered for those responsible for specifying and overseeing the use of certificated products:

- Obtain, from the certificate holder or Agrément South Africa's website, a **copy of the complete certificate**.
- Check whether the **certificate is still valid** by contacting Agrément South Africa.
- Check **who will install the product** or system. It is usually a requirement of the certificate that installation is carried out by the certificate holder, or a licensee appointed by the certificate holder, and registered with Agrément South Africa.
- Check that the product or system will be used for the 'uses' assessed, and in accordance with, the **conditions of certification.**
- Ensure that there are **no variations to materials** listed in the certificate, in the method of manufacture or installation. Amendments to certificates are usually required before changes can be made. If a change has not been addressed in a certificate, amendment authorisation in writing from Agrément South Africa must be provided by the certificate holder.
- In the event of a **product or system failure**, please alert Agrément South Africa.

To design proprietary and certified products, no generic methods are available. However, the broad principles of material design, as discussed in this chapter, apply. For specific products, the user should check with the supplier for specific guidelines.

Variations to Materials

Ensure that there are **no variations to materials** listed in the certificate, in the method of manufacture or installation. Amendments to certificates are usually required before changes can be made. If a change has not been addressed in a certificate, amendment authorisation in writing from Agrément South Africa must be provided by the certificate holder.

15. MATERIAL CLASSIFICATION SYSTEM FOR DESIGN

A major component in the condition assessment of a road for a rehabilitation design is investigating the materials and their condition in the existing pavement structure. From this investigation, a material class is derived for each layer that can be used for design. Traditionally, two methods are generally used to decide on a material class:

- **Specification method**. The material class is decided based on the specified limits for the material properties of each material, such as given in TRH14 or the Standard Specifications. These are essentially pass or fail type criteria, and if any one test fails the criterion for the material class then the material cannot be classified as that class. For example, if the CBR value is below the specification for a G6 material, then the material cannot be classified as a G6 even if all other available test results do meet the G6 criteria.
- **Subjective assessment.** In deciding on an appropriate material class, experienced engineers consider all the available test parameters and condition assessments to derive an appropriate material class for design purposes. While an experienced engineer is likely to derive an appropriate material class, when incorrectly done, inconsistent conclusions can be drawn. The resulting design assumptions are then not consistently supported by the available evidence.

Materials Classification System for Design

The objective of the material classification method is to provide a method for the consistent assessment of pavement materials using routine tests and indicators.

The concept behind the material classification system for design is to guide engineers in interpreting available pavement condition data, and to synthesize available information so that key design assumptions are derived in a consistent and rational manner. The objective of the material classification method is therefore to provide a method for the consistent assessment of pavement materials using routine tests and indicators. The method also allows the determination of the certainty associated with the assessment.

The material classification system is described in detail in the 2nd edition of TG2 (2009). The method is also detailed on www.asphaltacademy.co.za/bitstab. The method was originally developed as part of a research project on BSMs, but is also applicable to granular and cemented materials.

15.1 Design Equivalent Material Class (DEMAC)

The material classes adopted for this material classification method are aligned with TRH14 and TG2. These classification systems are regarded as suitable for new construction and rehabilitation design, as the behaviour and performance patterns of each material class is known with some certainty. However, with the material classification method, the obtained materials class is regarded as the design equivalent materials class (DEMAC), and does not necessarily meet the specifications for that material class as given in the specification. However, since materials to which design equivalent classes are assigned have been in service for some time, the raw material would conform to, or exceed, the specifications for the class, as stated in the specification, in almost all situations.

Design Equivalent Material Class

When design equivalent material class (DEMAC) is assigned to a material, it implies that the material exhibits in situ shear strength, stiffness and flexibility properties similar to those of a newly constructed material of the same class. For example, a layer in an existing pavement structure classified as a DE-G2 indicates that the material is considered to be equivalent to a G2 for design purposes, based on the available test evidence.

When a design equivalent material class is assigned to a material, it implies that the material exhibits in situ shear strength, stiffness and flexibility properties similar to those of a newly constructed material of the same class. For example, a layer in an existing pavement structure classified as a G2 design equivalent indicates that the material is considered equivalent to a G2 for design purposes, based on the available test evidence. For brevity, a DEMAC is denoted DE, e.g., DE-G2.

