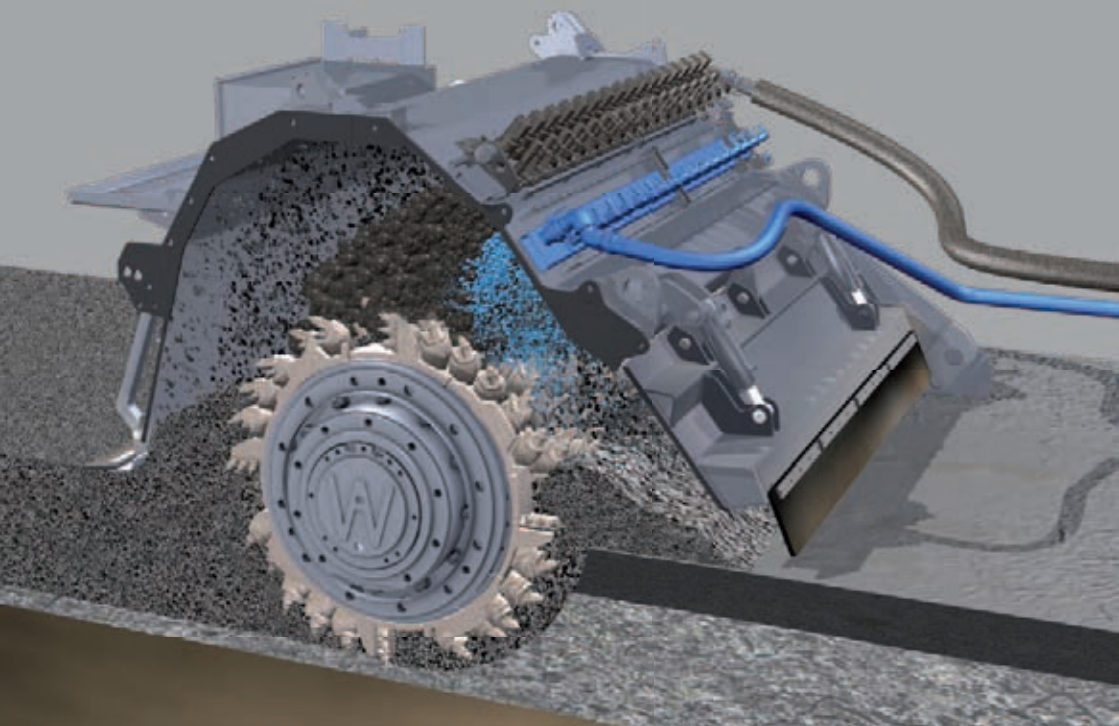
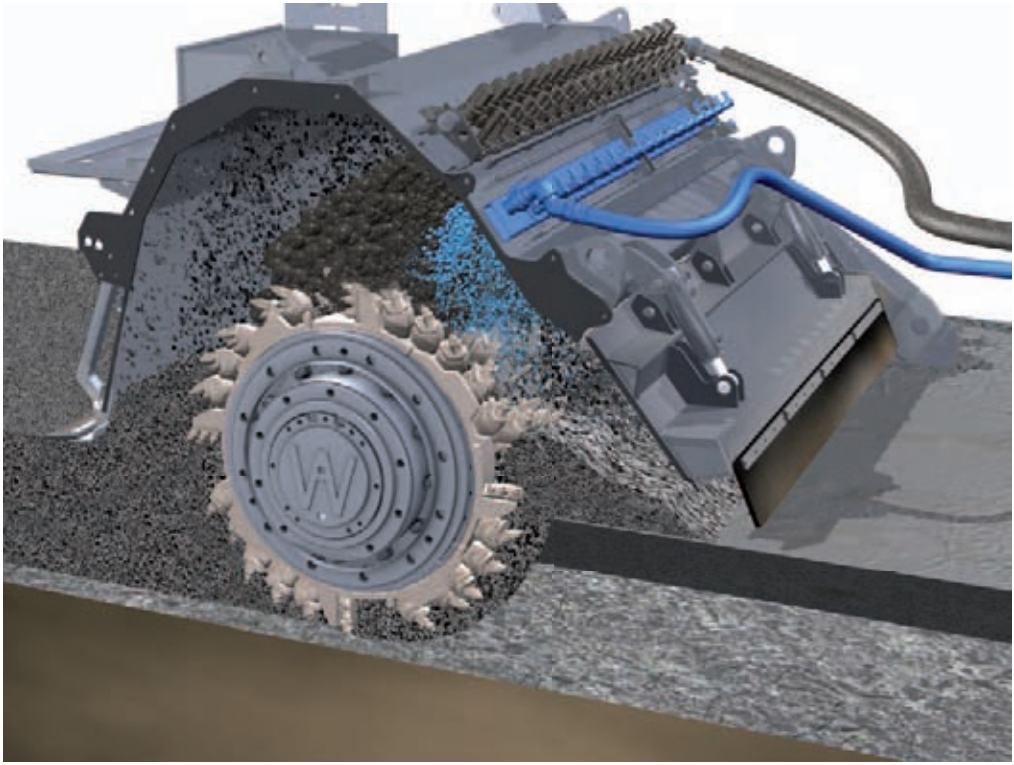
Cold Recycling

Wirtgen Cold Recycling Technology



Cold Recycling

Wirtgen Cold Recycling Technology



Wirtgen GmbH
Reinhard-Wirtgen-Strasse 2 · 53578 Windhagen · Germany

Phone: +49 (0) 26 45/131-0
Fax: +49 (0) 26 45/131-242

1st edition 2012

Acknowledgements

This first Edition of the Wirtgen Cold Recycling Technology manual was compiled by a team of specialists with extensive experience in all aspects of pavement rehabilitation, especially those relating to the reuse of material from existing road pavements.

The team included Engineers from Loudon International who have assisted Wirtgen and their customers in the application of cold recycling technology for nearly twenty years. In recognition of rapid advances being made in the specialised arena of pavement engineering, an invitation to join the team was extended to selected academics from some leading universities, especially Stellenbosch University. Their invaluable contributions are evident throughout the manual, particularly in those chapters concerned with pavement design and stabilising agents. In addition, engineers from Wirtgen GmbH provided valuable guidance in addressing the shortcomings of this publication's predecessor, the Second Edition of the Wirtgen Cold Recycling Manual and concerns previously voiced by customers and field engineers have been addressed in this new publication.

Wirtgen GmbH are grateful to all who contributed to this manual and invite feedback from those who read it. Any comments will be welcomed, regardless of the nature of such comments.
These should be sent to info@wirtgen.de

Preface

For the last twenty years, Wirtgen has been at the forefront of the development of cold recycling technology. During this time, both the technology itself and the machines that carry out the recycling work have evolved from simple beginnings to the current status where cold recycling is recognised worldwide as a normal process for constructing pavement layers, particularly for rehabilitating distressed pavements.

Cold recycling technology is currently employed to construct all types of pavements, ranging from minor access roads to major multi-lane highways. Where a pavement is in a distressed state, there is always an option to recycle the existing material and reap the benefits in terms of lower construction costs, improved durability (service life) and, equally important, a significant reduction in the negative impact that construction has on the environment.

The cold recycling process has been successfully used to rehabilitate and/or upgrade many thousands of kilometres of roads during the past twenty years. The list of projects on which Wirtgen recyclers have been deployed worldwide is exhaustive and covers all climatic regions on all continents (except Antarctica where there are no roads). Pavements in both the Developed and Undeveloped Worlds are increasingly being recycled as a solution to the deteriorating condition of their road networks.

Experience gained from these projects has allowed Wirtgen to actively push the barriers of technology by investing in research and development and trialling new ideas and concepts (e.g. bitumen stabilisation). These developments have all been of benefit to the global industry.

This 1st edition of Wirtgen Cold Recycling Technology is a compilation of lessons learned over the past twenty years. It includes everything that is needed to understand the technology, what it is, where it can be applied and how to design pavements that incorporate cold recycled materials. It is particularly useful for those with little experience in recycling and wish to learn about the technology. However, it is also useful for the more experienced practitioner since it includes advances achieved from the most recent research efforts, especially those in the exciting field of bitumen stabilisation.
In this regard, the technology has evolved from recycling materials that include mixtures of granular, cemented and asphalt materials to recycling material comprised entirely of recycled asphalt pavement (RAP) material.

In publishing this new manual, Wirtgen GmbH wish to share their knowledge and understanding of cold recycling, not only with their customers who have provided many of the experiences, but also with the global road construction community, in the belief that sharing is the path to advancement and a brighter future for all.

Glossary of abbreviations

AADT	Average annual daily traffic (Appendix 2)
AASHTO	American association of state highway and transportation officials
ADE	Average daily equivalent traffic (Appendix 2)
BSM	Bitumen stabilised material (Chapter 4)
BSM-emulsion	BSM made with bitumen emulsion (Section 4.1.4)
BSM-foam	BSM made with foamed bitumen (Section 4.1.4)
CIR	Cold in place recycling (Chapter 6)
CBR	California bearing ratio
CTB	Cement treated base
DCP	Dynamic cone penetrometer (Section 2.5.4)
ELTS	Effective long term stiffness (Section 2.6.4)
EMC	Equilibrium moisture content
ESAL	Equivalent standard axle load (80 kN) (Appendix 2)
FWD	Falling weight deflectometer (Section 2.4.1)
GCS	Graded crushed stone material
HMA	Hot mixed asphalt
HVS	Heavy vehicle simulator
ITS	Indirect tensile strength (Appendix 1)
LTPP	Long term pavement performance
MDD	Maximum dry density
OMC	Optimum moisture content
Pen	Penetration grade (bitumen standard test)
PMS	Pavement management system
PI	Plasticity index
PN	Pavement number (Section 2.6.4)
PWoC	Present worth of cost (Appendix 4)
RAP	Reclaimed asphalt pavement (asphalt millings)
SN	Structural number (Section 2.6.3)
TSR	Tensile strength retained (Section 4.3.11)
UCS	Unconfined compressive strength (Appendix 1)
UTFC	Ultra-thin friction course (asphalt surfacing layer)
WAM	Warm mix asphalt

Content

Content

Introduction

The Wirtgen Cold Recycling Manual was first published in 1998, in English. Due to advances in recycling technology, it was necessary to rewrite the manual after six years and, accordingly, the Second Edition was published in 2004.
The Second Edition was well received; within a couple of years it had been translated into a dozen different languages and, by the end of 2009, more than 50,000 printed copies had been distributed worldwide with at least the same number downloaded from the www.wirtgen.de website.

As with the First Edition, the Second Edition attracted considerable attention with an increasing number of reports, conference papers and other technical publications making reference to the manual. It would appear that the Second Edition of the Wirtgen Cold Recycling Manual has successfully built on the reputation established by its predecessor as the primary reference book for cold recycling technology.

Almost a decade has now passed since the Second Edition was published. During this time, interest in recycling has intensified, reflected by the increasing number of recycling machines that are sold around the world every year. This has encouraged the research and development team at Wirtgen headquarters to continue to improve the machines they manufacture, based on feedback received from their global network of service engineers and customers. Such an increase in field activity succeeded in attracting interest from the academic community with the result that cold recycling technology took a giant step forward thanks to their research efforts. Since 2004, much groundbreaking research work has been carried out, especially concerning bitumen stabilisation, a technology that is ideally suited to cold recycling.

These developments and improvements effectively outdated some sections of the Second Edition. This, coupled with the "reference document" status enjoyed by the Manual warranted a thorough review and update of the contents, a process that highlighted the need for a total rewrite. In addition, the information that needs to be included in the Manual has grown exponentially and is now too much for a single publication. Accordingly, the decision was taken to replace the manual with two publications:

- "Wirtgen Cold Recycling Technology" that focuses on aspects of pavement theory and design that are relevant to cold recycling. Included is a detailed explanation of cold recycling and is particularly useful for engineers involved with material utilization and pavement design.

- "Wirtgen Cold Recycling Application" that covers the practical aspects of applying the technology. This separate publication describes the various construction processes that can be used for cold recycling and is useful for practitioners and field engineers

As with the previous Cold Recycling Manuals, these new publications are focused on recycling "cold" material for use in flexible pavements. They do not include recycling "hot" material, nor do they consider rigid (concrete) pavements, both being separate specialist fields. In addition, they do not include half-warm and warm asphalt. Foamed bitumen technology is ideally suited to such mixes but adaptations to mix evaluations and pavement design are required and are not covered in this manual.

This 1st Edition of Wirtgen Cold Recycling Technology includes the following:

Chapter 1 provides an overview of pavements. The composition of pavement structures is explained along with a brief description of the main factors influencing the selection of the various materials used to construct the different layers and how they behave (and deteriorate) when subjected to dynamic wheel loads. This leads on to the subject of pavement rehabilitation and introduces the concept of cold recycling, both in situ and in plant.

Chapter 2 focuses on pavement rehabilitation and describes the engineering input required to formulate a suitable design, particularly those aspects that are relevant for cold recycling. Pavement investigations, material analyses and pavement designs are all covered in detail in a seven step procedure culminating with a section on economic analyses to assist in evaluating the financial merits of different rehabilitation options.

Chapter 3 explains cold recycling and the various applications that can be considered, both in situ and in plant. The range of Wirtgen recycling machines is introduced along with an explanation of the type of recycling best suited to each machine. Also included is a summary of the benefits to accrue from adopting a cold recycling approach and the suitability of the process for constructing pavement layers, for both new roads and for rehabilitating distressed pavements.

Chapter 4 focuses on the stabilising agents that are normally applied in the cold recycling process. Mix and pavement design procedures for both cementitious and bituminous stabilising agents are explained in detailed. Recent developments in the field of bitumen stabilised materials (BSMs) have been included. These developments have moved the technology forward from that covered in the Second Edition of the Wirtgen Cold Recycling Manual, triggering the need for a rewrite.

Chapter 5, entitled “Recycling Solutions”, uses a “catalogue design” format to show a series of typical pavement structures suitable for rehabilitation by recycling, including both cement and bitumen stabilisation options. Six traffic classes between 300,000 and 100 million equivalent standard axle loads (ESALs) are included, each with different subgrade support conditions that would normally be encountered for such a class. This is followed by an example of different options that can be used to rehabilitate a specific pavement with a structural capacity requirement of 20 million ESALs. An existing (distressed) pavement structure is used to select four different rehabilitation solutions as well as the maintenance requirements for each over a 20-year service life, together with the relevant rehabilitation requirement after 20 years. The whole-of-life cost for each option is then evaluated using different discount rates. In addition, the energy consumed by all construction activities during the service life of each rehabilitation option is evaluated.

Chapter 6 is focused on reusing 100% reclaimed asphalt pavement (RAP) material in a cold recycling process. This subject was not covered in any detail in the previous manuals and has been included to address the increasing interest being shown worldwide in recycling this specific type of material using a “cold process”. (The in situ recycling process is known in some countries as “cold in place recycling” (CIR) or “partial depth reclamation”).

A list of relevant Bibliography is included immediately after Chapter 6.

The four appendices contain a host of additional information, all relevant to cold recycling, but to include them in the chapters would make the manual cumbersome.

Appendix 1 describes the laboratory procedures for stabilised materials (mix designs). This is followed by a schedule of equipment required for carrying out the laboratory work.

Appendix 2 describes the methodology used to determine the correct pavement design criteria (structural capacity requirement) from traffic data.

Appendix 3 includes guidelines for compiling appropriate construction specifications for cold recycling projects.

Appendix 4 provides useful background information for economic analyses.

Pavement rehabilitation is becoming more important as the overall condition of the world's road infrastructure continues to deteriorate and many countries are facing a steady decline in the standard of their ageing road network.

Ever increasing maintenance and rehabilitation efforts required to retain acceptable levels of service place tremendous pressures on national budgets. This situation is exacerbated by the global pattern of growing traffic volumes compounded by increasing axle loads and tyre pressures, factors that all contribute to pavement deterioration. This downward spiral can only be addressed by a massive increase in road budgets coupled with innovation in the field of pavement engineering.

Since few road budgets are increasing in real terms, focus is being placed on innovation to achieve more with relatively less expenditure. Recycling clearly falls into this category and records show that the number of lane-kilometres of distressed pavement being rehabilitated using the cold recycling process is increasing annually. Simple economics is the main reason for this phenomenon as it reflects the cost effectiveness of the process.

1 Road Pavements

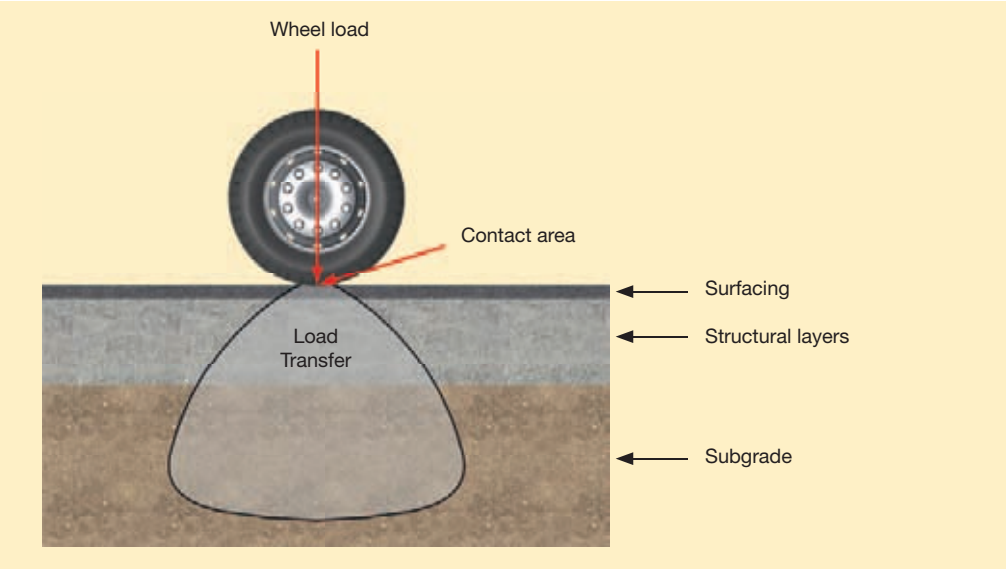
1.1 Pavement structures

Road pavements comprise three basic components:

- Surfacing: The riding surface which is usually the only part of a road that is visible.
- Structural layers: The load spreading layers, consisting of different materials, often extending to depths in excess of one metre.
- Subgrade: The existing "earth" upon which the road is built.

Subgrades are usually of relatively low bearing capacity and cannot carry traffic loading directly, so protective overlying layers are needed. The purpose of the surfacing is predominantly functional providing an all weather riding surface with properties of comfort, safety and environmental consideration (e.g. low noise). The structural layers distribute the high intensity surface loads generated by traffic over a wider area of subgrade, as illustrated in the figure below.

The individual layers in a pavement structure vary in composition (material type) and thickness. Those layers closest to the surface are constructed using high-strength materials (e.g. hot-mix asphalt) to accommodate the higher stresses. Individual asphalt layers seldom exceed 100 mm thickness. As the load is distributed over a wider area in the lower layers, the level of stress reduces and can be carried by poorer quality materials (e.g. natural gravels or lightly cemented materials). Consequently, the materials in the lower layers are



Load Transfer through the pavement structure

generally inexpensive relative to those in the upper layers. The thickness of these individual structural layers generally varies between 125 mm and 250 mm. Section 1.2 below discusses the various pavement components.

There are two fundamental types of pavement:

- Rigid pavements with a thick layer of high-strength concrete overlying a bound layer; and
- Flexible pavements constructed from natural materials with the upper layers sometimes bound (normally by bitumen and/or lightly cemented) to achieve the higher strength requirements.

Generally, only flexible pavements can be economically recycled in-situ. Rigid pavements constructed from high-strength concrete are usually demolished at the end of their useful lives. Although this manual is concerned only with flexible pavements that are characterised by bituminous surfaces, unreinforced concrete pavements have been successfully recycled in situ. (Information on this specialised application can be obtained from Wirtgen.)

Once constructed, a road is subjected to detrimental forces emanating from two primary sources, the environment and traffic. Both act continuously to reduce the riding quality and structural integrity. These destructive forces are discussed in Section 1.3. The sections that follow describe pavement deterioration mechanisms and what can and should be done to retard the process (maintenance), and measures to restore serviceability once deterioration has reached an unacceptable level (structural rehabilitation).

Ideal candidate for in situ recycling

1.2 Pavement components

Each of a pavement's three primary components described above, serves a specific, although different purpose, as explained below.

1.2.1 Surfacing

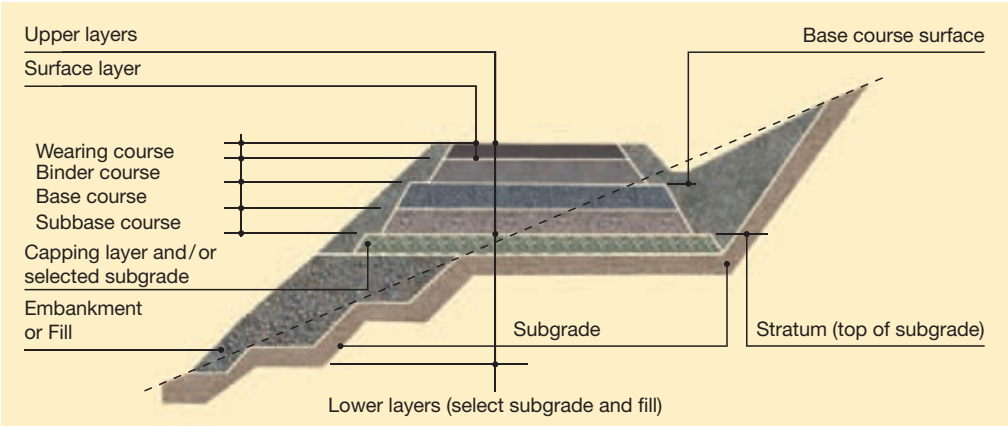
The surfacing is the pavement's interface with the traffic and the environment, its function being to protect the pavement structure from both, providing durability and waterproofing.