15.2 Holistic Assessment of Materials

The material classification system provides a framework for the rational synthesis of several different test indicators. The outcome of the assessment becomes more reliable as more test indicators are added to the assessment. This is because each test typically explains only a small part of the total material behaviour. The reliability of the assessment is greatly increased by increasing the sample size, and adding more indicators, i.e., test types, to the assessment. The system is therefore a holistic assessment, which works best when a comprehensive range of test indicators are used.

Because most pavement materials tests provide only a partial indication of the behaviour of a material, a certainty factor is assigned to each test indicator. The certainty factor represents the subjective confidence in the ability of a test to accurately indicate the material strength and stiffness. The certainty factor ranges from 0 to 1, with a value of 1 indicating absolute confidence in a test or indicator (a highly unlikely assignment).

15.3 Applicable Materials and Tests and Indicators

The material classification system is applicable to granular, bitumen stabilized and cement stabilized materials. For each material, a wide variety of tests are used to assess a design equivalent material class.

Certainty Factor for Tests and Indicators

Because most pavement materials tests provide only a partial indication of the behaviour of a material, a certainty factor is assigned to each test indicator. The certainty factor represents the subjective confidence in the ability of a test to accurately indicate the material strength and stiffness. The certainty factor ranges from 0 to 1, with a value of 1 indicating absolute confidence in a test or indicator.

15.3.1 Granular Materials

Granular materials are assessed to determine an appropriate material class for the existing material in the pavement or for a new material. A material class aligned with TRH14, i.e., DE-G1 to DE-G10 is assigned. See Chapter 4, Appendix A for the G classes. The indicators and tests used to classify a granular material are:

- Soaked CBR
- Density
  - Relative density
  - Consistency, as observed during trial pit excavation
- DCP penetration
  - FWD backcalculated stiffness
- Consistency, as observed during trial pit excavation
- Plasticity Index (PI)
- Moisture content
  - Measured in the laboratory and calculated as a percentage of the optimum moisture content
  - Visible, as observed during trial pit excavation
- Particle distribution
  - Grading
  - Grading modulus
  - Percent passing the 0.075 mm sieve
- Historical performance based on condition and traffic accommodated

Justification for the selection of these tests and indicators is given in TG2.

It is not necessary to have results for all the tests and indicators, the available data are used. However, the more data available, the higher the final certainty, and hence the confidence in the assigned material class.

15.3.2 Bitumen Stabilized Materials

BSMs are investigated either to assign a material class to the current condition, or to determine if a material is suitable for stabilization and to assign an appropriate material class for the stabilized material. BSMs are assigned to the material classes suggested in TG2, or "not suitable for treatment". The TG2 classes are summarised in Section 9.2.

With BSMs, the material prior to treatment, i.e., the parent material, and the material after treatment are assessed. The results for the material after treatment are obtained from the material in situ or from mix design results. The tests and indicators used for the untreated parent material are:

- Soaked CBR
- Relative density
- DCP penetration
- FWD backcalculated stiffness
- Plasticity Index (PI)

Available Indicators

It is not necessary to have results for all the tests and indicators for each material. However, the more data available, the higher the final certainty, and hence, the confidence in the assigned material class.

- Moisture content measured in the laboratory and used as a percentage of the optimum moisture content
- Particle distribution
  - Grading
  - Grading modulus
  - Percent passing the 0.075 mm sieve

From the treated material, the following tests results are used:

- Cohesion, retained cohesion, friction angle and tangent modulus from the triaxial test
- ITS
- UCS

15.3.3 Cement Stabilized Materials

Cement stabilized materials are only evaluated for their current condition, to determine if the material is still in a cemented state or in an equivalent granular state. The material classes assigned are aligned with C1 to C4 from TRH14 (see Chapter 4: 5.3.2 of this manual) or as an equivalent granular material. The following tests and indicators are used:

- DCP penetration
- FWD backcalculated stiffness
- Consistency, as observed during trial pit excavation
- Evidence of active cement, based on chemical tests such as phenolthalein and hydrochloric acid (see Chapter 6: 7.4.1.1.

Should a cement stabilized material be assigned as an equivalent granular material, then the tests and indicators listed in Section 15.3.1 for granular materials are used to classify the material as a DE-G1 to DE-G10.

15.4 Determining the Material Class

The material classification system utilises all available indicator and test data to derive an appropriate material class, and assigns a certainty associated with the material class. The method uses fuzzy logic and certainty theory. The ideas behind the system workings are conceptually, and briefly, explained here. For a more detailed explanation refer to TG2 or Jooste et al (2007).