Protection from traffic. Traffic affects the surfacing in two ways:

- stresses imparted by wheel loads at the surface are predominantly in the vertical plane, but horizontal stresses can become significant, particularly with the turning and braking actions of traffic and on steep gradients. The strength and stiffness characteristics of the material used in the surfacing must be able to withstand all these stresses without crushing or deforming; and
- abrasion of tyres on the surface, especially whilst cornering, tends to polish the surface. In time this polishing effect reduces the friction properties (skid resistance) and texture depth of the surface. Such surfaces become slippery, especially when wet, and can be hazardous.

Protection from the environment. The surfacing is continually being subjected to various forms of attack from the environment.

Thermal effects, oxidation and ultraviolet radiation are most aggressive. A surfacing therefore needs to have the following properties:



Road design in embankment and cut

- elasticity to allow it to repeatedly expand and contract as the temperature changes; and
- durability to absorb the daily bombardment of ultraviolet radiation, sporadic exposure to water and chemical effects whilst maintaining acceptable performance.

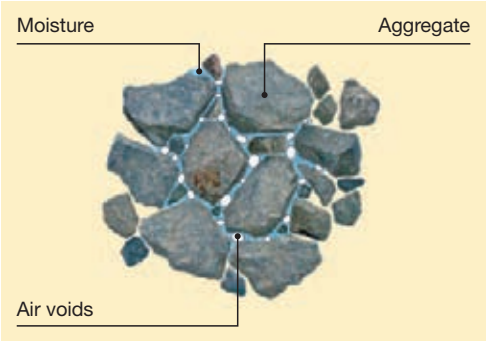
In addition to skid resistance, the bituminous surfacing provides flexibility, durability and superior waterproofing. Hot-mix asphalt (with a bitumen content of approximately 5% by mass) is generally used as a premium surfacing for heavily trafficked roads, whilst the more economical chip-seal surface treatments are applied where the traffic volumes are lower.

1.2.2 Structural layers

The pavement structure transfers the load from the surface to the subgrade. As previously described, the stresses applied by a wheel at the surface are effectively reduced within the pavement structure by spreading them over a wide area of the subgrade.
The pavement structure generally consists of several layers of material with different strength and stiffness characteristics, each layer serving the purpose of distributing the load it receives at the top over a wider area at the bottom.
The layers in the upper part of the structure are subjected to higher stress levels than those lower down and therefore need to be constructed from stronger and stiffer material.
The figure on the previous page shows the different layers that are typically used to construct flexible pavements.

The response of a layer to an imposed load depends largely on the material properties (elasticity, plasticity and viscosity) and the characteristics of the load (magnitude, rate of loading, etc.).

Flexible pavements are constructed from three types of material:



Unbound – Granular

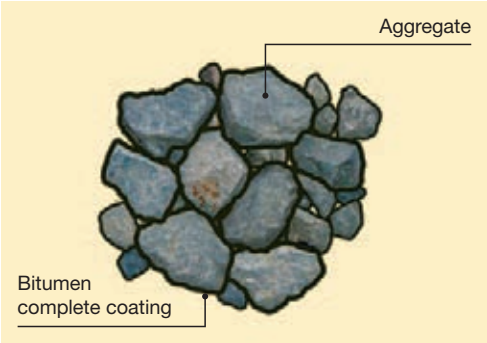
- Unbound (granular) materials, which include crushed stone and gravels, transfer applied loads through the individual particles, or skeleton, of their matrix. Inter-particle friction maintains structural integrity, but under repeated loading (often associated with an increase in moisture content), a gradual densification process occurs as the particles re-orientate and move closer together. This can occur at any level in the pavement structure, ultimately resulting in deformation at the surface. Such deformation is normally manifest as wide radius rutting in the wheel-paths.

- Bound materials, which include cement stabilised materials and asphalt, act more like a wide beam. Applying a vertical load to the surface of a beam generates horizontal compressive stresses in the upper half of the beam and horizontal tensile stress in the lower half, with maximum horizontal stresses at the top and bottom. The strain resulting from these stresses, particularly the tensile strain at the bottom, ultimately leads to a fatigue type of failure after many load repetitions. Cracks develop at the bottom of the layer and then propagate vertically as the load repetitions continue.
- Non-continuously bound materials, which comprise bitumen stabilised materials (BSMs) with either foamed bitumen or emulsified bitumen as binders, behave like granular materials with retained inter-particle friction but increased cohesion, and stiffness. Permanent deformation is the main mode of distress of BSMs. Bitumen is non-continuously dispersed in these materials and fatigue is therefore not a design consideration.

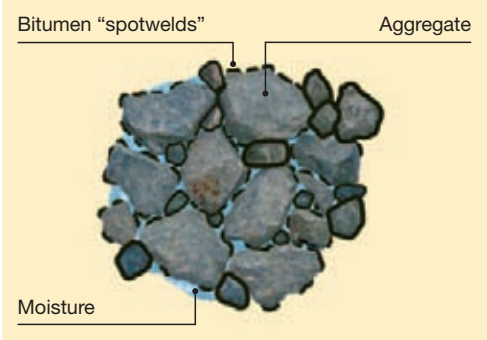
Deformation occurring in unbound and non-continuously bound material and fatigue cracking of bound material are both related to the number of load repetitions. This allows the functional life of a pavement to be determined in terms of the number of times it can be loaded before it "fails", termed the "structural capacity" of the pavement.

Note:

- Bitumen stabilised materials (BSMs) are non-continuously bound



Bound – Hot Mix Asphalt



Non-continuously bound – Bitumen stabilised

1.2.3 Subgrade

The natural material supporting a pavement structure can be either in-situ material (cut condition) or imported (fill condition). The strength characteristics of this material dictate the type of pavement structure required to spread the applied surface load to a magnitude that can be supported without the subgrade failing due to permanent deformation.

Pavement design methods usually use subgrade strength and stiffness as primary input parameters and aim at providing a structure of sufficient thickness and strength to protect the subgrade.

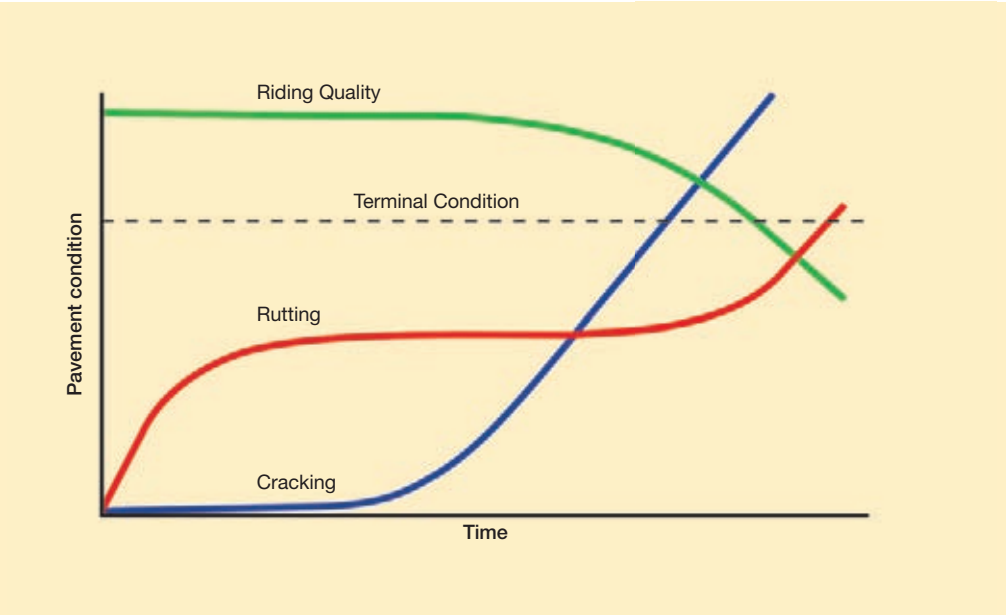
This approach was first adopted in the 1950s with the empirical California Bearing Ratio (CBR) "cover design" method and has endured into the 21st century. In general, thick pavement structures are required to protect poor subgrades and such thickening is often achieved by the addition of "selected subgrade", or "capping" layers.

In some cases, subgrades can comprise collapsible soils, heaving clays, soft/consolidating clays and dispersive/erosive soils. For such conditions, specialist geotechnical investigations, testing and design is required.

1.3 Primary considerations for the pavement structure

Roads are built throughout the world in all types of climate, from hot dry deserts to high rainfall regions and icy tundra conditions. Yet, regardless of the environmental conditions, every road is designed to withstand traffic loading by the same fundamental mechanism of transferring the high intensity forces imparted at the surface by the wheel loads to lower levels that the subgrade can accommodate without deforming.

The specific environmental conditions and anticipated traffic loading are the two primary structural design considerations for any pavement and are discussed separately below. These factors determine the pavement condition and rate of deterioration. Generally, pavement deterioration is measured indirectly by assessing riding quality, but obvious visible features such as rut depth and surface cracking are also relevant. Each mechanism of distress has its own defining performance function versus time as shown in the figure below.



Evolution of Pavement Distress

1.3.1 Environmental conditions

Environmental conditions are considered separately for the surfacing and the underlying structural layers.

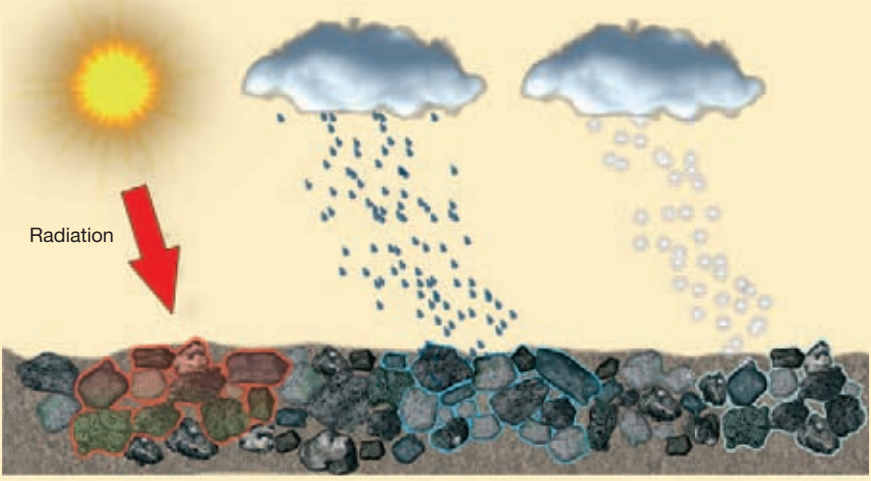
The surfacing. In addition to traffic, road surfaces are exposed to sunshine, wind, rain, snow and other natural elements. Of importance are the consequences of these elements on the engineering properties of the road surface which manifest in:

- thermal effects which cause changes in volume as materials expand and contract in response to changing temperatures. The daily temperature range of the road surface is important. In desert areas, the surface of a blacktopped road can experience a temperature range in excess of 70° C between dawn and noon, whereas road surfaces inside the Arctic Circle during winter will be buried under snow and ice and therefore remain at a relatively constant temperature;
- freezing effects which create the phenomenon known as frost heave. Repeated freeze/thaw cycles can cause major damage to road surfaces;
- radiation effects which cause road surfaces to experience a type of “sunburn”. The ultraviolet radiation to which the road surface is subjected causes the bitumen to oxidise and become brittle. This process is known as ageing; and
- moisture effects where rainfall penetrates the voids of the surfacing and builds up pore pressures under wheel loading, breaking the bond between the bitumen and the aggregate, leading to stripping and ravelling of the asphalt.

The pavement structure. Water is the greatest enemy of road structures. Water saturation causes materials to soften and deteriorate, and also provides inter-particle lubrication when load is applied. The bearing capacity of a material in a dry state is significantly greater than in a wet state and the more cohesive (or clayey) the material, the more susceptible it is to moisture. In addition, water that is present when frost progresses into a pavement structure will expand and cause extensive damage when it thaws. Hence the importance of preventing water ingress into a pavement structure, especially into the poorer quality materials found in the lower layers.

Environmental factors are responsible for most cracking that initiates at the surface. The major contributor to this phenomenon is ultraviolet radiation from sunlight that causes a continuous slow hardening of the bitumen. With hardening comes a reduction in elasticity that results in cracking when the surface contracts as it cools or flexes under wheel loads.

Once the surface integrity has been lost due to cracking, the pavement tends to deteriorate at an accelerated rate due to water ingress. The primary environmental factors effecting pavements are shown in the figure below.



Environmental effects

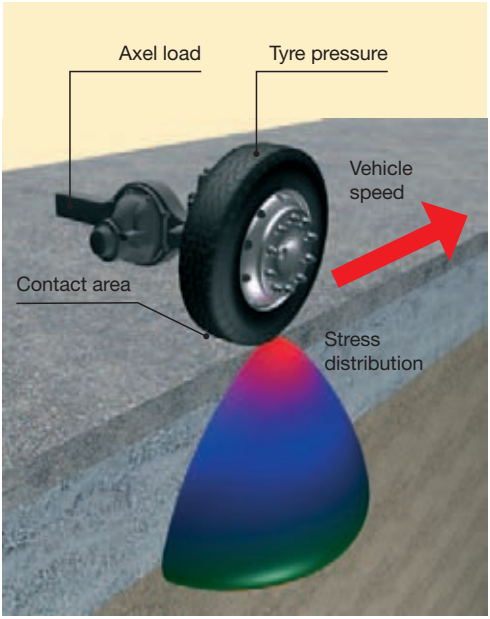
1.3.2 Traffic loading

Roads are constructed to carry traffic. The volume and type of traffic that a road is expected to carry dictates the geometric and structural requirements. Transportation engineers work with anticipated traffic statistics (in terms of vehicle numbers, composition and sizes) in order to determine the geometric capacity requirements (alignment, number of lanes, etc.). Pavement engineers need anticipated traffic statistics (in terms of vehicle numbers, configuration and axle mass) to determine the structural requirements. Accurate predictions of future traffic volume and type is therefore of paramount importance.

Important features of the traffic from a pavement design perspective are those that allow definition of the magnitude and frequency of surface loads that the road can anticipate during the expected life of the pavement. The load that is imparted on a road surface by a tyre is defined by three factors:

- force (in kN) actually carried by the tyre which, together with
- inflation pressure (in kPa) determines the "footprint" of the tyre on the road. This footprint defines the area on the surface that is subjected to the load, and
- speed of travel that defines the rate at which the pavement is loaded and unloaded.

Passenger cars typically have tyre pressures in the range of 180 to 250 kPa and support less than 350 kg per tyre, or 7 kN on an axle. This loading is structurally insignificant when compared to that imparted by a large truck used for hauling heavy loads, which range between 80 and 130 kN per axle (depending on legal limits and mass control) with tyre pressures ranging between 500 to 1,300 kPa. Clearly, the loading of such heavy vehicles will have the greatest influence on the strength requirements of a pavement and is therefore discussed in Chapter 2, Pavement Rehabilitation and covered in detail in Appendix 2, Determining Structural Capacity from Traffic Information.



Traffic loading

1.4 Pavement distress mechanisms

Traffic loading is responsible for the development of ruts and for cracking that initiates within the bound layers. Every vehicle using a road causes a small measure of deformation in the pavement structure. The deformation caused by a light vehicle is so small that it is insignificant whilst heavily loaded vehicles cause relatively large deformations. The passage of many vehicles has a cumulative effect that gradually leads to permanent deformation and/or fatigue cracking. Overloaded axles cause a disproportionate amount of damage to the pavement structure, accelerating such deterioration. This deterioration is caused by two different mechanisms, namely:

- permanent deformation caused by densification, where stresses from repeated loading cause the individual particles within the pavement layer to move closer together, resulting in a loss of voids or shearing of particles past each other (localised shear failure). In granular and non-continuously bound material, such a loss of voids leads to an increase in strength (denser materials are stronger).
  In asphalt the converse applies. It must, however, be appreciated that a reduction in the void content in asphalt not only causes rutting in the wheel paths, but it also allows the bitumen to start acting as a fluid when warm, creating a medium for hydraulic pressures to be generated from the imposed wheel loads. This causes lateral displacement, or shoving along the edges of ruts; and
- fatigue cracking of bound materials. These initiate at the bottom of the layer where the tensile strain caused by wheel loads is at its maximum. These cracks then propagate to the surface. Top down cracking can occur in thick asphalt layers. Permanent deformation of the underlying material exacerbates cracking by effectively increasing the tensile strain imposed by wheel loads.

1.4.1 Advanced pavement distress

Once a crack penetrates through the protective surfacing, water can ingress into the underlying pavement structure. As previously described, the softening effect of water leads to a reduction in strength that results in an increased rate of deterioration under repeated wheel loads.

In addition, water in a saturated material becomes a destructive medium when the pavement comes under load. Similar to a hydraulic fluid, the water transmits predominantly vertical wheel loads into pressures that rapidly erode the structure of a granular material and causes stripping of bitumen from the aggregate in asphalt. Under these conditions the fines fractions of the pavement material are expelled upwards through the cracks (known as “pumping”) resulting in voids developing within the pavement. Potholing and rapid pavement deterioration soon follows.

Typical pavement distress with pumping

Where temperatures drop below 4°C, any free water in the pavement expands as it freezes, creating hydraulic pressures, even in the absence of imposed wheel loads. Frost heave caused by repeated freeze/thaw cycles are the worst scenario for a cracked pavement, resulting in rapid deterioration.

Under dry desert conditions, cracks in the surfacing lead to a different type of problem. At night when temperatures are usually relatively low (often below freezing) the surface contracts, causing the cracks to widen and act as a haven for wind-blown sand. When temperatures rise during the day, the surface is restricted from expanding by the sand trapped within the crack, resulting in large horizontal forces that cause localised failure (spalling) at the edge of the crack.

These forces can ultimately lead to the surfacing lifting off the pavement structure in the vicinity of cracks, making for extremely poor riding quality.

Another failure condition often seen in desert environments is block cracking caused by an extremely low moisture content in dense material. This phenomenon is known as “pore fluid suction pressure”. Due to the low relative humidity regime, water is lost from the pavement structure due to evaporation, reducing the moisture content to levels similar to those achieved when air drying samples in the laboratory. At such low moisture contents, the menisci of the tiny water droplets remaining within the small voids of a compacted material exert sufficient tensile forces to cause the material to crack.

This condition is likely to manifest wherever relative humidity levels are low and the road is unsealed, allowing the moisture in the pavement structure to evaporate. It has also been cited as the cause of top-down cracking at high altitudes (> 2,000 m). The only effective treatment for this condition is to seal the road so that the equilibrium moisture content is retained (i.e. the hydro-genesis effect). If the material is allowed to dry out, severe and deep cracking will occur, even in compacted sand. If the material is allowed to wet up, capillary forces will reduce and the “apparent cohesion” will dissipate.

One further cause of surface cracking, particularly in thin asphalt surfacings, results from the lack of traffic. The kneading action of traffic keeps bitumen alive. Oxidation and subsequent hardening causes thermal cracks to initiate at the surface of the bitumen binder. Subjecting the bitumen to stress repetitions causes sufficient strain to close the cracks as they form, thereby preventing them from propagating.

1.5 Pavement maintenance and structural rehabilitation

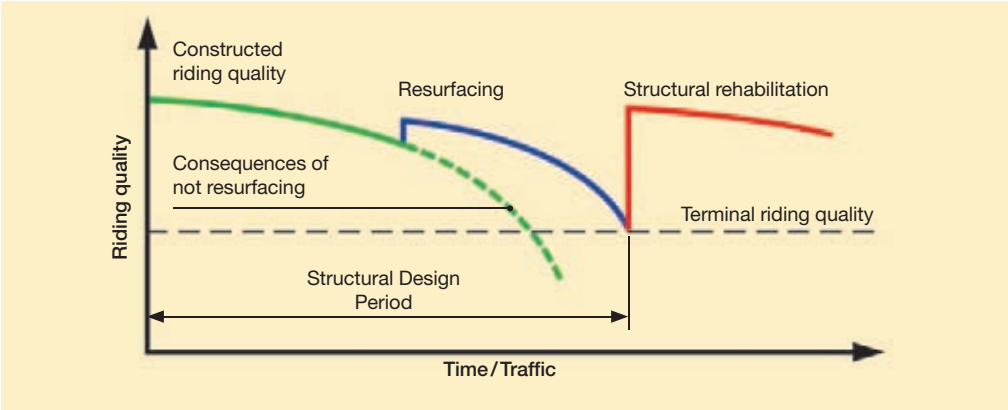
Pavement maintenance activities are normally focused on keeping water out of and away from the pavement structure. This involves maintaining the surfacing in a waterproof state and ensuring that drainage measures are effective so that water cannot pond on the road surface or along the road edge.

Water normally ingresses the upper pavement structure through cracks in the surfacing, often assisted by water ponding on the surface. Cracks should therefore be sealed as they appear and road verges trimmed to promote runoff. If addressed early, ageing effects can be effectively treated by the application of a light fog spray of dilute bitumen emulsion. More serious conditions require a chip seal application where traffic volumes are low, or a conventional hot-mix asphalt overlay.

Such measures, which are aimed at maintaining the flexibility and durability of the surfacing, only really address deterioration due to the environment. Deformation and fatigue cracking caused by traffic loading cannot be treated effectively by superficial maintenance activities and require some form of structural rehabilitation.

Pavement deterioration begins at a relatively slow pace. Pavement indicators can be used to monitor the rate of deterioration. Road authorities often employ a data-base system, known as a Pavement Management System (PMS), to continuously monitor the riding quality of all road pavements within their network, thereby drawing attention to those that most require attention. The figure below provides a typical PMS plot that illustrates the effectiveness of timely maintenance and rehabilitation measures.