(i) Step 1: Determine Statistics from the Data

The first step involves calculate statistics for all the available test data, including:

- 10^{th} percentile
- 90^{th} percentile
- Median
- Number of observations

Some data are not conducive to calculating a statistic. For example, gradings are described by the percent passing several sieve sizes and therefore there is no single value with which to calculate statistics. In this case, the grading is converted to a rating, based on how it fits to an appropriate grading envelope used for specifications. Similarly, consistency and visible moisture observations are converted to a rating. Specific guidelines are provided for the conversions. For all test data where a value is obtained, the statistics are calculated from the values.

(ii) Check Limits for Classes and Tests

The method provides criteria to assess the test results or ratings for each material class and test. The criteria for the DCP penetration rate for granular materials are given in Table 32. If, for example, the DCP penetration rate is 6.0 mm/blow, the material class is a DE-G6, based on this result.

(iii) Calculate the Certainty that a Material Falls into a Material Class Based on Each Test

The certainty that a material falls in into a particular material class is calculated graphically, as illustrated in Figure 63 for the DCP penetration. Essentially the calculation requires constructing a triangle, with the base of the triangle going from the 10^{th} to the 90^{th} percentile. The apex of the triangle is at the median value, with the height of the triangle set equal to 1. The area of the triangle within the limits of each class as a proportion of the whole triangle area is calculated, and gives the certainty of that material class. The certainty values range from 0 to 1. Figure 63 includes a worked example and the certainties of DE-G2 to DE-G5 are calculated. For this example, based just on the DCP, this material is most likely a DE-G4, with a 0.78 certainty.

Table 32. DCP Criteria for Granular Materials

Design Equivalent Material Class	DCP Penetration Rate (mm/blow)
DE-G1	< 1.4
DE-G2	1.4 to 1.8
DE-G3	1.8 to 2.0
DE-G4	2.0 to 3.7
DE-G5	3.7 to 5.7
DE-G6	5.7 to 9.1
DE-G7	9.1 to 14.0
DE-G8	14.0 to 19.0
DE-G9	19.0 to 25.0
DE-G10	> 25

(iv) Adjust Certainty with Certainty Factor for Each Test

The certainty calculated in the previous point is adjusted for the certainty factor (Section 15.2). For example, the certainty factor for the DCP for granular material is 0.4 and therefore, the final certainty of a DE-G4 is therefore 0.78 x 0.4 = 0.31.