This figure highlights the importance of taking timeous action to maintain as high a riding quality as possible. The rate of deterioration is indicated by the riding quality. Poorer riding quality encourages faster rates of deterioration through dynamic loading. As the riding quality reduces, the scale of



Managing pavement maintenance and rehabilitation by monitoring riding quality

remedial measures becomes greater, as does the cost of such measures.

The decision as to which remedial measures to undertake to either improve a pavement or just maintain it at its current riding quality is often dictated by budgetary constraints. Short-term holding measures can be extremely cost effective. Pavement rehabilitation is sometimes postponed until it is combined with an upgrading exercise to improve the geometrics of the road and add additional lanes. Each rehabilitation decision needs to be taken independently within the context of the overall road network. But, to do nothing and allow the pavement to deteriorate further is generally the worst decision because of the exponential rate of deterioration with time.

1.6 Rehabilitation options

There are usually many options available for the rehabilitation of a distressed road and sometimes it is difficult to determine which is the best. However, the answer to two important questions that must be asked at the outset will assist in selecting the "correct" one, the one that is most cost-effective in meeting the road owner's needs. The two important questions are:

- what is actually wrong with the existing pavement? A cursory survey consisting of a visual inspection coupled with a few basic tests (eg. deflection measurements) will normally be sufficient to be able to understand the distress mechanism. Of importance is to determine whether the distress is confined to the surfacing (upper pavement layers) or whether there is a structural problem; and secondly
- what does the road authority really want and what can they afford? Is a 15-year design life expected, or is a smaller capital outlay envisaged that will arrest the current rate of deterioration and hold the pavement together for a further five years?

The answers to these two questions will narrow down the rehabilitation options to only those that will be cost-effective within the context of, essentially, the nature of the problem and the time frame. By separating the nature of the problem into two categories (surface and structural) from the time frame (short-term or long-term), selecting the best option is simplified.

One other important point that affects the decision is the practicality of various rehabilitation methods. Traffic accommodation, weather conditions and availability of resources can all have a significant influence on how a project is executed and may preclude certain options.

This whole exercise has one sole purpose: determining the most cost-effective solution to the actual problem within the context of the project environment.

Note:

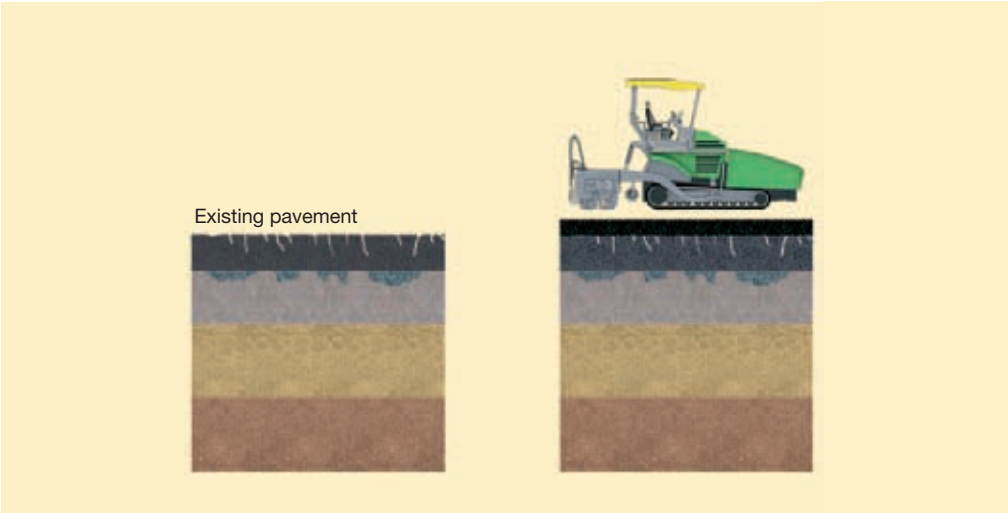
- Rehabilitation design must address the root cause of distress cost effectively

1.6.1 Surface rehabilitation

Surface rehabilitation measures address problems that are confined to the uppermost part of the pavement, usually within the top 50 mm to 100 mm. These problems are normally related to aging of bitumen and cracking that initiates at the surface due to thermal forces.

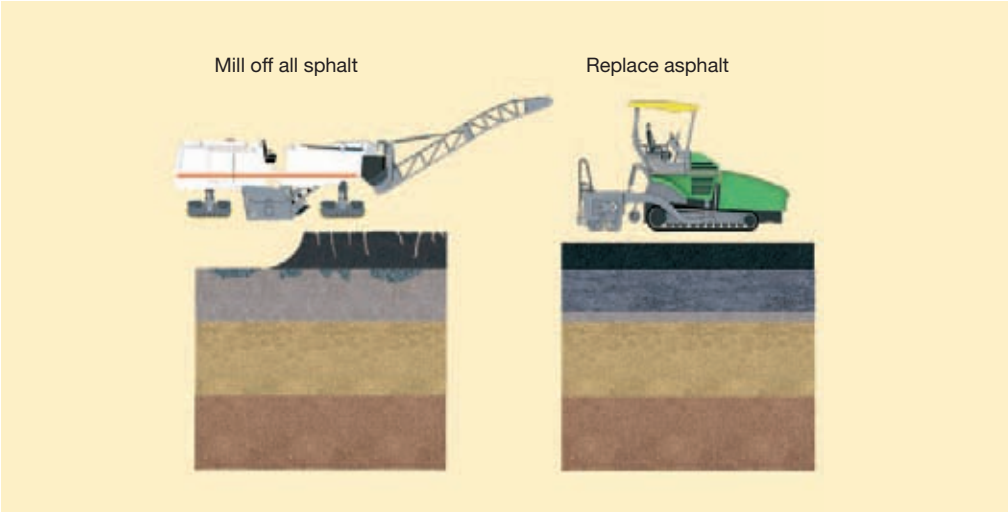
The most commonly used methods for dealing with this type of problem include:

- **Asphalt overlay.** Paving a thin (40 – 50 mm) hot-mix asphalt overlay on the existing surface. This is the simplest solution to a surface problem since the time required to complete the work is short and there is minimal impact on the road user. Modified binders are often used in the asphalt to improve performance, thereby extending the life of the overlay. Active cracks in the existing surface will reflect quickly through a new overlay and therefore need to be identified and addressed either by applying a stress-relieving bandage or by patching. Repeated overlays, however, increase road surface elevations that can cause drainage and access problems.



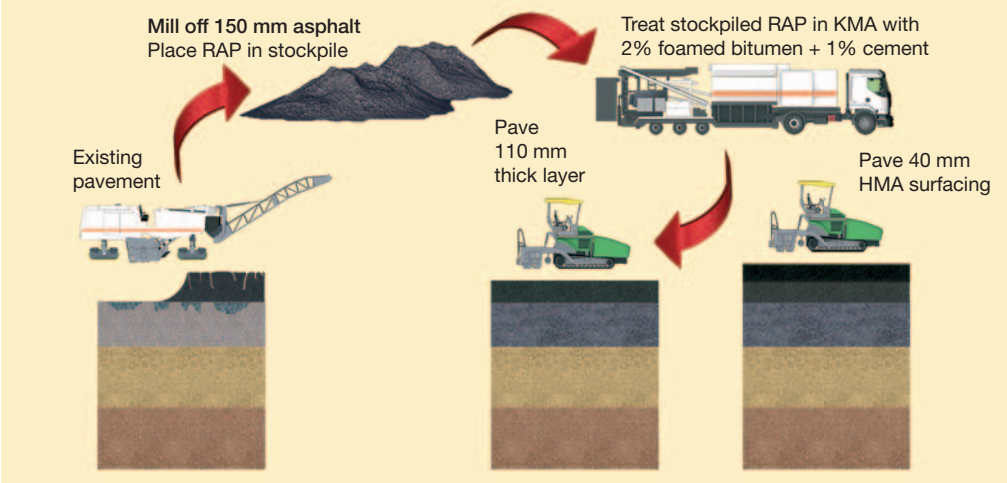
Asphalt overlay

Mill and replace. This method removes the offending cracked layer of asphalt and replaces it with fresh hot-mix asphalt, often with a modified binder. The process is relatively fast due to the high production capabilities of modern milling machines. The problem is removed with the layer of asphalt and pavement levels are maintained.

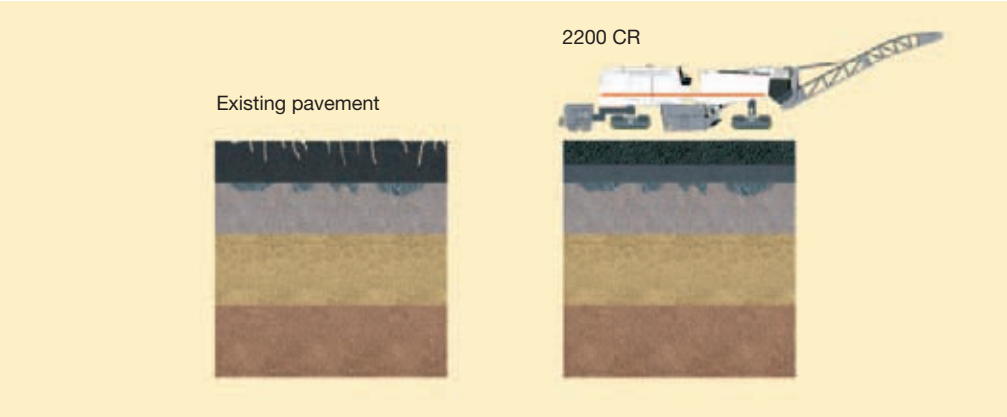


Mill & Replace

Recycling a relatively thin (100 mm – 150 mm) layer of asphalt material from the existing pavement. Such recycling can be done either "in-plant" by transporting milled material to a cold mixing plant KMA 220 or in place by using the 2200 CR or WR 4200.



Recycle upper 150 mm



Recycle upper 100 mm

1.6.2 Structural rehabilitation

Rehabilitation to address problems within the structure of a pavement is normally treated as a long-term solution. When addressing structural problems, it should be remembered that it is the structure of the pavement that is distressed, seldom the materials within the structure. In addition, upgrading an existing pavement by strengthening the structure (e.g. upgrading an existing gravel road to blacktop standards) may be regarded as a form of rehabilitation.

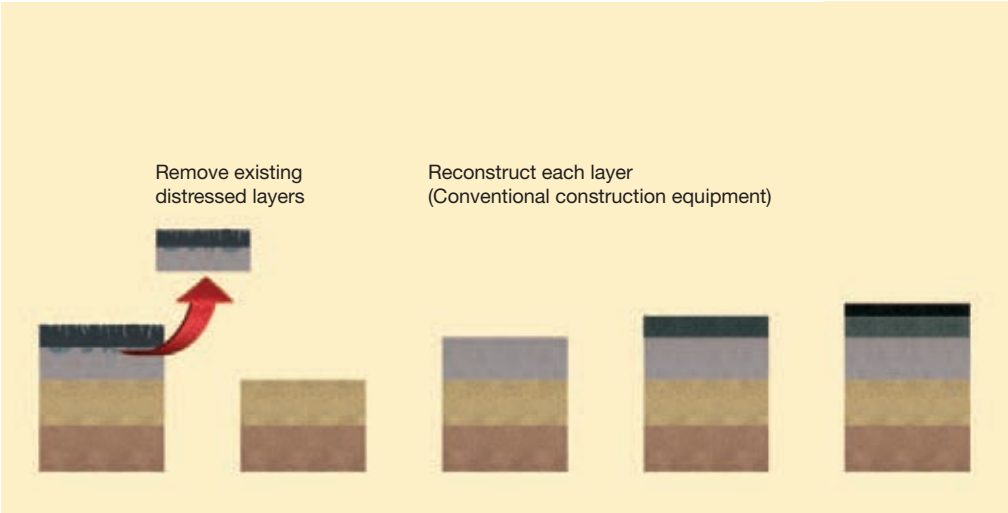
Densification (or compaction) of granular materials is, in fact, a form of improvement since the higher density of a natural material leads to better strength characteristics. However, the consequences of densification and the resulting deformation can lead to problems manifesting in the overlying layers, especially where such layers are constructed from bound material.

As a rule, structural rehabilitation should aim to maximise the salvage value of the existing pavement. This implies that material that has densified should not be disturbed. The continuous kneading action of traffic takes many years to achieve this state and the benefits that such high densities offer should be utilised where possible.

Various options that are popular for structural rehabilitation include:

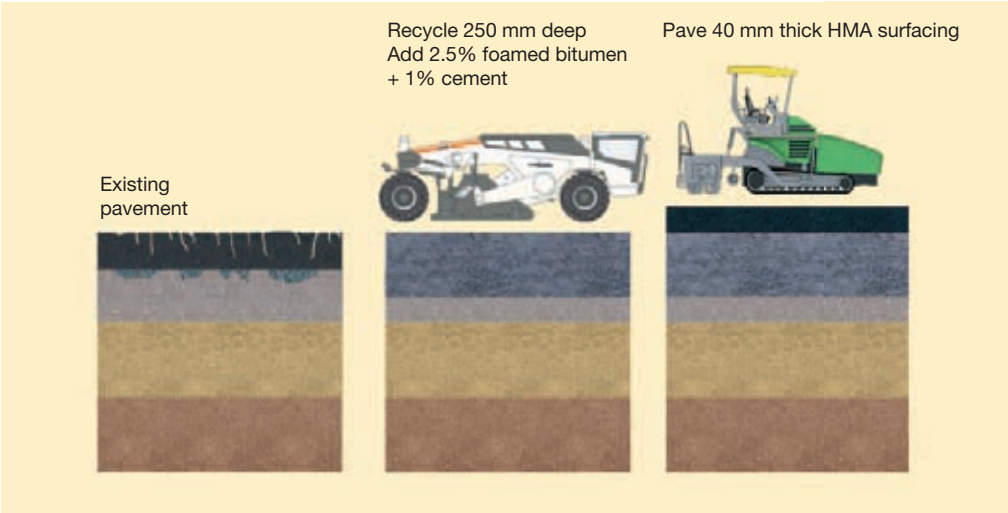
- **Total reconstruction.** This is often the preferred option when rehabilitation is combined with an upgrading exercise that demands significant changes to the alignment of the road. Essentially, reconstruction implies “throw-away-and-start-again”. Where traffic volumes are high, it is often preferable to construct a new facility on a separate alignment, thereby avoiding traffic accommodation problems.

- **Construction of additional layers** (either from untreated or treated granular material and/or asphalt) on top of the existing surface. Thick asphalt overlays are often the easiest solution to a structural problem where the traffic volumes are high. However, as described above, an increase in surface elevations often gives rise to unforeseen drainage and access problems.



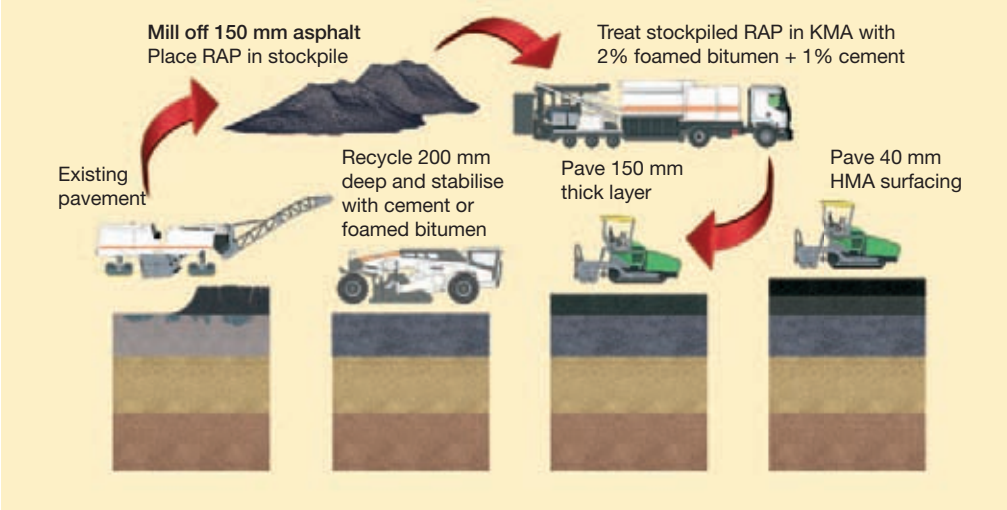
Total reconstruction

Deep recycling to the depth in the pavement at which the problem occurs, thereby creating a new thick homogeneous layer that can be strengthened by the addition of stabilising agents. Additional layers may be added on top of the recycled layer where the pavement is to be significantly upgraded. Stabilising agents are usually added to the recycled material, especially where the material in the existing pavement is marginal and requires strengthening. Recycling aims for maximum recovery from the existing pavement. In addition to salvaging the material in the upper layers, the pavement structure below the level of recycling remains undisturbed.



Deep recycling

- **Combining two recycling methods**, in-place with in-plant. This option allows for an increased depth of existing pavement to be treated and requires that a portion of the upper pavement be initially removed and placed in temporary stockpile. The underlying material is then recycled/stabilised in-place. The material placed in temporary stockpile is then treated in-plant and paved on top of the in-place recycled layer, thereby achieving additional structural capacity. The thickness of the paved layer can be selected to suit the final surface level requirements. For example, where existing surface levels are to be maintained after rehabilitation and a 40 mm thick asphalt surfacing is required, the thickness of paving for the upper stabilised layer is reduced by 40 mm to allow the final surface levels to match those prior to rehabilitation.



Two -part recycling

The purpose of considering several pavement rehabilitation options is to determine the most cost-effective solution. This manual is aimed at providing sufficient information together with a design approach that will allow recycling to be included as one of the options. Economic evaluations of the different options will then help identify the optimal solution, as discussed in the next chapter.

2 Pavement rehabilitation

As was explained in the previous chapter, pavements deteriorate with time and usage. As the end of the service life approaches, the rate of deterioration in the riding quality increases due to deformation, cracking, potholing and other such distress symptoms. The maintenance efforts necessary to hold the surface together and maintain an adequate level of service tend to escalate until the point is reached where it makes more economic sense to rehabilitate the whole pavement rather than addressed localised areas of distress. Although there are no definitive guidelines and each road has its own unique features, there appears to be general consensus that once 15% of the surface area has been patched, it is cheaper to rehabilitate the whole road than to continue applying patches on an ad hoc basis.

Heavy patching indicates the end of the service life

2.1 General

Pavement rehabilitation is the term used to describe the work required to reinstate a distressed road and restore the structural integrity of the pavement. Where a road is properly designed and constructed and where routine maintenance and resurfacing interventions are undertaken timeously, the need for rehabilitation can be delayed until the pavement reaches a terminal condition due to structural deterioration. However, in practice, such maintenance and resurfacing activities are often not carried out, resulting in the need for the pavement to be rehabilitated sooner than was originally envisaged. In addition, pavement rehabilitation is often included with strengthening and/or geometric improvements required to accommodate increased traffic volumes.

This chapter describes the various steps that are included in a pavement rehabilitation exercise. These are explained and guidelines given in order to provide a practical overview of an extremely involved (and at times complex) process that has recently become an area of specialisation in the field of Pavement Engineering. These explanations and guidelines are certainly not all-inclusive and reference should be made to the literature included in the Bibliography (after Chapter 6) should more detailed information be required.

The various steps in a pavement rehabilitation exercise include gathering of relevant information (e.g. traffic data), conducting surveys and tests to identify and determine the composition and condition of the various layers in the existing pavement structure (e.g. deflection measurements), summarising and interpreting all available data to allow alternative design options that meet the required design life (structural capacity) to be formulated and, finally, to decide which option is most attractive. Although these steps are common to all pavement rehabilitation exercises, the focus of this chapter is on identifying and understanding the materials in the upper pavement structure and their potential for being recycled.

A flowchart is included to illustrate the various steps and methods used in the pavement rehabilitation exercise. Methods that are normally used for investigating distressed pavements are outlined. Different methods for designing pavements are described, particularly those most suited to pavements that are rehabilitated by recycling.

2.2 Pavement rehabilitation: investigation and design procedure

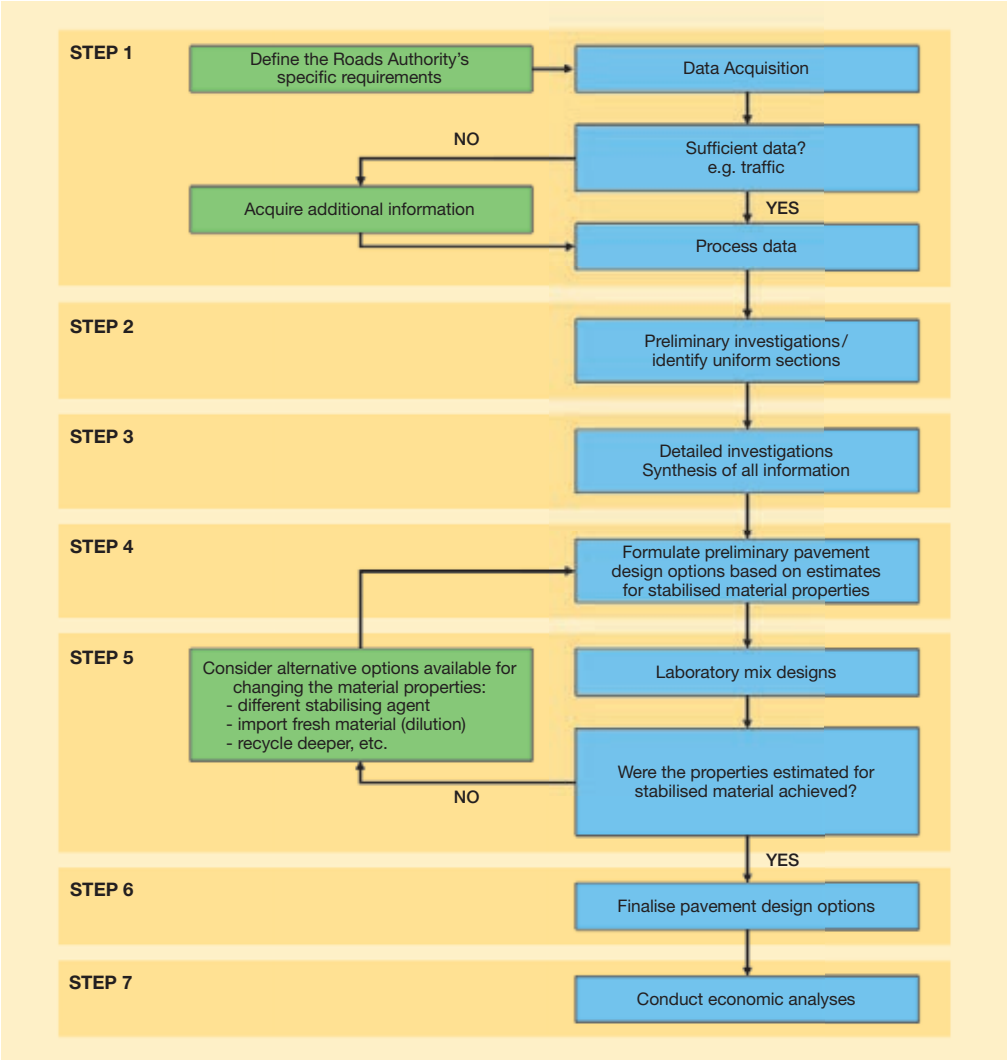
The need for rehabilitation is usually triggered by an unacceptable level of distress that reflects as a poor surface condition (e.g. bad riding quality, pothole development, etc.), often highlighted and prioritised by using an appropriate Pavement Management System (PMS). Once a road has been identified for rehabilitation, a full pavement investigation and design procedure needs to be followed to determine the most appropriate rehabilitation solution.

The primary objectives for investigating an existing pavement are to determine the composition of the pavement structure, gain an understanding of the behaviour of the materials in the various layers and establish the cause of distress that has increased the demand for maintenance measures.

The flowchart on the facing page is applicable to all rehabilitation projects and can be adapted according to specific needs. The various activities are grouped under seven sequential steps:

Step 1. Acquisition of available information
Step 2. Preliminary investigations and identification of uniform sections
Step 3. Detailed investigation of each uniform section and synthesis of all data
Step 4. Preliminary pavement design options based on estimates of stabilised material properties
Step 5. Laboratory mix designs to determine stabilised material properties
Step 6. Pavement design finalisation
Step 7. Economic and other analyses to indicate the optimal solution

The following sections describe each step in detail.


STEP 1
Define the Roads Authority's specific requirements
Data Acquisition
NO
Sufficient data? e.g. traffic
Acquire additional information
YES
Process data
STEP 2
Preliminary investigations / identify uniform sections
STEP 3
Detailed investigations Synthesis of all information
STEP 4
Formulate preliminary pavement design options based on estimates for stabilised material properties
STEP 5
Consider alternative options available for changing the material properties:
- different stabilising agent
- import fresh material (dilution)
- recycle deeper, etc.
Laboratory mix designs
Were the properties estimated for stabilised material achieved?
NO
YES
STEP 6
Finalise pavement design options
STEP 7
Conduct economic analyses

2.3 STEP 1: Data acquisition / process available information

At the start of every pavement rehabilitation exercise, those responsible for formulating the design must have an unambiguous understanding of what is required in terms of:

- Design life. Is a short-term or long-term service life required?
- Functional properties of the rehabilitated road (e.g. specific requirements for riding quality, skid resistance and noise levels)
- Available budget. The level of funding available for the rehabilitation works and for the routine maintenance measures that will be required during the service life.

These requirements provide the design engineer with the scope and limits of the project. The investigation phase commences by sourcing all information on the existing pavement that is available. This information falls under two major headings described below:

- records of historical information; and
- traffic data to determine the structural capacity requirements.

2.3.1 Information on the existing pavement (historical information)

All available information should be sourced and analysed in order to place the project in context and provide an early appreciation of what can be expected when starting the field investigations. Where available, construction and maintenance records can provide valuable information on:

- details of the pavement that was originally constructed;
- the thickness of as-built layers;
- details of materials used in the construction of the original layers as well as those used in any subsequent rehabilitation or improvement measures;
- results of quality control tests conducted during construction; and
- geological data along the route.

In addition, as much information as possible should be obtained about locally available construction materials. The type, quality and quantity of materials that can be obtained from both commercial sources and previously opened borrowpits and quarries should be investigated for possible use in the rehabilitation works. Also, the location and distance from site of any established asphalt plant should be established.

Furthermore, meteorological records from the closest weather station should be obtained and analysed to determine seasons that are best suited for the type of construction that is envisaged.

The proximity of locally available materials will influence rehabilitation options

2.3.2 Design traffic

The volume and type of traffic that a road is expected to carry during its service life dictates the structural requirements. Pavement engineers therefore need traffic statistics (in terms of vehicle numbers, configuration and axle loads) to estimate the anticipated structural capacity requirements for rehabilitation.

Structural capacity requirements
The structural requirements of a pavement are referred to as the "structural capacity" that defines the loading a pavement can withstand before it deteriorates to the stage that a "failure condition" is reached. Structural capacity is expressed in terms of "equivalent load" repetitions with the load carried on a single axle expressed in tons (or kN), usually in accordance with the maximum legal axle load that varies between 8 tons and 13 tons, depending on the country. Different axle loads are related to this maximum to obtain an "equivalent" value. For example, where the maximum legal axle load is 8 tons, a single axle load of 4 tons would be between 0.1 and 0.3 "equivalent 80 kN axle loads", depending on the function used to relate a 4 ton load to an equivalent 8 ton load.
(This relationship function is also known as the "damage factor").

The term "equivalent **standard** axle load" (ESAL) originated with the AASHO Road Tests carried out in the USA during the late 1950s that marked the advent of pavement engineering. The "standard" axle load referred to in ESALs is 8 tons or 80 kN and, as explained above, different axle loads are related to an equivalent number of 80 kN axle loads with the structural capacity of a pavement expressed in terms of millions of ESALs.

Where the legal axle load is different from 8 tons (e.g. 11 tons in Germany), the structural capacity of a pavement is usually defined in terms of "equivalent axle load" repetitions with the axle load stated in kN (e.g. 25 million equivalent 110 kN axle loads). Using an appropriate damage factor, the number of equivalent 110 kN loads can be converted to ESALs (e.g. 25 million equivalent 110 kN axle loads would translate to some 90 million ESALs).

Traffic spectrum comprised of many different vehicle types

Structural capacity is often referred to as the "design traffic" or the "bearing capacity" of a pavement and, provided they are all quoted in terms of millions of equivalent axle load repetitions, they are the same.

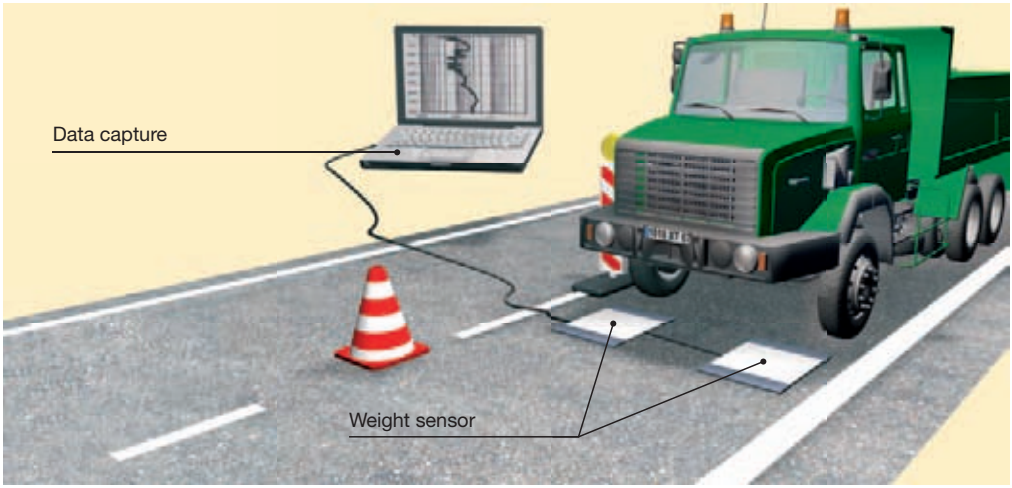
Note:

- Design traffic
  Accurate traffic data and growth information are essential

Pavements are therefore designed to provide for a specific structural capacity. Although a design life is often quoted in years, pavements are actually designed to accommodate the number of load repetitions that are anticipated during that period. Any unforeseen changes in the estimated traffic loading will therefore impact on the design life. This is one of the most fundamental aspects of pavement engineering and is so important that a full explanation is included in Appendix 2, entitled "Determining Structural Capacity from Traffic Information".

Obtaining additional traffic information
Where the available traffic data is insufficient, particularly when designing the rehabilitation of heavy pavements, additional information must be obtained. Classified traffic counts should be undertaken and weigh-in-motion data obtained to estimate the percentage of heavy vehicles currently using the road, the average number of axles per heavy vehicle and the average mass carried on each axle. This information should be supplemented wherever possible with information obtained from weighbridge stations (including the results of any tyre pressure surveys).

It must always be borne in mind that the information used to calculate structural capacity is based on assumptions concerning traffic growth rates, damage factors and other data that can only be estimated. It is therefore important to carry out sensitivity analyses to understand the consequence of varying these estimated parameters.



Weigh-in-motion station to measure and record axle loads

2.4 STEP 2: Preliminary investigations

Before any field surveys or investigations commence, it essential that the existing road surface is accurately pre-marked with an appropriate referencing system (usually the chainage or km distance (e.g. km 121 + 400) is adopted). It is normal to paint prominent marks every 20 m on the centre-line or outer edge of the carriageway and to write the chainage value every 100 m. These marks are then used as the primary reference for all survey and test locations.

Road pavements are seldom uniform over long distances. Both the underlying geology and the materials used in the construction of the individual layers will vary along the length of the road. All roads are therefore comprised of a series of different sections of relative uniformity and the length of each section will be different. These sections are known as “uniform sections” and may be as short as a few hundred metres or as long as several kilometres. Uniform sections are identified visually by changes in distress patterns. Deflection measurements are also useful for identifying differences in the underlying pavement structure.

2.4.1 Determination of uniform sections

One of the main objectives of undertaking preliminary investigations is to identify uniform sections. This is usually achieved by analysing available construction records, analysing any deflection data, and from conducting a comprehensive visual inspection. Similar distress symptoms and/or deflection measurements indicate similar conditions in the underlying pavement structure. This information is used to identify the boundaries between the different uniform sections, and the type of distress (indicating the mode of failure). The following sections describe how this is achieved in practice.

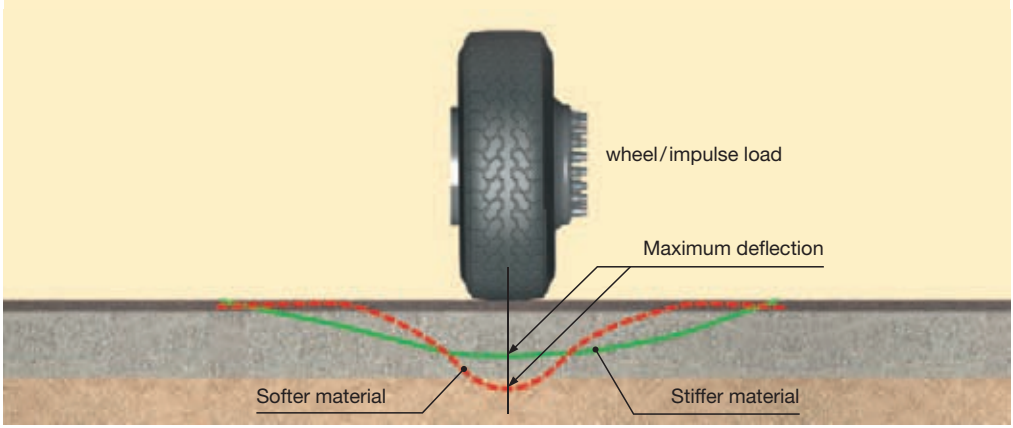
Deflection method

When a load is applied to the surface of a road, the pavement deflects. Deflections may be measured by applying a load on the pavement, either an impulse (falling weight) or a known wheel load that simulates a heavy vehicle. The magnitude of the deflection that occurs under a given load, as well as the shape of the "deflection bowl" produced by the load, provides a useful means of assessing the in-situ properties of the pavement.

Various methods for measuring pavement deflection have been developed, primarily for use as indicators of the structural condition and load carrying capacity of the pavement. Those most widely used are the Benkelman Beam and the Falling Weight Deflectometer (FWD).

Measuring pavement deflections using a FWD

The sketch below illustrates typical deflection bowls measured on two different pavements.



Typical deflection bowls

As primary input for their PMS, road authorities normally carry out deflection surveys at 3 to 5 year intervals on all major roads in their network. Where available, such information is invaluable for the initial definition of uniform sections using simple statistical techniques (cumulative-sum analysis) to identify where changes occur. The cumulative-sum values of maximum deflections are calculated using the formula:

$$S_i = (\delta_i - \delta_{mean}) + S_{i-1} \quad \text{(equation 2.1)}$$

where S_i = cumulative-sum value at location i;
δ_i = maximum deflection at location i; and
δ_{mean} = mean of maximum deflection for the entire section.
S_{i-1} = cumulative-sum value at location before location i

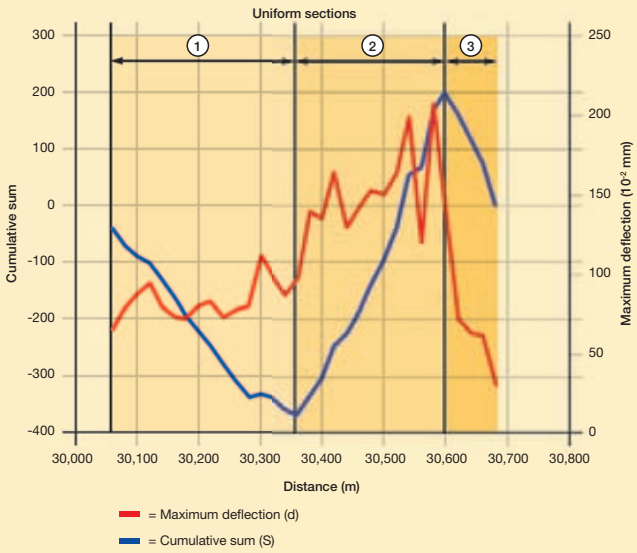
The cumulative-sum value is then plotted at each respective location, normally together with the maximum deflection value plotted on the same graph, as shown in the sketch opposite. A relatively constant slope for the cumulative-sum plot indicates sections of similar pavement response, or a uniform section.

Note: The cumulative-sum method is not restricted to maximum deflection. This method is often used with other deflection indices, such as the Surface Curvature Index (SCI).

Other methods

Where no deflection data is available, uniform sections must be identified by other means. As-built construction information (when available) is often used as an initial guideline, supplemented by a detailed visual assessment, as discussed below. However, when the required structural capacity of the pavement exceeds 10 million ESALs, it is always advisable to undertake a FWD survey at the

Distance (m)	Maximum deflection (d)	Cumulative sum (S)
30,060	64.80	-41.14
30,080	76.70	-70.38
30,100	86.60	-89.71
30,120	94.00	-101.65
30,140	79.10	-128.49
30,160	72.70	-161.73
30,180	71.30	-196.36
30,200	79.50	-222.80
30,220	82.40	-246.34
30,240	71.70	-280.58
30,260	76.80	-309.71
30,280	78.90	-336.75
30,300	110.40	-332.29
30,320	98.70	-339.53
30,340	86.70	-358.76
30,360	97.40	-367.30
30,380	139.60	-333.64
30,400	134.70	-304.88
30,420	164.00	-246.81
30,440	129.50	-223.25
30,460	142.50	-186.69
30,480	152.30	-140.33
30,500	150.10	-96.16
30,520	163.50	-38.60
30,540	198.90	54.36
30,560	119.60	68.02
30,580	208.60	170.69
30,600	132.80	197.55
30,620	72.10	163.71
30,640	63.20	120.98
30,660	61.10	76.14
30,680	29.80	0.00
Mean (D)	105.94	



Identification of uniform sections

outset. In addition to identifying uniform sections, the information derived from such a survey is invaluable for the statistical assessment of various in-situ pavement properties (see Section 2.5.5).

Note:

- Cumulative-sum methods can be used to identify uniform sections based on deflection measurements or other relevant information (e.g. subgrade CBR value) collected along the length of the road.

2.4.2 Visual inspection

Visual inspections are undertaken by walking the length of the road and recording all relevant features that can be observed or detected. Detailed notes are taken of all distress that is evident at the surface over the full width of pavement as well as other observations concerning drainage, geological changes and geometric features, (e.g. steep grades, sharp curves, cuttings and high embankments). The mode and type of distress that can be recognized during the inspection are normally classified into the categories shown below.

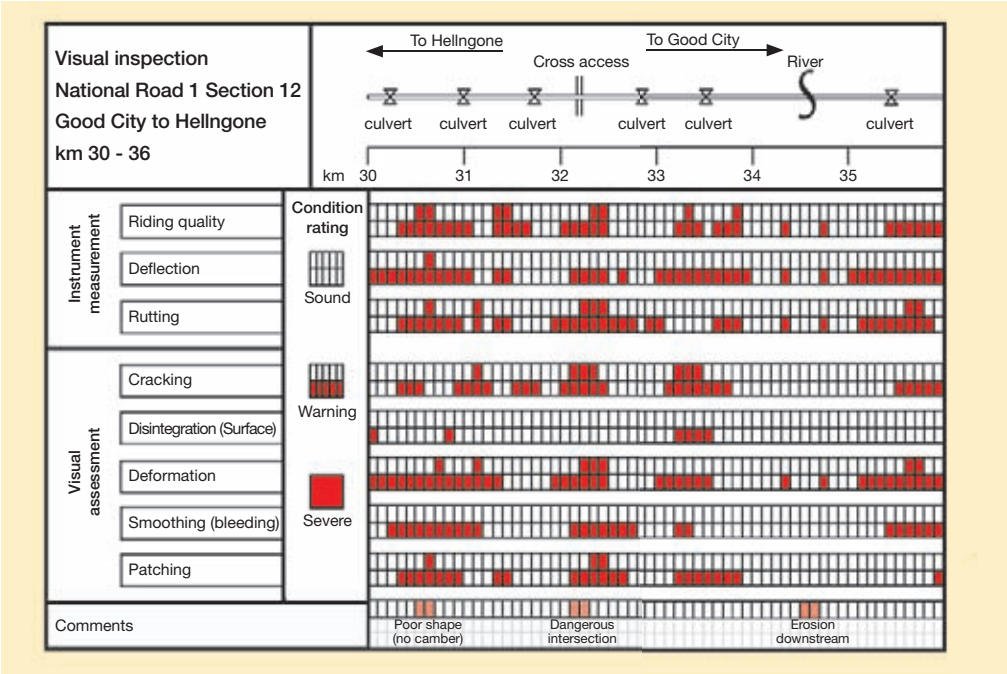
Note:

- Visual inspection data provide valuable clues regarding the cause of distress

Mode of distress	Type of distress	Description
Surface damage	Environmental damage Traffic damage	Ravelling, stone loss Thermal cracking Rutting (asphalt) Stripping, bleeding, polishing
Structural damage	Permanent deformation Cracking Advanced damage	Rutting in wheel paths Lateral shoving Longitudinal in wheel paths Crocodile Other (transverse, etc.) Potholes, patches, etc.
Functional condition	Drainage Riding quality	Erosion, washouts, etc. Edge break Undulations, corrugations, etc.

The different modes and types of pavement distress are recorded for each occurrence in terms of location, severity and frequency. Visual inspections provide valuable clues regarding the causes of pavement distress since failure patterns tend to be highlighted when all the data is summarised on one sheet. This feature is illustrated in the example shown opposite.

Whilst carrying out the visual inspection, digital photographs of the road surface are normally taken at regular intervals (± 250 m in both directions) and to record specific features (e.g. localised failure). In addition, video clips are powerful tools for recording traffic-related problem areas that should be addressed under the rehabilitation project (e.g. dangerous traffic movements).


Visual inspection
National Road 1 Section 12
Good City to Hellngone
km 30 - 36
To Hellngone
To Good City
Cross access
River
culvert
culvert
culvert
culvert
culvert
culvert
km 30
31
32
33
34
35
Instrument measurement
Riding quality
Deflection
Rutting
Visual assessment
Cracking
Disintegration (Surface)
Deformation
Smoothing (bleeding)
Patching
Condition rating
Sound
Warning
Severe
Comments
Poor shape (no camber)
Dangerous intersection
Erosion downstream

The primary difference between surface and structural distress is shown graphically in the following scetches.

Distress confined to the surfacing

Distress due to structural inadequacy

2.4.3 Reassessment of uniform sections

The uniform sections initially determined from deflection analyses should then be reassessed using information from the visual inspection, together with all other available information (e.g. construction records). This process allows a more accurate definition of the boundaries between individual uniform sections and facilitates the identification of sections with a similar pavement structure.