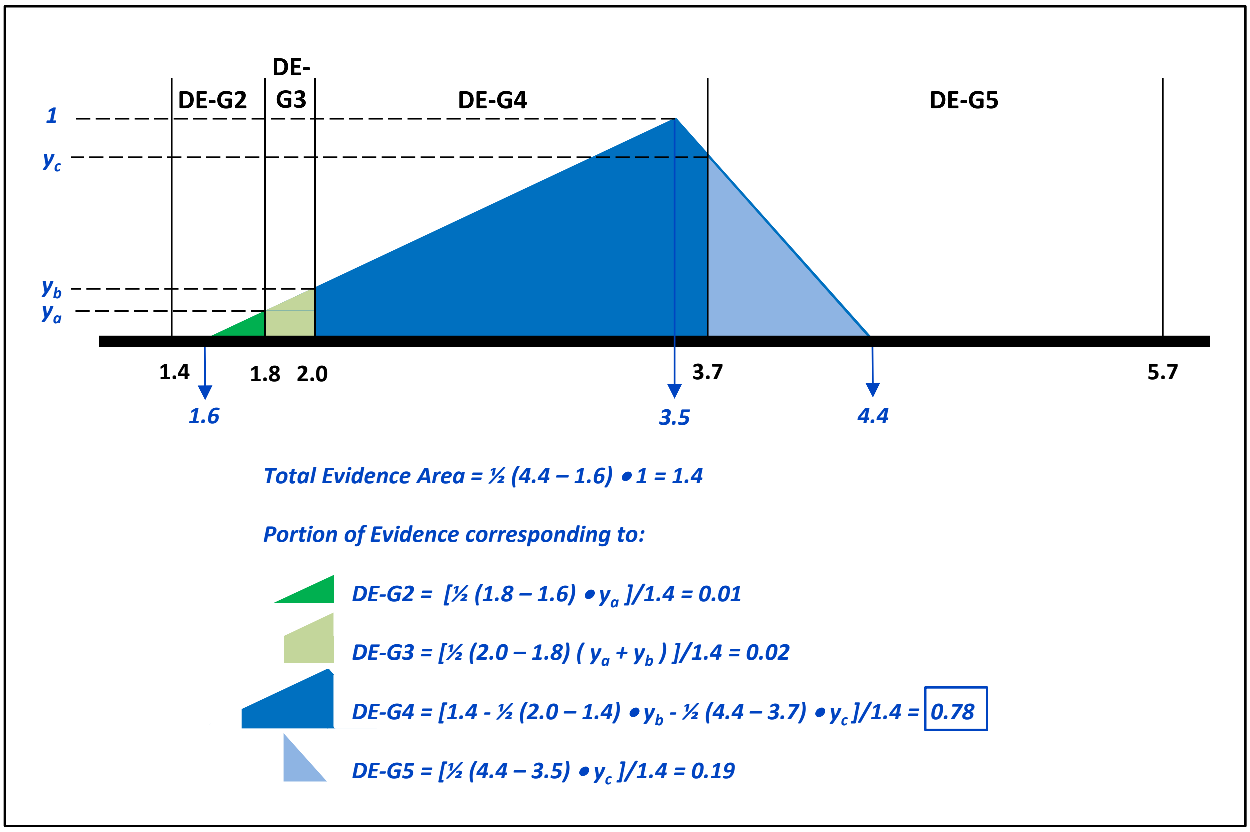


Figure 63. Calculating the Certainty that a Material Falls into a Material Class

(v) Calculate the Cumulative Certainty of Each Material Class

The certainty of a material falling into a material class is calculated for each available test result. Using these certainties, the cumulative certainty is calculated. The equations for this calculation are given in TG2. The most appropriate class for the material is that which has the highest cumulative certainty after all available data are considered. An example is given in Figure 64, where the blue dot represents the median and the red line is between the 10th and 90th percentiles. The cumulative certainty is already adjusted for the certainty factor associated with the test. The cumulative certainty ranges from 0 to 1. In this example, the most appropriate material class is a DE-G5, which has a cumulative certainty of 0.57.

		Test Limits for Material Class				Cumulative Certainty for Material Class			
Test or Indicator	Samples	G4	G5	G6	G7	G4	G5	G6	G7
DCP Penetration	12					0.13	0.29	0.06	0.00
FWD Stiffness	67					0.26	0.32	0.11	0.00
Grading Analysis	3					0.37	0.34	0.11	0.00
% Passing 0.075	3					0.43	0.37	0.11	0.00
Plasticity Index	5					0.46	0.47	0.11	0.00
California Bearing Ratio	2					0.49	0.54	0.16	0.03
Relative Moisture Content	4					0.52	0.57	0.19	0.00

Outcome: Material is most likely a **G5** design equivalent

Confidence: Confidence of the assessment is **medium**. For structural rehabilitation, it is recommended that the sample size and number of test indicators be increased.

Figure 64. Determining the Material Class and Associated Cumulative Certainty

The final certainty value gives an indication of the confidence that can be associated with the outcome. A certainty exceeding 0.7 is considered high, and should be achieved for projects where structural rehabilitation is likely. A low certainty, less than 0.3, is indicative of low confidence and additional data should be gathered to allow a more confident assessment.

15.5 Software

While the material classification system can be implemented in a spreadsheet, there is software available on the Asphalt Academy website, www.asphaltacademy.co.za. The software runs online, and the data are prepared in a Microsoft Excel template and uploaded to the software. Instructions on the method and the software, and the template for importing the data, are on the website.

15.6 Planned Updates

SANRAL has commissioned a revision of the South African Mechanistic-Empirical Design Method. Part of this work will result in the use of new tests or different methods of interpreting common tests. Any such changes will be incorporated into this material classification system for design.

Future upgrades of the material classification system include expanding the assessment of cement stabilized material to also classify the suitability of cement treating an existing material, and the likely material class for such a treated material. This work will be done in 2013 and the results will be published and included in the software. All changes and additions will be detailed on www.asphaltacademy.co.za.

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TRH Revisions

Many of the TRH guideline documents are in the process of being updated. See the SANRAL website, www.nra.co.za for the latest versions.