2.5 STEP 3: Detailed investigations

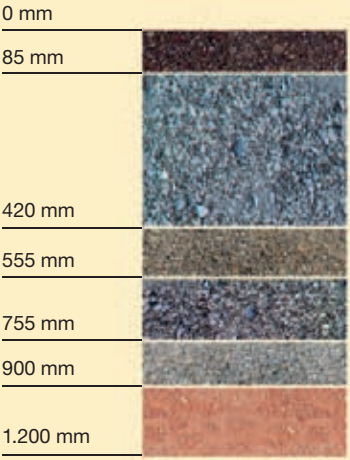
For each uniform section, a detailed investigation is required to evaluate the existing pavement structure (components and mode of distress) and to determine in-situ subgrade support. The tests and surveys normally employed in a detailed investigation are described below.

2.5.1 Excavating test pits

Test pits are without doubt the most important source of information concerning an existing pavement structure. In addition to gaining a visual appreciation of the different layers and materials in the pavement structure, test pits provide the opportunity to determine the in situ condition of the various materials and to take bulk samples from each layer for laboratory tests (for classifying the materials and for stabilisation mix designs).

The following important information can be accurately determined from test pits:

- individual pavement layer thickness;
- moisture content of the in situ material in each layer;
- in situ density of the material in each layer; and
- condition of the material in the various layers (e.g. the degree of cracking, cementation or carbonation of any cement-stabilised layer).



Test pit

Example of testpit excavation and profile (Depth from surface)

A minimum of two test pits are normally excavated for each uniform section, one where distress is evident, and the other where there is no distress. Test pits are generally located in the outer wheel path of the traffic lane and are sometimes positioned to straddle the shoulder and traffic lane. Test pits are usually 1.0 m in length (across the wheel path), 0.75 m wide (along the wheel path) and at least 1.0 m deep. Additional shallow slots (0.5 m wide) are often excavated across the full width of a traffic lane as a means of investigating the depth to which deformation extends and to determine the existance of any pavement widening, as well as the location of the boundary between the original and widened pavements.

Test pits must be carefully excavated so that each individual layer of different material type can be separated and removed independently. Each type of material encountered is carefully removed (usually by hand) and heaped separately next to the excavation for later sampling. As the digging progresses, density and other in situ tests can be carried out on each successive layer as it is exposed.

Once the excavation is complete, the pavement profile is carefully measured and recorded, as shown on the previous page. Bulk samples from each different layer are retained for laboratory testing before the test pit is backfilled.

Note:

- Representative samples must be used for laboratory tests

2.5.2 Laboratory testing

Bulk samples from test pits are tested in the laboratory to determine the quality of the material in each of the different layers, as well as the underlying subgrade. The testing programme must also include samples of aggregates that may be required for blending with any in situ recycled material. Representative samples of such blend materials are to be obtained from the same borrow pits and quarries that will be used as the respective sources for construction.

Laboratory tests normally carried out on these samples include: sieve analysis, Atterberg limits and California Bearing Ratio (CBR). The results are primarily used for material classification, i.e. to provide an indication of relevant parameters (such as elastic modulus) for use in analysing the existing pavement structure. The results are also used to indicate the material's suitability for stabilisation and which stabilising agent(s) would be appropriate.

2.5.3 Extracting core specimens

Extracting core specimens by rotary drilling through layers of bound material is relatively quick and less destructive than excavating test pits or inspection holes. Provided full recovery is achieved, core specimens can then be measured to accurately determine the thickness of the layer(s) of bound material (e.g. asphalt and cement stabilised material). Where required, core specimens recovered from asphalt layers may be tested for volumetric composition/engineering properties and the unconfined compressive strength determined by testing specimens recovered from cemented layers.

Note:

- The recoverable length of a core specimen is limited by the height of the core barrel used.
- When determining the thickness of bound layers from recovered cores, ensure that full recovery is achieved (i.e. that the specimen did not break during extraction and/or that drilling continued to the bottom of the bound layer).
- Larger diameter core barrels (150 mm diameter) are preferred, especially where the bound material includes aggregate larger than 19 mm.
- Unbound materials cannot be recovered and sampled by coring.

Extracting core specimens from layers of bound material

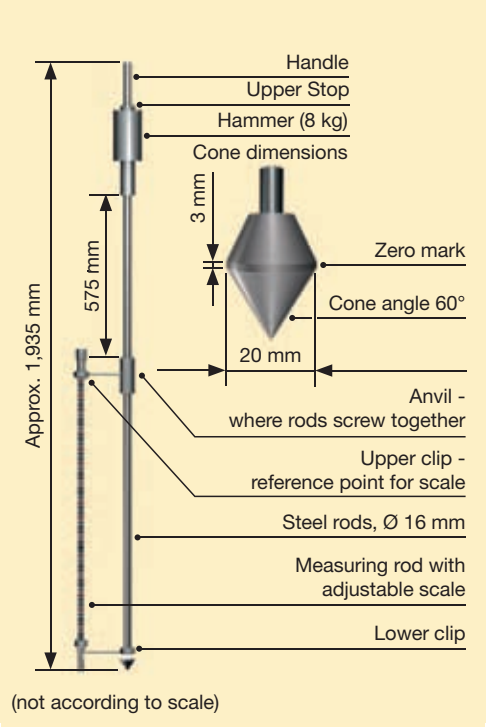
2.5.4 Dynamic cone penetrometer (DCP) probes

The DCP is a simple tool consisting of a steel rod with a hardened steel conical tip that is driven into the road pavement using a drop hammer of standard mass falling a constant distance. The penetration rate, measured in mm/blow, provides an indication of the in situ bearing strength of the material in the different pavement layers and a change in penetration rate indicates the boundary between layers. DCP probes are normally driven to a depth of 800 mm, or deeper into heavier pavement structures. The penetration rates can then be plotted and used to indicate the thickness of the various layers and the properties of the in situ material in each layer.

DCP penetration rates correlate well with the California Bearing Ratio (CBR) in relatively fine materials and only reasonably well with coarser granular materials (at the in situ density and moisture content). Correlations of penetration rate with the Unconfined Compressive Strength (UCS) of lightly cemented materials have also been developed. In addition, the DCP penetration rate provides a rough but useful guide for the elastic modulus of in situ pavement materials.

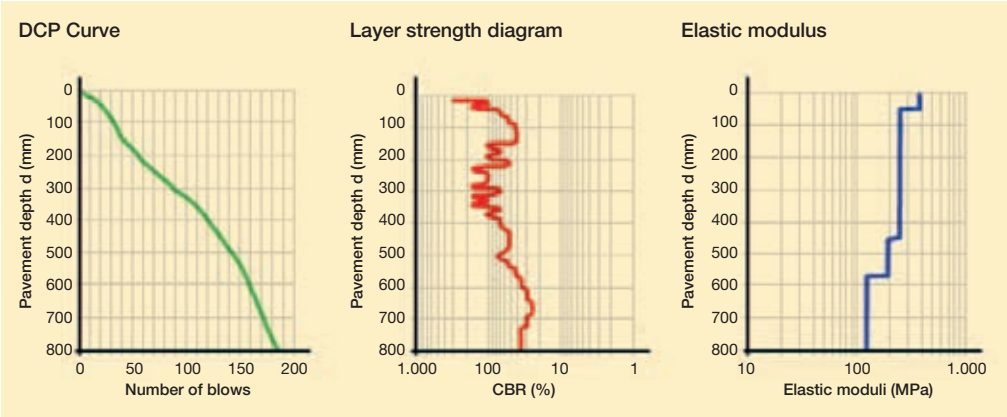
Since the coefficient of variation is often relatively high, numerous DCP probes are normally required to achieve statistical reliability. Measurements should therefore be analysed statistically to obtain the relevant percentile value (normally the 20th percentile is used for minor roads and the 5th percentile for major highways).

Individual DCP penetration measurements are normally taken once for every five blows. These measurements are then analysed using computer software to indicate in-situ CBR, UCS, Resilient Modulus and layer thickness, as shown in the example analysis opposite.



DCP dimensions

Difficulties are sometimes experienced when attempting to drive a DCP probe through coarse or bound material. A “refusal” condition is usually defined by a penetration of < 1 mm for each of two successive sets of 5 hammer drops (i.e. < 2 mm measured for 10 drops of the hammer). If this condition is encountered when attempting to drive the probe through the upper portion of a pavement, a 25 mm diameter hole should first be drilled through the layers of coarse and/or bound



material, thereby starting to probe at a lower depth in the pavement where the cone can penetrate.

Alternatively, if cores are extracted from bound layers in the upper pavement, DCP probes can be started from the bottom of the core holes. Where this procedure is followed, surplus water should be removed from the bottom of the core hole as soon as the core has been extracted. It must be appreciated that the water used to cool the core barrel whilst drilling will influence the penetration measured for the first 50 mm to 100 mm of the DCP probe.

2.5.5 Analysis of deflection measurements

As was introduced in Section 2.4.1, deflection measurements can be analysed to provide valuable in situ information about the materials in the pavement structure. In addition to assisting with the delineation of uniform sections, the relevant deflection measurements within each uniform section can be analysed statistically, at the appropriate level of confidence, and the deflection bowl back-analysed (e.g. only the 95th percentile deflection bowl for a specific uniform section is analysed.)

Actual layer thicknesses from field measurements and in situ layer moduli values indicated by DCP probes should be used as guidelines for each layer (especially the subgrade) when back analysing the deflection bowls. The results of these analyses provide estimates of the in situ stiffness values for the various pavement layers. As discussed in Section 2.6 below, this information is required for pavement modelling.

2.5.6 Rut depth measurements

Rut depths are generally measured manually using a straightedge positioned transversely across the wheel paths in each traffic lane. The maximum rut depth is recorded. Rut depths can also be measured using sophisticated mobile road surveillance equipment employing laser measuring techniques (e.g. ARAN – automatic road analyser).

The width of rut in the pavement surface (also referred to as the radius of rut) indicates the source of deformation within the pavement structure. Narrow ruts are generally caused by instability in layers of asphalt whilst wider ruts indicate permanent deformation in the underlying layers. The correlation of rut depth and deflection (measured at the exactly same point) also assists in determining whether the upper or lower layers in the pavement structure have deformed.

Measuring the rut depth in the wheel path

Note:

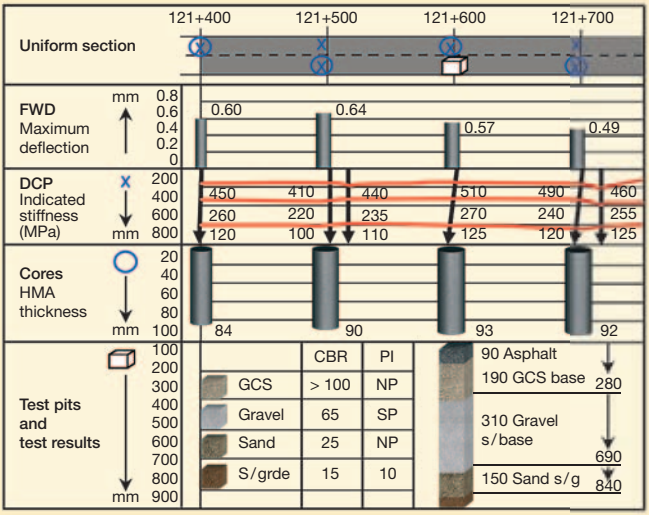
- The depth of rut measured is influenced by the length of straight edge used.

2.5.7 Synthesis of all available data

The detailed investigation phase culminates with the compilation of a summary sheet that includes all data specific to each uniform section, as shown in the example below.

The information shown in the summary sheet is typical for a comprehensive investigation. Since this summary sheet (or strip map) contains the primary input required for pavement design, all details concerning the different pavement layers and material characteristics must be included. A separate summary sheet (or strip map) is compiled for each uniform section.

When read in conjunction with the visual assessment summary (Section 2.4.2), the relevant mode of distress (failure) and problem areas within the existing pavement structure are easily identified. This allows the design engineer to focus on alternative rehabilitation measures to address the identified weakness and problem areas, as described in the next section.



2.6 STEP 4: Preliminary pavement rehabilitation design options

Once the investigations are complete and the summary sheets described above have been compiled, each uniform section can be considered in isolation and relevant rehabilitation design options formulated. As with all diagnostic procedures, the key to finding the best solution is to identify all possibilities at the outset. However, a sense of reasonableness must prevail in identifying alternatives since some will obviously be inappropriate (usually due to excessive cost and/or constructability implications) and can be disregarded.

When all alternatives have been identified, a subjective selection process is then followed to identify the three most appropriate options, thereby reducing the amount of analytical work required for designing the different pavements. These three alternative solutions must have similar structural capacities and, where a solution calls for additional layers and/or a thick asphalt overlay, the implications of raising road levels must be included in the analyses.

Pavement design for rehabilitation is different from designing new pavements, as discussed in the following sections.

2.6.1 Pavement design approach

Over the past 60 years, many pavement design methods have been developed, ranging from relatively simple empirical methods to the more complex modelling approaches that require sophisticated computer software.

The various pavement design methods can be summarised under two primary headings:

- **Empirical methods.** These include:
  - the CBR cover design method, based on the strength of the underlying subgrade;
  - catalogue design approach, based on typical pavement structures for specific applications;
  - the DCP design method that uses data from DCP surveys to indicate shortcomings in existing pavements;
  - the Structural Number method that assigns coefficients to various material types; and
  - the Pavement Number method that uses "intelligent" structural numbers.
- **Analytical methods.** These methods all include an analytical process that is followed by interpretation (empirical element) to translate the results of the analyses to structural capacity (known as transfer functions):
  - mechanistic analyses. These methods are based on stress and strain analysis using multi-layer linear-elastic, elasto-plastic or finite element models; and
  - methods using deflection measurements (deflection bowl analyses).

As a general rule, more heavily trafficked pavements (> 10 million EASLs) should always be designed using an analytical approach. An empirical method may suffice for lighter pavements but, where there is any doubt that a design may not be appropriate for the anticipated traffic loads, it should be checked using an analytical method.

2.6.2 Catalogue design methods

Catalogue design methods are prescriptive in the types and quality of materials required for a suitable pavement structure. The catalogue provides a list of pavement types appropriate for different support conditions and structural capacities. Although this design approach is usually developed using analytical procedures, it is both restrictive (as it cannot include all options) and not easily transferable (as it is often developed for local materials and climatic conditions). The support conditions used in catalogue designs also need to be analysed on the basis for which the design options were developed. Catalogue designs are therefore of little use for pavement rehabilitation.

2.6.3 Structural number method

Based on experience, structural coefficients have been developed for certain pavement materials for use in structural design. The AASHTO 1993 pavement design method uses a structural number that is determined by summating the product of these structural coefficients and the respective layer thickness. If the total exceeds a certain minimum number for the specific subgrade condition and structural capacity requirement, the pavement structure is considered to be adequate.

The Structural Number (SN) approach is simple as it uses known materials with a track record of performance in given climatic conditions. Caution should be exercised when using this method where climatic conditions are severe or where local materials are significantly different. Furthermore, since there is no inherent control system for maintaining pavement balance in terms of relative stiffness of overlying layers, this design method is not recommended for pavements with a structural capacity requirement in excess of 10 million ESALs.

Pavement		Structural coefficient (per in)	Layer thickness		SN layer
Layers	Material		mm	inches	
	Asphalt	0.4	100	4	1.6
	GCS CBR > 80	0.14	200	8	1.12
	Natural gravel CBR > 45	0.12	300	12	1.44
	Subgrade > 15			Σ SN =	**4.16**

Structural number calculation example

2.6.4 Pavement number method

This method is similar to the structural number method, but uses the "effective long term stiffness" (ELTS) values for different pavement materials in place of structural coefficients.
This design method (described in TG2 (2009)) is based on the analyses of long term pavement performance (LTPP) exercises combined with laboratory research and heavy vehicle simulator (HVS) trials.

The pavement number method summates the product of the ELTS value and the respective layer thickness. The number obtained is then used in a "frontier curve" (derived from the LTPP exercise) to indicate the structural capacity of the pavement structure.

As with structural numbers, the pavement number (PN) method is simple to use. The main difference between the two methods is the procedure followed in determining appropriate ELTS values as opposed to selecting a structural coefficient. This procedure includes a comprehensive classification system for the materials in the various layers and takes into account the climate, the location of the layer within the pavement structure and the amount of cover over the subgrade. It utilises the modular ratio rule to ensure pavement balance.

Different material classes and their respective modular ratio / maximum allowable stiffness value are summarised in the table below (see also TG2 (2009)).

Material type	Class	Principal strength characteristic	Modular ratio	Maximum stiffness (MPa)*
HMA	AC	Marshall	5	2500
Bitumen stabilised	BSM Class 1	ITS_{DRY} > 225 kPa ITS_{WET} > 100 kPa	3	600
Cement stabilised	CTB Class C3	1.5 < UCS < 3 MPa	4	550
Crushed stone	G1	CBR > 100	2.0	700
	G2		1.9	500
	G3		1.8	400
Natural Gravels	G4	CBR > 80	1.8	375
	G5	CBR > 45	1.8	320
	G6	CBR > 25	1.8	180
	G7	CBR > 15	1.7	140
Soils	G8	CBR > 10	1.6	100
	G9	CBR > 7	1.4	90
	G10	CBR > 3	1.2	70

* These maximum stiffness values are relevant only to the empirical Pavement Number model. They are not intended as input for mechanistic models (see Section 4.3.12)

Pavement structure			Layer thickness	Modular ratio	Maximum stiffness	ELTS	PN layer
Layers	Material	Class	mm		MPa	MPa	
	Asphalt	AC	100	5	2,500	2,340	23.4
	GCS CBR > 80	G1	200	2	700	468	9.36
	Natural gravel CBR > 45	G5	300	1.8	320	234	7.02
	Subgrade > 15	G7	Subgrade cover 600 mm		130	Σ SN =	**39.78**

Pavement number calculation example

Pavement Numbers may be used with confidence to design pavements with a structural capacity up to 30 million ESALs, the limit of the frontier curve. This is an artificial limit since it is dictated by the maximum traffic volume carried on pavements that were included in the LTPP data set. As this data set is expanded and updated, the frontier curve will be extended to include pavements with structural capacities in excess of 30 million ESALs.

2.6.5 Mechanistic design methods

The mechanistic design process uses structural mechanics and material models to analyse stresses, strains and deflections that develop within a pavement structure when a load is applied. The relevant stress, strain or deflection value is then related to structural capacity (number of load repetitions to failure) by means of a "transfer function", an empirical relationship derived from research and/or pavement performance data.

Multi-layer linear elastic theory is generally used for analysing the pavement model, mainly because it is relatively simple to use and easy to set up. (Other analytical tools that can be used for modelling include finite element analyses incorporating non-linear-elastic, elasto-plastic and stress-dependent constitutive models.) The pavement is modelled by defining the layer thickness together with the relevant properties of the material in each layer (in terms of elastic modulus and Poisson's ratio). The loading condition is defined (axle mass, tyre/wheel configuration and tyre pressure) and the response of each layer is then calculated.

The input parameters for the materials in each layer are obtained during the detailed investigation stage. Material classification from laboratory tests provides an indication of stiffness and relevant Poisson Ratio values for the specific materials. In addition, analyses of DCP probes and deflection bowls are invaluable for the in situ properties of pavement materials.

The mechanistic design method has distinct advantages for rehabilitation design as it enables non-standard materials in existing pavement layers to be modelled effectively. In addition, the method accommodates all rehabilitation options, especially thick stabilised layers that are a feature of recycled pavements.

Applied load:		80 kN per axle carried on dual tyres, tyre pressure 750 KPa			
Pavement		Layer thickness	Poisson's ratio	Resilient modulus range (moisture dependent)	Failure criterion (evaluation parameter)
Layers	Material	mm		MPa	
	Asphalt	100	0.4	2,500 – 5,000	Tensile strain
	GCS CBR > 80	200	0.35	300 – 800	Deviator stress ratio
	Natural gravel CBR > 45	300	0.35	150 – 400	Deviator stress ratio
	Subgrade > 15	Infinite	0.35	70 – 140	Vertical strain

Input parameters for a multi-layer linear elastic model

2.6.6 Deflection based methods

Deflection based rehabilitation design methods are primarily used to determine the thickness of asphalt overlays. Deflection measurements are analysed to determine the effective stiffness of the existing pavement structure and this is used as the primary input in a mechanistic exercise to indicate the thickness of asphalt required to accommodate applied loading conditions, given the effects of reflective cracking from existing bound layers. An iterative approach is adopted to determine the thickness of asphalt overlay that prevents stress levels in the new material from exceeding the elastic limit, thereby achieving the required structural capacity.

Alternatively, deflections can be used to gain an indication of the strain in the upper part of the pavement which will indicate the thickness of asphalt overlay required.

2.6.7 Summary of pavement design approaches

Investigation and design phases should be integrated with the primary objective being to understand the behaviour of the existing pavement. The second objective is to determine the most cost-effective pavement design that satisfies expectations regarding design life and functional properties, that minimises maintenance interventions and one that produces a pavement that can be rehabilitated at the end of the design life at minimal cost.

Mechanistic design methods are favoured as a means of checking the adequacy of the existing pavement, identifying weaknesses, and for designing pavements that satisfy the rehabilitation requirements. Empirical design methods may be used for lower levels of design traffic or as a first attempt at rehabilitation design.

2.7 STEP 5: Laboratory mix design

Laboratory mix designs play the role of verifying the suitability of selected materials for treatment with additives. Such additives can include stabilising agents, chemicals, aggregates and natural materials. Mix designs are a fundamental part of the pavement investigation and design procedure and serve the purpose of establishing the most effective method of treating the materials to improve their engineering properties. (Chapter 4, "Stabilising Agents" includes detailed information on stabilising agents.)

Representative samples taken from the material to be treated are subjected to mix design testing. These samples should be prepared to simulate as closely as possible the material that will be produced on site during the actual treatment process. Where bound material from an existing pavement (e.g. asphalt) is to be recycled in situ, samples should be obtained using a small milling machine to simulate the grading of the material that will be produced by a recycler.

Mix design procedures that can be used for stabilising with cement, bitumen emulsion and foamed bitumen are described in Chapter 4 and included in Appendix 1. These procedures essentially incorporate five basic steps:

Step 1: Initial selection of stabilising agents, taking into account:

- suitability of a specific stabilising agent in relation to the type and quality of material to be treated. The initial selection of the most appropriate stabilising agent is based on the results of laboratory tests carried out on the untreated material;
- required engineering properties of the stabilised material;
- availability, in terms of being able to procure sufficient daily volume requirements, as well as consistency of the quality of stabilising agent that can be supplied; and
- the relative cost of different stabilising agents.

Based on the above, a decision is made to proceed with the full mix design using the most appropriate stabilising agent. Alternatively, several mix design options can be investigated simultaneously to determine which is the most appropriate.

Step 2: An optimisation exercise is carried out by preparing several identical portions of the sample and mixing each with different amount of stabilising agent. Simultaneously, sufficient water is added to bring the mixture to its optimum moisture content (for compaction). Typically at least four mixes are prepared, each one mixed with a different stabiliser content.

Step 3: Specimens are manufactured using standard compaction effort.

Step 4: The specimens are cured, ideally to simulate field conditions.

Step 5: After curing, the specimens are subjected to various tests to assess their engineering properties, as well as their susceptibility to moisture.

In order to determine the optimum stabiliser content, the results of these tests are plotted against the stabiliser content for each of the mixes. The stabiliser content that best meets the desired properties is regarded as the optimum stabiliser content (also termed the optimum application rate of stabilising agent).

Note:

- Mix design results must be integrated with pavement designs

2.8 STEP 6: Finalise pavement design options

The initial pavement design options described above in Section 2.6 were, of necessity, based on assumed values for the engineering properties of the material in the new stabilised layer(s). The actual properties are determined by carrying out the mix designs described above in Section 2.7. If the actual properties are significantly different from those assumed, the initial pavement design and material utilisation needs to be reviewed. Where the assumed values were not achieved in the mix designs, the following options may be considered:

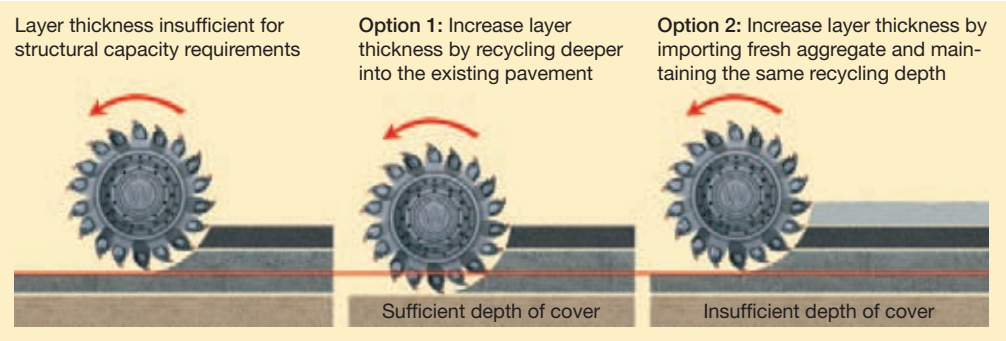
- Increase the layer thickness. On recycling projects, this increase will result in a deeper cut that may incorporate different material (usually of poorer quality) from the underlying pavement. Where the change in material is significant, the mix design must be repeated to determine the correct properties.

  Alternatively, the recycled material can be blended with new imported aggregate to effectively increase the layer thickness. Where it is obvious that recycling deeper will not address the shortfall in material properties, blending with an imported good-quality aggregate (e.g. crushed stone) may be considered. However, such a change invariably necessitates repeating the mix design.

- Use a different stabilising agent or blend of alternative stabilising agents. This option requires a new mix design.

- Import and stabilise an additional layer on top of the existing pavement structure. This will create a deeper pavement, thereby reducing the levels of stress and strain in the existing portion of the pavement.

The pavement design is then finalised by inputting the mix design values for stabilised material into the pavement design model and refining the layer thickness of stabilised material to meet the structural capacity requirements.



2.9 STEP 7: Economic analyses

Economic analyses are considered an effective tool for selecting the most appropriate rehabilitation option. Alternative pavement rehabilitation designs cannot be compared on the basis of construction costs alone. In addition to the cost of maintenance that will be required during the service life of the road (dependent on the pavement type, structure, materials, etc), the “salvage value” (cost of rehabilitation at the end of the service life) also needs to be included in the economic analysis. Therefore, all costs incurred during the service life of the pavement (the “whole-of-life costs) should be integrated into relevant calculations.

The economic analysis method normally used to compare alternative pavement options is the Present Worth of Cost (PWoC) method. This method is based on estimates of all costs that will be incurred during the life of the pavement (the whole-of-life costs that include initial construction costs, plus the cost of all routine maintenance, plus rehabilitation at the end of the service life), discounted to account for the time value of money. In such an exercise, it is of paramount importance to adopt the correct discount rate so that the present worth of future maintenance and rehabilitation costs are realistic.

It is often difficult to estimate comparative construction and future maintenance costs. Local knowledge of materials and the environment, as well as data on pavement performance functions (normally obtained from a Pavement Management System) will assist in determining realistic future maintenance measures and their timing.

The process of evaluation is explained by means of an example in section 5.2.
Appendix 4 contains a comprehensive background to economic evaluations and includes some of the techniques that are applicable to the economic analyses of different pavement structures (e.g. PWoC, Benefit / Cost Ratios, Internal Rate of Return and Net Present Value).

3 Cold recycling

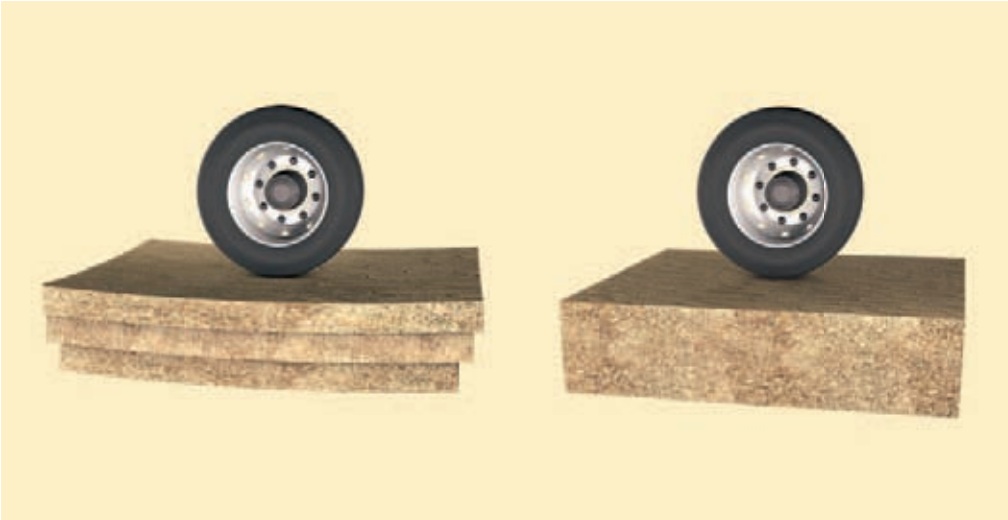
3.1 General

Cold recycling is the term used for recovering and re-using material from an existing pavement, without the addition of heat. Unlike hot recycling that is confined to heating and reusing asphalt material, the field of cold recycling enjoys a wide range of applications, from relatively thin layers comprising mainly asphalt material (also known as “cold in-place recycling”) to thick layers that include two or more different pavement materials (also termed “full depth reclamation” (FDR)).

This chapter describes the various applications of cold recycling and introduces the equipment required to carry out the work, specifically the range of Wirtgen machines. Also covered are the benefits that accrue from adopting cold recycling and, in addition, the major factors that influence the suitability of using such a process on a specific project.

In situ recyclers e.g. WR 2500 S can work to depths in excess of 300 mm, break down and utilise 100% of the in situ material, maintain a preset horizon within the existing pavement and produce a quality of mixed material that is comparable with off-site mixing. Conflict with public traffic is limited to supply vehicles accessing the work zone. Weak areas in the underlying pavement structure are not disturbed and exposure to inclement weather is significantly reduced by processing the material in a single pass and compacting the treated product immediately behind the recycler. In addition, production rates as high as 10,000 m² per shift are possible and the risk of failures occurring due to poor quality work is drastically reduced.

With these capabilities, recyclers have introduced pavement engineers to a whole new range of possibilities, the most important being the ability to construct thick monolithic layers of stabilised material. From a structural perspective, a single 300 mm thick layer of stabilised material has a far higher load-carrying capability than two separate 150 mm thick layers constructed one on top of the other. This concept is well known in the building industry where laminated timber beams are used as structural members.



A thick layer of bound material is similar to a laminated timber beam

3.2 The cold recycling process

Cold recycling can be achieved either “in-plant” by hauling material recovered from an existing road to a central depot where it is fed through a mixing unit, or “in-place” using a recycling machine. Plant processing is generally the more expensive option in terms of cost per cubic metre of material processed, primarily due to haulage costs that are absent from in-place recycling. However, in-plant processing becomes attractive when:

- additional pavement layers are required. In-plant processing is usually preferred where previously-stockpiled material recovered from existing pavements can be recycled and used to construct a new pavement layer. Material treated with a bitumen stabilising agent can also be placed in stockpile for later use. This is becoming increasingly popular for dealing with stockpiles of reclaimed asphalt pavement (RAP) material;
- different materials are to be blended in accurate proportions; and/or
- the material in the existing pavement is highly variable and requires a process of selection; and/or
- the material in the existing pavement is so hard that it cannot be properly pulverised in situ. Such materials are removed from the road and pre-treated before being reused as a pavement material (e.g. crushing lumps of aged asphalt or concrete).

Where it can be adopted, in-place recycling will always be the preferred recycling method due solely to the economic advantages that are offered. In light of pavement deterioration worldwide, rehabilitation of existing pavements far exceeds the demand for new roads and in-place recycling has been universally accepted as the preferred method for addressing the enormous backlog in pavement rehabilitation. In-place recycling therefore justifies the focus of this manual with in-plant treatment receiving less attention, but always remaining an option. Each process is discussed separately in the following sections.

Note:

- Cold recycling existing pavements is the universally preferred method of structural rehabilitation

3.2.1 In-plant recycling

In-plant recycling allows materials from an existing pavement to be selected and pre-treated, thereby increasing the level of confidence that can be achieved in the final product. The main benefits to accrue from in-plant compared to in-place treatment are:

- Control of input materials. Whereas in-place recycling allows no control over the type of material that is recovered from an existing pavement, a required end-product can be obtained by blending different aggregates in accurate proportions when mixing in-plant. Input materials can be selected, pre-treated (e.g. by crushing and screening), stockpiled and tested prior to mixing. The proportioning of various input materials can then by changed as and when required to obtain the required mix.
- Quality of mixing. An adjustments can be made to the mixer to vary the time the material is retained within the mixing chamber, thereby improving the quality of the mix.

Typical in-plant operation with stockpiles of input material

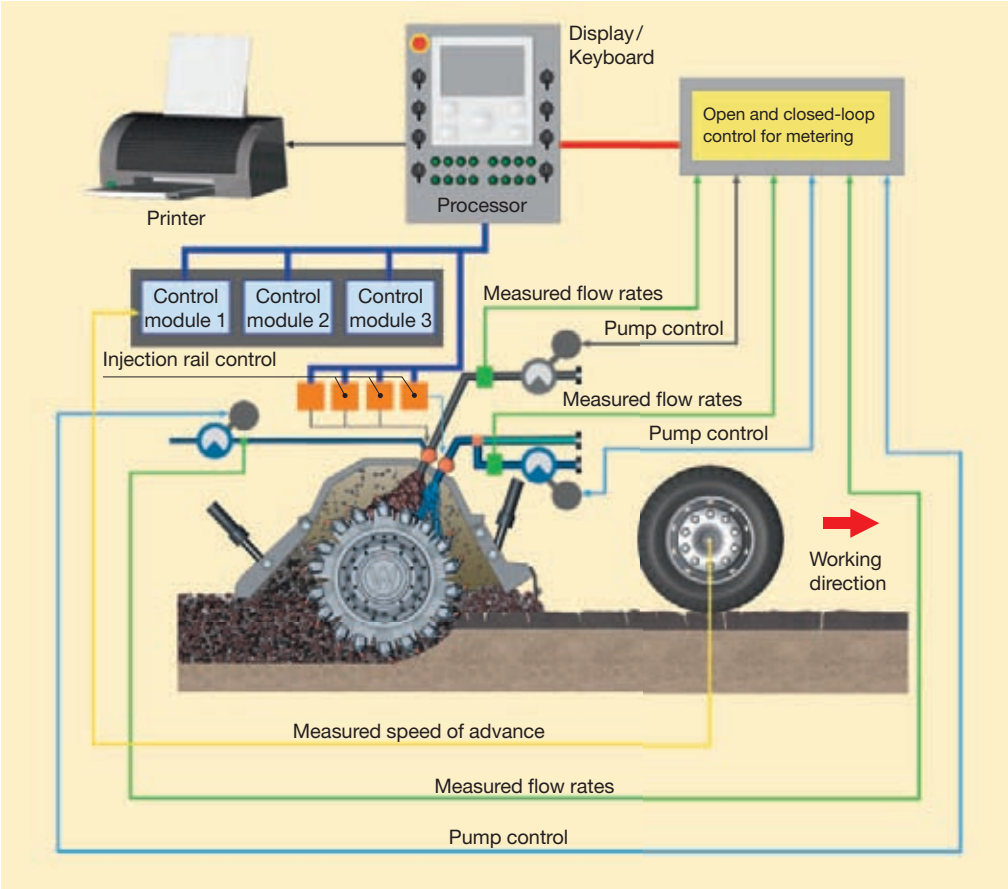
- Stockpiling capabilities. Particularly with bitumen stabilised materials, the mixed product can be placed in stockpile and used when required, thereby removing the inter-dependency of the mixing and placing processes. Cognisance must, however, be taken of the strict time restrictions applicable to stockpiling mixes that include cement.

Placing the mixed material on the road as a new pavement layer can then be undertaken by paver, grader or, where required, by hand using labour-intensive methods.

3.2.2 In-place recycling

Recycling machines have evolved over the years from modified milling machines and basic soil stabilisers to the specialised recyclers of today. Since they are specifically designed to recycle thick pavement layers in a single pass, modern recyclers tend to be large powerful machines, mounted either on tracks or high flotation pneumatic tyres. During the past 20 years, Wirtgen has invested heavily in developing this technology and their current range of high-performance machines pays testament to the advances that have been achieved. A brief overview of the different types of recyclers manufactured by Wirtgen and their typical applications is included in Section 3.3.

The heart of all recycling machines is the cutting drum that is fitted with a large number of point-attack tools. Different tools are available to suit different cutting conditions (e.g. hardness, abrasiveness, etc.). The drum normally rotates upwards and, as the machine advances, the in situ pavement material is pulverised by the tools and lifted into the mixing chamber that encloses the drum (the drum housing). Recyclers are equipped with at least one pumping system for adding fluid (e.g. water) to the recovered material. The rate of delivery of the fluid is metered accurately by means of a micro-processor that regulates the flow in accordance with the volume of material in the mixing chamber. The fluid is injected into the mixing chamber through a series of nozzles spaced equidistant on a spraybar that spans the full width of the chamber as illustrated in the diagram on the next page.



Micro-processor control for the injection systems on the Wirtgen WR 2500 S

Recycling is undertaken by coupling bulk supply tankers to the recycler. The recycler pushes or pulls the tankers supplying the additives required in the mix (e.g. bitumen emulsion).

The combination of tankers coupled to the recycler is configured in accordance with the particular recycling application and the type of stabilising agent that is applied.

Sequency of machine in use

The simplest combination consists of a recycler coupled to a single tanker containing water. As the machine advances, the in situ pavement material is recovered and mixed with water drawn from the tanker. The micro-processor ensures that the required amount of water is injected into the mixing chamber through the spraybar that is mounted on the leading face of the drum housing (illustrated in the sketch on the next page).
The rotating drum mixes the water with the recovered material to achieve a uniform consistency. The rate of water addition is controlled to achieve the moisture content that will allow a high level of density to be achieved when the material is compacted.

As the recycler advances, the mixed material falls back into the void created by the cutting drum and is struck off by a sturdy door fitted to the rear of the drum housing. As shown in the above sketch, a roller follows behind the recycler to compact the material before a grader is used to cut final levels. Not shown in the sketch is the final compaction and finishing process that uses both vibratory and pneumatic-tyred rollers working together with a water tanker.

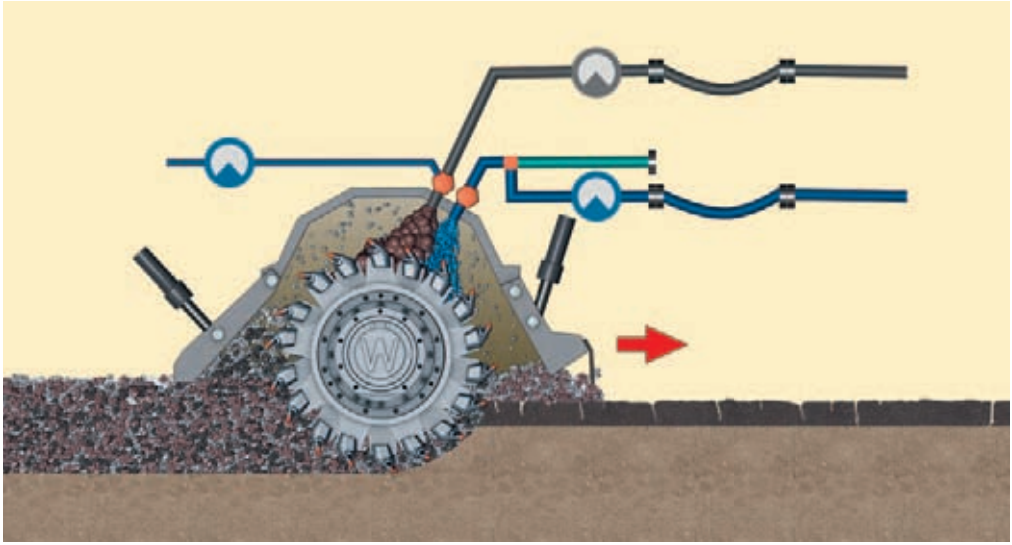
Powdered stabilising agents (e.g. cement or hydrated lime) are normally spread on the surface of the existing road ahead of the recycling operation. As the recycler advances, the powder is lifted and mixed together with the recovered material and water, all in a single operation.

Note:

- The micro-processor is of paramount importance since it controls the rate of application of water and bitumen

Alternatively, the powder can be mixed with water to form a slurry suspension that is then injected into the mixing chamber. Where this method of application is adopted, a special mixing unit is coupled to the recycled. This “slurry mixing unit” manufactures the slurry by combining the precise amounts of both cement and water required to treat the volume of material being recycled. The slurry is then pumped across to the recycler by means of a flexible hose and injected through the spraybar.

Where a bitumen stabilising agent is added (either bitumen emulsion or foamed bitumen), a second application system is mounted on the recycler complete with separate spraybar that is attached on top of the drum housing, as illustrated in the sketch below.



Dual application of water and foamed bitumen

This application requires a bulk tanker containing the bitumen to be coupled to the recycler. In addition, active filler (cement or hydrated lime) is normally added with a bitumen stabilising agent (as explained in Chapter 4). Where the active filler is spread as a powder on the road surface ahead of the recycling process, the bitumen tanker is coupled directly to the recycler and a water tanker pushed in front as the lead vehicle. However, where the active filler is added as slurry, the slurry mixing unit is always placed immediately ahead of the recycler and the bitumen tanker becomes the lead vehicle, as illustrated in the sketch below.

The in-place recycling process described and illustrated above focused on tyre-mounted recyclers. Although the process is similar for track-mounted machines, there are some fundamental differences and these are described in the following section.

Recycler coupled to a slurry mixer and bitumen tanker

3.3 In-place recycling machines

This section includes a general overview of the three different types of in-place recycling machines manufactured by Wirtgen and their respective capabilities. Individual machines are not described, nor are any specifications included. These can all be found in the companion publication “Wirtgen Cold Recycling Application” that is concerned with the application of recyclers and the various construction processes.

The different types of in-place recycling machines are:

- **Tyre-mounted recyclers.**
  These machines are designed primarily for recycling. Wirtgen has several models of different mass and capability in their product range; a mid-size machine is shown in the picture.

As shown in the illustration on the facing page, the point attack tools are positioned in a chevron pattern and mounted on stanchions to promote mixing.

Such a tool configuration promotes mixing in the vertical plane but not the horizontal. This means that the recovered material is not thrown sideways (laterally) or lengthways (longitudinally) from its original location within the pavement (the maximum movement measured from tests is 200 mm). This means materials in the recycled horizon will be returned to the pavement, after recycling, in approximately the same location, reflecting any original differences.

The Wirtgen WR 2400 tyre-mounted recycler

The cutting drum on a tyre-mounted recycler

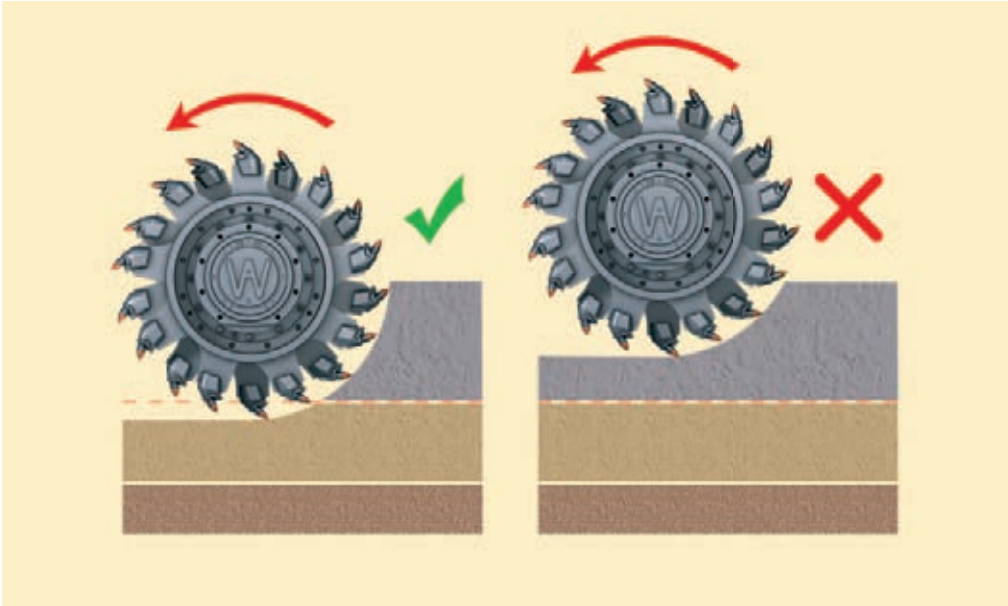
The rear wheels of tyre-mounted recyclers run on the treated material

Layers of dense unbound material are easily broken apart, returning to their original uncompacted state. The degree to which layers of bound material (e.g. asphalt) are pulverised is influenced mainly by the advance speed of the recycler, but also by the rotation speed of the drum. The faster the advance speed and the slower the speed of rotation, the coarser the product. Large lumps of material that are not pulverised in the process tend to be thrown to the bottom of the layer.

The rear wheels of tyred mounted recyclers are located inside the extremities of the cutting drum and therefore run on the outer edges of the recycled material. As shown in the picture, the material in the wheel paths is compacted whilst the material between the wheel paths remains in a loose ("fluffed") state.

Tyre mounted recyclers are also used extensively to pulverise thick layers of bound material in existing pavements (larger machines are used for thicker layers). Increasing use is also being made of these machines to pre-treat thick layers (up to 500 mm thick) in the lower portion of the pavement. Such pre-treatment includes breaking down clods of material or soft rock and the addition of water to achieve a consistent moisture content that facilitates compaction.

The cutting drum of tyred mounted recyclers must first penetrate through to the underside of the bound material (e.g. asphalt) before the machine can advance and pulverise the asphalt, as shown in the following sketches.



Tyre-mounted recyclers must attack bound material (asphalt) from underneath the layer

Track mounted recyclers.
The cutting drum of these recyclers is the same as that used for milling asphalt with the point attack tools mounted in a helical pattern to windrow the material to the centre of the drum.

Instead of being lifted onto a belt and removed (as happens when milling), the recovered material exits through a door in the rear of the milling chamber and passes between the rear tracks to be spread across the width of cut by a variable screed fitted with individual left and right feed augers. The required surface levels and shape can often be achieved by using such a screed, thereby eliminating the need for a grader to cut final levels.

As with the tyre-mounted recyclers, micro-processor controlled pumping systems are incorporated with spraybars attached to the outside of the drum housing to inject fluid additives into the material in the milling chamber.

Unlike tyre-mounted recyclers, the windrowing action of the tool pattern promotes blending of the material recovered by each half-width of the drum.

The Wirtgen 2200 CR track-mounted recycler

The standard milling drum on the Wirtgen 2200 CR

Milling machines are designed and built to provide stability when cutting into layers of hard asphalt. Both the drum housing and the milling drum are attached to the frame of the machine and the depth of cut is varied by lifting or lowering the whole machine. This means that:

- the drum does not have to penetrate to the underside of thick bound layers to be able to recycle the material. It is therefore possible to recycle only the upper portion of thick bound layers using these machines; and
- the volume of the milling chamber is constant, regardless of the depth of cut. The amount of material that can be mixed is therefore limited and this restricts the depth of cut that can be recycled to a maximum of 250 mm (or less when applying a cohesive stabilising agent).

These machines are ideal for recycling 100% asphalt material. A flagship of the Wirtgen Cold Recycler product range is the 2200 CR equipped with a 3.8 m wide drum to allow the full width of a traffic lane to be recycled in a single pass. In addition, the rotation of the drum is changed from up cutting (anti clockwise) to down cutting (clockwise) to promote fragmentation when recycling thin asphalt layers only.

Paving screed fitted to the rear of the Wirtgen 2200 CR

- **Machines fitted with an on-board twin shaft pugmill mixer.**
  Wirtgen manufactures one model of recycler that falls into this category: the WR 4200 shown in the picture on the facing page.

  This track-mounted machine is capable of recycling to a maximum depth of 200 mm and includes:

  - an adjustable working width ranging between 2.8 m and 4.2 m. This allows the entire width of one traffic lane to be recycled in a single pass. It also permits the location of longitudinal joints (between adjacent cuts) to be selected to fall outside the trafficked wheel paths;
  - the width of recycling can be varied whilst working, making it easy to recycle tapering sections often associated with highway interchanges and toll plazas;
  - the material in the recycling horizon is milled and lifted into a twin-shaft pugmill mixer mounted on the machine. Two pumping systems allow for the accurate addition of stabilising agents and water and the mixing quality achieved is similar to that of conventional stationary mixing plants; and
  - lifting the recovered material off the road and mixing on board the recycler achieves a uniformity of mix across the full cut width. In other words, the material recovered from the width of cut is fully cross-blended.

The Wirtgen WR 4200 with twin shaft pugmill mixer

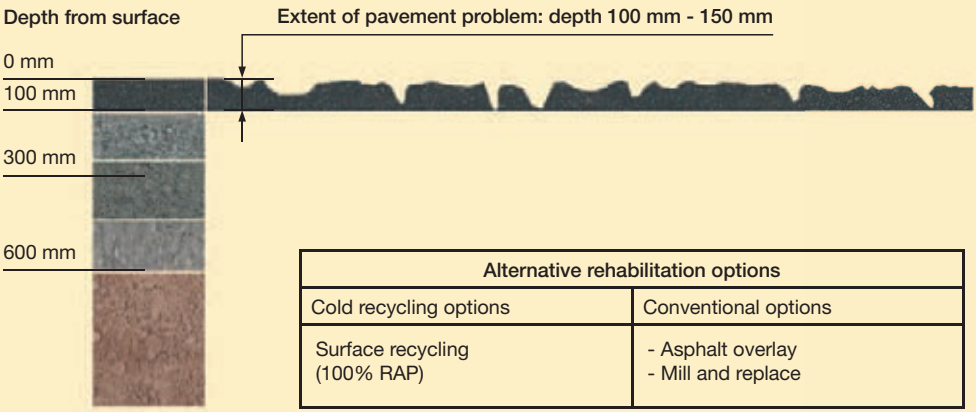
The treated material is discharged from the pugmill mixer onto the road as a windrow and spread by auger. A variable width paving screed is attached to the rear of the machine for placing the recycled material true to the required profile. This screed is equipped with both tampers and vibration for pre-compaction.

3.4 Cold recycling applications

The cold recycling process has numerous possible applications for maintaining and rehabilitating road pavements. However, each application will be project specific with three primary factors dictating the method of recycling that is appropriate:

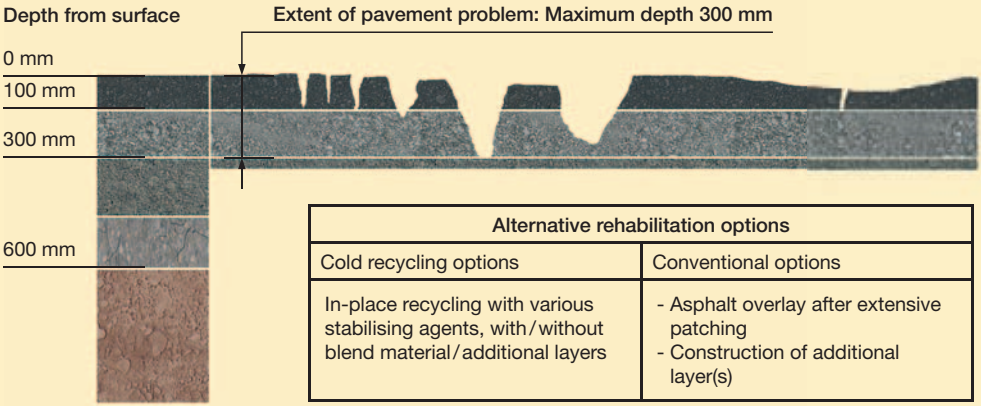
- The type of pavement distress that needs to be addressed;
- The quality of material in the recycling horizon; and
- The outcome required (i.e. service life expectations).

The following sketches show three different pavement distress conditions with some of the different options that can be applied for addressing the relevant distress.



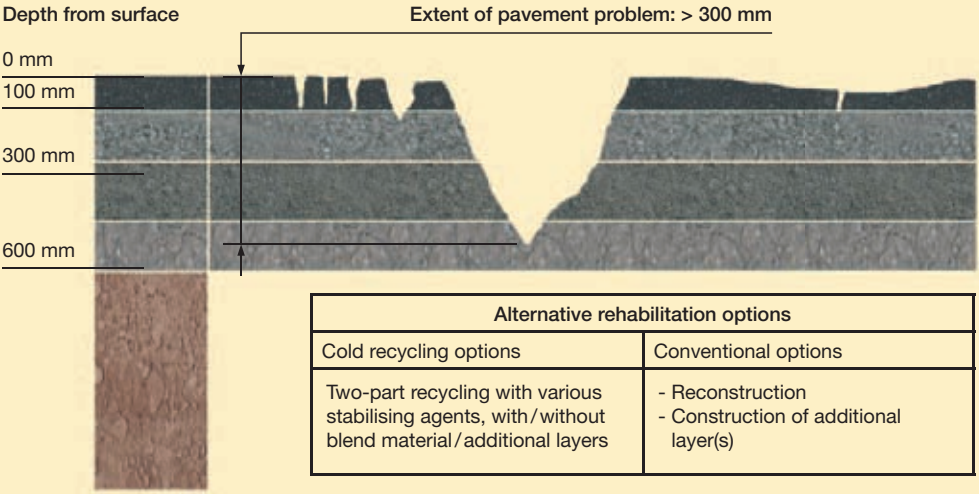
Alternative rehabilitation options	
Cold recycling options	Conventional options
Surface recycling (100% RAP)	- Asphalt overlay - Mill and replace

Rehabilitation options for upper pavement/surfacing distress



Alternative rehabilitation options	
Cold recycling options	Conventional options
In-place recycling with various stabilising agents, with/without blend material/additional layers	- Asphalt overlay after extensive patching - Construction of additional layer(s)

Rehabilitation options for structural distress in the upper pavement layers



Alternative rehabilitation options	
Cold recycling options	Conventional options
Two-part recycling with various stabilising agents, with / without blend material / additional layers	- Reconstruction - Construction of additional layer(s)

Rehabilitation options for deep-seated structural distress

The materials encountered within the recycling horizon can be classified in to two primary material types:

- 100% reclaimed asphalt pavement (RAP) material where the depth of recycling encounters only asphalt; and
- Blend of RAP / Granular material where the recycling depth includes layers of different materials used to construct the upper portion of the pavement. These include RAP, bituminous surfacing materials, crushed stone and natural gravels, as well as materials that were previously stabilised (mainly crushed stone and natural gravel).

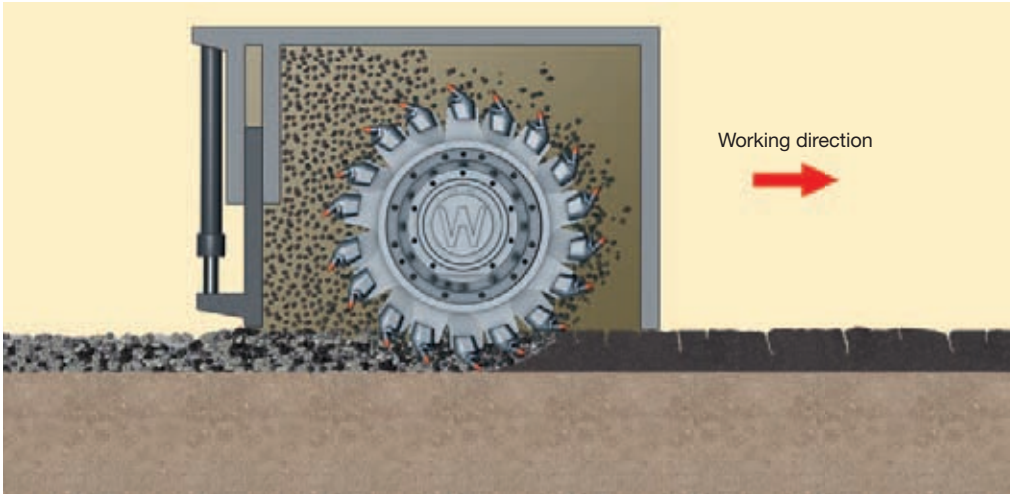
These two material classes are discussed in the following sections together with the different types of treatment that may be considered for improving the engineering properties of the recycled material, where necessary, in order to meet service life expectations of the rehabilitated pavement.

3.4.1 100% RAP recycling

Recycling only RAP material requires consideration of the following factors:

- nature and composition of the existing asphalt (i.e. type of mix, grading of aggregate, binder content, ageing of bitumen, etc.);
- type and cause of distress (i.e. permanent deformation (wheel path rutting/shoving) or cracking (thermal or fatigue mode));
- extent of distress (i.e. isolated or extensive); and
- purpose of recycling (i.e. holding action or restoring structural integrity).

Depending on the structural requirements, the recycled RAP material may be treated with an additive, or returned to the pavement as a granular material (see Chapter 6).



Recycling 100% RAP material

3.4.2 Blend of RAP / Granular material

Structural distress is usually addressed by recycling the existing pavement to depths in excess of 200 mm. The recovered material will normally include the surfacing and base layers and typically comprises bituminous surfacing material (e.g. RAP or aged chip seal material) and granular material from the underlying base (e.g. crushed stone or previously stabilised gravel). Distress in such pavements is usually manifest in the form of severe cracking in the surfacing, deformed granular layers and potholing. The structural capacity demands and the type of traffic carried on the road will largely dictate whether the materials recovered from the existing pavement are sufficiently strong or if they need to be enhanced by stabilisation, as discussed below.

Without stabilising agents

Rehabilitation by recycling the upper portion of an existing pavement does not always demand the addition of stabilising agents to improve the engineering properties of the recycled material. The two types of treatment discussed below are often considered where the road carries relatively low volumes of traffic.

- **Reprocessing.** Both surfaced and unsurfaced roads constructed from natural materials are often rehabilitated or upgraded to blacktop standards by recycling the existing base material. The primary objective of this type of recycling is to recover material from the existing pavement that already meets the strength requirements. Water is added whilst recycling to achieve the optimum moisture content for compaction. The mixed material is then placed and compacted to the correct layer thickness, surface shape and density.

- **Mechanical modification.** Surface distress or structural deterioration is sometimes caused by a mechanical deficiency in the existing base material (e.g. poor grading or plasticity). Such deficiencies can sometimes be addressed at minimal cost by blending the existing base material with a suitable imported material (e.g. graded crushed stone) that is spread as a layer on top of the existing road surface prior to recycling. Water is added during the recycling process to achieve the optimum moisture content for compaction. The mixed material is then placed and compacted to the correct layer thickness, surface shape and density.

Mechanical modification achieved by spreading new material before recycling

It should be noted that blending a clayey material with sand using this technique is not always successful, especially where the moisture content of the clay is above the optimum. Under such conditions, it is best to first “pulverise” the in situ clayey material with a recycler and allow the fluffed material to dry back. The loose material must then be pre-shaped and compacted before importing the sand and using the recycler to mix the material by making a second pass.

With stabilising agents
Stabilising agents are used to improve the engineering properties of a material and different stabilising agents used to enhance different properties. For example, stabilising with hydrated lime reduces the moisture susceptibility of a material by modifying the clay fraction, whereas a bitumen stabilising agent increases the flexural strength of a material. These are discussed in the following chapter.

Where the structural capacity of an existing pavement needs to be increased to meet additional traffic demands, the depth of recycling is generally increased to achieve the thickness of new stabilised layer that will provide such additional capacity. However, this is only possible where the existing pavement includes layers of good quality material that are sufficiently thick to accommodate such an increase in the depth of recycling.
If these conditions do not exist, an additional layer of material may be imported and spread on top of the existing road surface prior to recycling, thereby achieving the required increase in thickness.

Unsurfaced roads are often upgraded to blacktop standards by recycling the existing gravel wearing course with an appropriate stabilising agent. The upgrading of such roads to blacktop standards is normally undertaken for the following reasons:

- **Economics.** High maintenance costs normally associated with increased traffic volumes;
- **Environmental concerns.** Annual gravel loss of between 25 mm and 50 mm is common for unsurfaced roads, requiring continual regravelling with material imported from borrowpits. In addition, dust generated by unsurfaced roads has been shown to be the cause of health problems for local residents and can cause serious damage to agricultural produce.
- **Strategic decisions.** Safety concerns in wet conditions and/or political priorities.

Since the structural capacity requirement for these roads is usually low (< 300,000 ESALs) the thickness of existing gravel wearing course material normally recycled is between 100 mm and 150 mm when stabilising with bitumen. Stabilising with either cement or hydrated lime requires the depth of recycling to be increased to between 175 mm and 200 mm to achieve a similar structural life. The new base layer is normally surfaced with a relatively light chip seal with a polymer modified binder applied on a cemented base.

Where the existing base was originally constructed from cement stabilised material, distress is often a consequence of shrinkage cracking that manifests as block cracks at the surface. The distance between individual cracks reduces with time and the level of distress increases (normally associated with water ingress and pumping) until the stage is reached when rehabilitation is required. Such previously stabilised materials can usually be recycled and restabilised.

3.5 Benefits of cold recycling

Some of the more important benefits that can be realised by adopting cold recycling for pavement rehabilitation include:

- **Environmental benefits.** Full use is made of the material in the existing pavement and the volume of new material that needs to be imported from quarries is minimised. As a result, haulage is drastically reduced, as is the damage caused by heavy vehicles travelling on existing roads in the vicinity of the project. The overall energy consumed by recycling is significantly less compared to all other rehabilitation options.

- **Quality of the recycled layer.** Consistent, high quality mixing of the in-situ materials with water and stabilising agents is achieved using modern recyclers. Micro-processor controlled pumping systems ensure the accurate addition of fluids (water and stabilising agents). The tool pattern and the method of mounting on the cutting drum are specifically designed to promote mixing and achieve a homogeneous product.

- **Structural integrity.** Modern recyclers are capable of producing thick layers of bound material that are homogeneous and do not contain weak interfaces between thinner pavement layers (the lamination effect).

- **Subgrade disturbance is minimised.** Disturbance of the underlying pavement structure is minimal since recycling is typically a single-pass operation and the wheels of the recycler are not in contact with lower layers since they run on top of the recycled material.

- **Shorter construction time.** Recyclers are capable of high production rates that significantly reduce construction times compared to most alternative rehabilitation methods. Shorter construction times reduce project costs. Other benefits accrue to the road user since traffic is disrupted for shorter time periods.

- **Safety.** One of the most important benefits of this process is the relatively high levels of traffic safety that can be achieved. The full recycling operation can be accommodated within the width of one traffic lane. On roads with two lanes, recycling is usually undertaken in half-widths with one-way traffic accommodated on the opposite half during working hours. The full road width is usually opened to traffic outside working hours, including the completed recycled lane.

- **Cost effectiveness.** The above benefits all combine to make cold recycling a most attractive process for pavement rehabilitation in terms of cost effectiveness.

3.6 Applicability of the cold recycling process

Each road rehabilitation project is different in terms of the structure of the existing pavement, the quality of materials in the various pavement layers and the service life requirements. The most cost-effective method of rehabilitation will always be project-specific. It is therefore important to determine the most appropriate solution for each project and that solution may not necessarily be one based on recycling. The following important factors need to be considered in evaluating the appropriateness of recycling for a specific project:

- **Type of project.** The most effective solution for a particular country or region is influenced by the local environment, whether the project is concerned with a highly-trafficked urban street where only night work will be permitted, or a low volume unsurfaced rural road in urgent need of upgrading. Very different solutions and standards of service are required in these two extreme cases. It is important to take cognisance of the local standards for road construction, as well as the perceptions of the local population regarding the levels of service that they regard as acceptable.

- **Physical environment.** Local topography must be considered when determining the most appropriate method of rehabilitation. In particular, steep gradients may dictate the type of construction that is practically possible. Climate plays a vital part in the choice; solutions that satisfy the requirements for pavements in a dry region will certainly not be suitable for high rainfall areas and the effect of temperature extremes will also influence the suitability of different options.

- **Availability of materials.** The feasibility of various recycling options is significantly influenced by the availability of construction materials, especially stabilising agents. These must be procurable in sufficient quantities that are of a consistent and acceptable quality. Recyclers consume large quantities of water and stabilising agents and it is necessary at the outset to determine whether the required volumes can be reliably sourced and delivered.

The next chapter focuses on stabilising agents, especially those that are generally used with recycled materials. In the past, cement has been used more than all other stabilising agents combined but, as a consequence of the rigidity introduced by such treatment, the current trend is towards bitumen stabilisation and the durability offered by improved flexibility.

4 Stabilising agents

More than two thousand years ago, the pioneering road construction technology of the Babylonians and Mesopotamians was developed further by the Romans. In addition to their advanced segmented block paving systems (cobble stones), the Romans also used a form of lime treatment to improve pavement strength for heavily-loaded transport wagons. Today, many different kinds of stabilising agents are used worldwide to overcome the inherent limitations of natural materials. In addition to increasing the strength and stiffness characteristics of a material, stabilising agents improve durability and resistance to the effects of water and the environment.

Good quality road construction material is becoming increasingly scarce in many parts of the world and is often simply not available. The economic and environmental impact of importing and transporting a suitable material has encouraged innovation and the formulation of alternative solutions, such as the development of stabilisation techniques that utilise locally available resources. The required layer strength and stiffness can often be achieved using a local “marginal” material with the addition of small amounts of stabilising agents at a relatively low cost. These techniques are as applicable to recycling as they are to new construction. By adding a stabilising agent, the material recovered from an existing pavement can be improved, thereby eliminating the need to import new material to achieve the required strength in the rehabilitated pavement structure.

The purpose of stabilising agents, their behaviour and, more importantly, the primary factors influencing their selection or exclusion must be clearly understood. This chapter endeavours to address these issues and remove any misconceptions.

4.1 Types of stabilisation agents

4.1.1 General

Currently, a wide range of stabilising agents is in use globally, including:

- Wetting agents i.e. surfactants (surface active agents) e.g. sulphonated oils
- Hygroscopic salts e.g. calcium chloride
- Natural and synthetic polymers
- Modified waxes
- Petroleum resins
- Bitumen
- Cementitious stabilisers e.g. cement, lime, fly ash etc

All stabilising agents aim to achieve the same objective of binding the individual aggregate particles together to increase strength and stiffness and/or make the material more water-resistant and durable. Some agents are more effective than others when used with specific materials, some have clear cost advantages, but all have a place in the market and most are best applied using modern recycling machines.

New proprietary products are continually being developed and it is important for the industry that they be given a fair trial. Innovation should always be promoted since no single stabilising agent can claim to be the best for all applications. Engineers should maintain an open-minded approach when faced with making the decision as to which agent to use on a specific project. Such decisions are invariably influenced, in order of importance, by:

- **Price:** The unit cost of stabilising will always be the primary concern;
- **Availability:** Specific stabilising agents may not be available in some parts of the world. For example, bitumen emulsion is not manufactured in some countries;
- **Material characteristics:** Some stabilising agents are more effective than others on certain material types. For example, lime should be used in preference to cement for treating soils with high plasticity (PI >10);
- **Durability:** The desired effects of stabilisation should remain effective for the service period; and
- **Policy:** Some road authorities have rigid policies concerning the use of certain stabilising agents, often influenced by past experiences.

The approach that is adopted towards stabilising agents differs between countries and road authorities. Where these differences are dictated by policy they are often derived empirically rather than through sound technical evaluation. Technology knows no borders; strength characteristics measured anywhere in the world are comparable, provided the materials are similar and the testing criteria common. There is therefore no real reason for ruling out a specific stabilising agent that meets all relevant technical requirements.

Being at the frontier of technology can be a risky and lonely experience. Engineers are inherently conservative and tried-and-tested practices are most often preferred to experimentation with new products. Cementitious stabilising agents and, to a lesser extent, their bituminous relatives, have been well researched. They are used extensively and standard test methods are available for determining optimum mix designs and quality assurance requirements. In addition, both cement and bitumen see wide usage in the construction industry and are generally available worldwide. It is therefore not surprising that they are the most popular stabilising agents and, accordingly, are the focus of this chapter.

4.1.2 Material behaviour

Unbound (granular) materials in flexible pavements exhibit a stress-dependent type of behaviour. This means that. when confined within a compacted pavement layer, the effective stiffness characteristics increase as the loading state is increased. When materials are repeatedly loaded to stress levels that are a significant proportion of their ultimate strength, shear deformation occurs. This shear deformation accumulates, resulting in permanent deformation (rutting).

Adding a stabilising agent binds the material particles together, changing the behaviour under load such that a layer of bound material tends to act more like a slab with very different stress patterns. The fundamental difference in stress distribution for unbound and bound materials (the cone supporting the load and generation of bending stresses respectively), is shown in the figure below.

It is important to understand that hot mix asphalt (HMA) and bitumen stabilised materials (BSMs) are fundamentally different. As was explained in Section 1.2.2, although bitumen is the common binding agent, the manner in which the bitumen is dispersed among at the aggregate particles in very different. HMA is a continuously bound material whilst a BSM is a non-continuously bound material.

Cement stabilised materials are also very different from bitumen stabilised materials. Adding cement to a material promotes rigidity whilst adding bitumen tends to promote flexibility. In addition to being continuously bound, cement stabilised materials are prone to shrinkage that manifests as block cracking in the layer, which is exacerbated by repeated loading. As shown in the below figure, tensile stresses develop in the lower portion of layers constructed from bound material as the

Bound material = bending

Unbound material = stress distribution “cone”

pavement deflects under load. Repeated loading (normally millions of repetitions) causes the material to suffer fatigue failure, or “bottom-up” cracking. The type of binding agent used is one of the primary determinants of the number of load repetitions a layer can withstand before such cracking develops. This is applicable to cement stabilised materials and HMA.

BSMs are less stiff than cemented materials but have improved shear properties. The main mechanism of failure of BSMs is permanent deformation under loading. BSMs with typical bitumen contents of less than 3% do not experience fatigue cracking because they are non-continuously bound. These concepts are all discussed in the following sections.

4.1.3 Cementitious stabilising agents

Lime, cement and blends of these products with fly ash, ground blast furnace slag and other such pozzolanic materials, are the most commonly used stabilising agents. Apart from the early Roman experiments with lime as an agent, cement has been in use for the longest period of time; the first recorded application as a formal stabilising agent being in the USA in 1917.

The primary function of these agents is to increase bearing strength. This is achieved either by significantly increasing tensile and compressive strength of the material or by reducing plasticity. Cement is the stabiliser that provides most enhancement of strength. Free lime released during the hydration process reacts with any clay particles that may be present, thereby reducing plasticity. Lime, however, is comprised predominantly of free lime and is therefore the preferred stabilising agent for more plastic materials (PI>10%). The use of cement and cement blends should be limited to the treatment of materials with a Plasticity Index of less than 10. The strength achieved is governed by the amount of stabilising agent added and the type of material being treated. It must, however, be recognised that adding more cement to obtain higher strengths can be detrimental to the performance of the layer. Cementitious stabilising agents produce semi-brittle materials. An increase in the strength of a stabilised layer results in an increase in the brittleness, with a consequential reduction in flexibility. Higher strength levels in the cemented layer attract more stresses from the wheel loading as a consequence. This invariably leads to accelerated crack proliferation under repeated heavy traffic loading, thereby reducing structural performance. It is therefore important that the performance requirements of the stabilised layer are unambiguously understood and that a proper mix design is conducted on representative samples to determine the correct stabiliser application rate.

Note:

- Cement stabilisation increases strength and stiffness but introduces shrinkage cracking

4.1.4 Bitumen stabilising agents

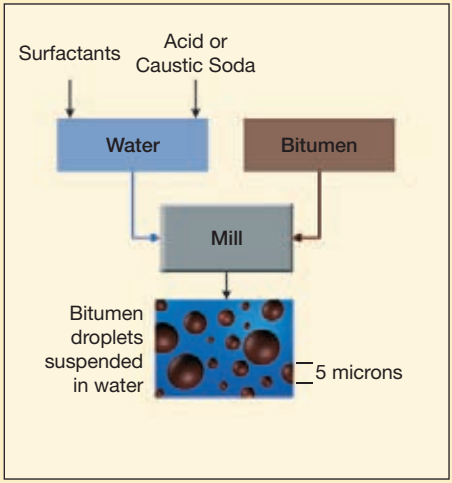
Due largely to technological advances (design procedures and construction methods) and the benefits that accrue (economic and environmental), the use of bitumen as a stabilising agent has become increasingly popular. Although there are many forms of bitumen, only two are used as stabilising agents: bitumen emulsion and foamed bitumen, both produced from relatively soft road-grade bitumen (e.g. 80 Pen). Both may be used to treat a wide range of pavement materials, dispersing the bitumen in a non-continuous manner, the hallmark of a bitumen stabilised material (BSM). Treating a material with cut-back bitumen is not a stabilisation process since the bitumen disperses in a continuous manner, as in asphalt.

Materials stabilised with bitumen do not suffer from the shrinkage cracking phenomenon associated with cement stabilisation. A layer constructed from a BSM is relatively flexible compared a layer of the same material treated with cement. BSMs may be trafficked immediately after construction due to the substantial increase in cohesion that is realised when the material is compacted. This cohesion reduces the tendency of the material to ravel under the action of traffic. Bitumen stabilisation improves the strength of a material and reduces the detrimental effects of water.

The following describes the two bitumen stabilising agents and explains the key differences

Bitumen Emulsion

This binder comprises bitumen emulsified in water. The bitumen is dispersed in the water in the form of an oil-in-water type bitumen emulsion. The bitumen is held in suspension by an emulsifying agent, which determines the charge of the bitumen emulsion. Cationic bitumen emulsion has a positive charge; anionic bitumen emulsion has a negative charge. Bitumen emulsion is manufactured in a specialised plant and has a shelf life of several months in barrels, provided it is stored appropriately.



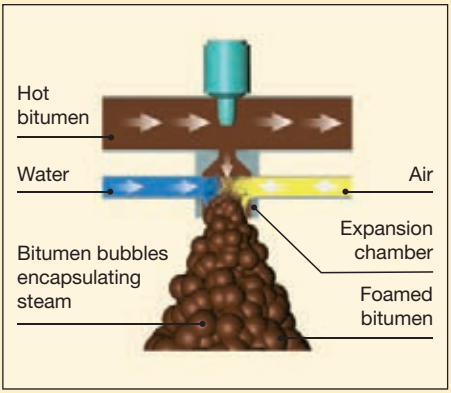
Manufacture of Bitumen Emulsion

When an emulsion is mixed with aggregate, the charged bitumen droplets are attracted to the oppositely charged aggregate particles, focusing on the smaller fractions due to their surface area and charge concentration features. The moisture and type of aggregate play an important role in dispersing the bitumen emulsion and "breaking" (separating the bitumen from the water) during mixing. Since the bitumen emulsion acts as a lubricating agent, the break should occur only after the material has been compacted. The treated material will have a "speckled" appearance due to the concentration of bitumen on the finer particles (as was illustrated in Chapter 1) resulting in localised non-continuous bonds ("spot welding").

Material stabilised with bitumen emulsion is termed **BSM-emulsion**.

Foamed Bitumen

This binder is produced by injecting water into hot bitumen, resulting in spontaneous foaming. The physical properties of the bitumen are temporarily altered when the injected water, on contact with the hot bitumen, is explosively transformed into vapour, which is trapped in thousands of tiny bitumen bubbles. The foaming process occurs in an expansion chamber (a relatively small thick-walled steel tube, approximately 50 mm in depth and diameter) into which bitumen and water (plus air on some systems) is injected at high pressure. Foamed bitumen bubbles collapse in less than a minute.



Foamed Bitumen Production in Expansion Chamber

Foamed bitumen is produced at the mixing chamber and incorporated into the aggregate while still in its "unstable" foamed state. The greater the volume of the foam, the better the distribution of the bitumen in the aggregate.

During mixing, the bitumen bubbles burst, producing tiny bitumen splinters that disperse throughout the aggregate by adhering to the finer particles (fine sand and smaller) to form a mastic (as was illustrated in Section 1.2.2). The moisture content of the material prior to mixing plays an important role in dispersing the bitumen. On compaction, the bitumen particles in the mastic are physically pressed against and adhere to the larger aggregate particles resulting in localised non-continuous bonds ("spot welding").

Material stabilised with foamed bitumen is termed **BSM-foam**.

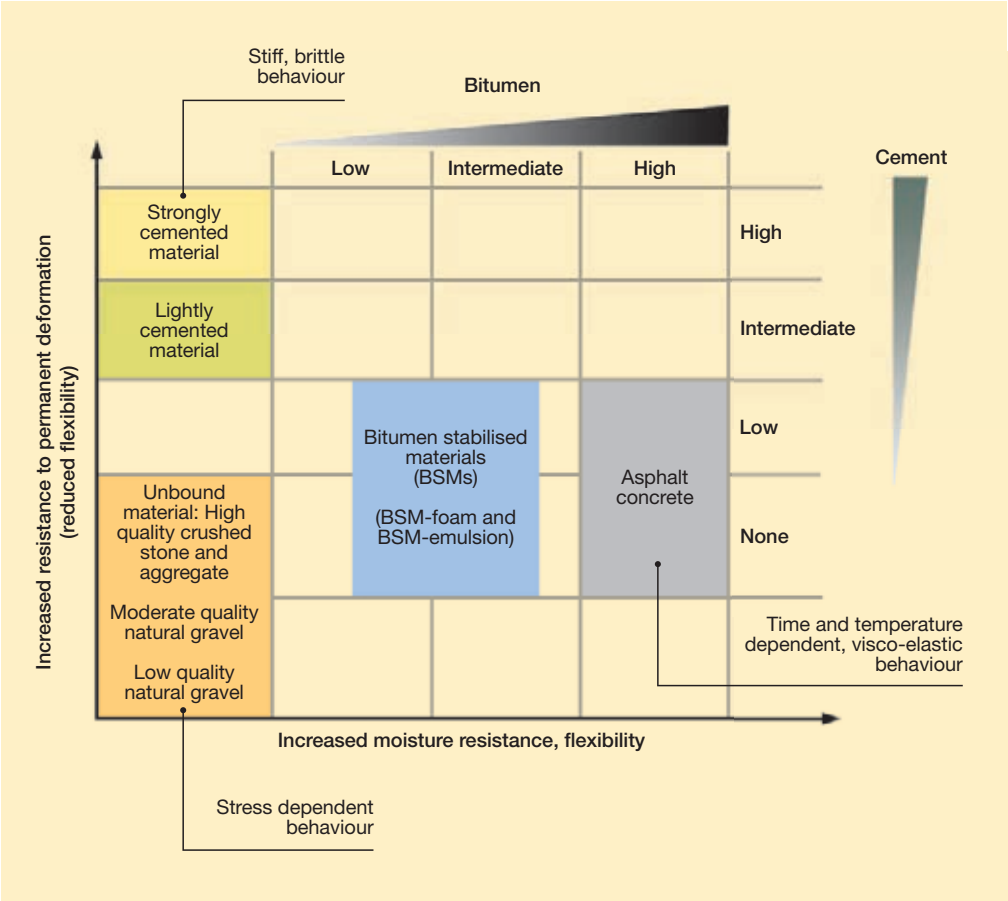
Current practice prefers to treat BSM-emulsion and BSM-foam equally in terms of their performance properties. The main features of the behaviour of BSMs are:

- A significant **increase in cohesion** in relation to that of the parent granular material, with **no significant reduction in friction angle**,
- Acquiring of **flexural strength** due to the visco-elastic properties of the bitumen,
- Improved **moisture resistance and durability** in relation to the parent granular material.

Some materials treated with a bituminous stabilising agent have poor retained strength properties (ie. they lose strength when immersed in water). This can be addressed by the addition of an "active filler" such as hydrated lime or cement. Small amounts of active filler (1% by mass) can significantly increase retained strength without affecting the flexibility of the layer. Active filler acts as a dispersion catalyst with foamed bitumen and promotes breaking when used with bitumen emulsion. It is therefore common practice to use cement or hydrated lime in conjunction with bituminous stabilising agents.

4.1.5 Summary of different stabilising agents

Conceptually, the effect of adding cement and/or bitumen to a pavement material and the properties of the different products are illustrated in the figure below.



Pavement material behaviour

4.2 Stabilising with cement

4.2.1 General

Cement is the most commonly used stabilising agent; its use worldwide far exceeds all other stabilising agents combined. The main reasons for this are cost and availability; cement is manufactured in most countries throughout the world and is relatively inexpensive. Another reason is its proven track record as a construction material. There is a plethora of standards, test methods and specifications available and cement stabilised layers have provided excellent service on thousands of kilometres of roads.

Cement stabilisation, however, requires a proper design approach. The primary function of cement addition is strength gain and the Unconfined Compressive Strength (UCS) has achieved global acceptance as the principal design criterion. However, several factors other than UCS need to be considered, such as the rate of strength gain, the Indirect Tensile Strength (ITS), cracking potential and durability issues. These are addressed in the following sections.

4.2.2 Factors affecting strength

The compressive and tensile strength achieved in a cement stabilised material is largely determined by the amount of cement that is added, the material type, the density of the compacted material and extent of curing. Strength generally increases in a linear relationship with cement content, but at different rates for different materials and cement type. Density plays a major role in determining the ultimate strength whilst ambient temperature directly affects the rate of strength gain; the higher the ambient temperature, the faster the rate of gain of strength.

Crystalline bonds start forming between particles as soon as cement comes into contact with water in the mixing process. Some of these bonds are destroyed when the material is compacted, thereby reducing the strength that can be achieved. In addition, such bonding has the effect of reducing the maximum density achievable. It is therefore important to expedite the placing and compaction operations and complete them as soon as possible after recycling, in order to achieve maximum density as well as obtaining the anticipated strengths from the compacted material.

This is particularly important where ambient temperatures exceed 40º C and where the material is prone to rapid strength gain (e.g. amorphous silica reaction). Under such conditions, an alternative stabilising agent to ordinary Portland cement should be investigated, such as blends of slagment and/or lime, with a slower rate of strength gain. It should also be noted that the finer the cement powder, the faster the rate of cementation.

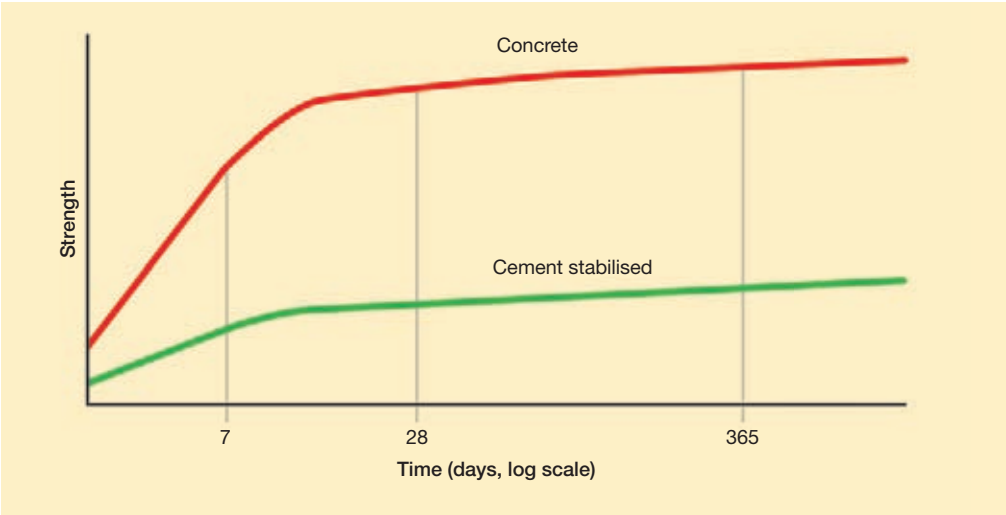
Note:

- The strength of cemented material generally increases linearly with cement content

4.2.3 Cracking of cement stabilised layers

All cement-treated materials, including concrete, are prone to cracking. The rate of gain of compressive and tensile strength in cement stabilised material is a function of time, as shown in the sketch.

Tensile stresses develop within a cement-treated material as a result of shrinkage and/or traffic and, if these exceed the tensile strength at that time, cracks occur.



Strength/time relationship for cemented material

Such cracks can be controlled and are not necessarily detrimental. However, it is important to recognise that cement treated material tends to crack for two very different reasons. The first is caused by shrinkage that is a function of the chemical reaction that takes place when cement hydrates in the presence of water and is therefore not traffic induced. The second is caused by the repeated loading of traffic over a period of time. Crack initiation and subsequent propagation are entirely different processes, warranting that they be considered separately.

Shrinkage cracks. Cracks are inevitable when a material is treated with cement. As cement hydrates, complex "finger-like" calcium silicate crystals form, bonding the material particles together. In addition to heat generation, numerous other changes take place during this chemical reaction. As the bonds develop, the material experiences a volume change and shrinks, causing cracks that are commonly referred to as shrinkage cracks. These shrinkage cracks are unavoidable and are one of the features of working with cement.

The intensity (crack spacing) and magnitude (crack width), known collectively as the degree of cracking, is largely influenced by:

- **Cement content.** The shrinkage that occurs during hydration is a function of the amount of cement present. Increasing the cement content therefore increases the degree of cracking and is one of the underlying reasons for minimising the addition of cement to achieve only the design requirements. However, as discussed below, strength and durability requirements have to be balanced and it is therefore not always possible to keep cement addition low;

- **Type of material being stabilised.** Some materials tend to shrink more than others when treated with cement. In addition, some plastic materials tend to be active, exhibiting significant volume changes between moist and dry states. Where the PI of the material is in excess of 10, the addition of lime, or a combination of lime and cement should be used to reduce plasticity, ideally to a non-plastic state;

- **Compaction moisture content and cement : water ratio.** The degree of cracking is a function of the amount of moisture that is lost as the cement hydrates and the material dries. Limiting the moisture content (or lowering the water : cement ratio) at the time of compaction to less than 75% of saturation moisture content can significantly reduce the degree of cracking;

- **The rate of drying.** When cement treated material shrinks, internal stresses are induced within the material. The degree of cracking is largely determined by the rate of strength development relative to the rate of shrinkage stress development. If the material dries quickly then shrinkage stresses will inevitably be greater than strength development and the crack pattern will be intense (2 m x 2 m) with narrow cracks (typically hair-line). Slow drying will see a less intense pattern develop (6 m x 4 m) with wider cracks. Proper curing of the completed layer will prevent the surface from drying out, thereby reducing both the intensity and magnitude of cracking; and

- **Inter-layer bonding.** A rough interface with good bonding between the cemented layer and the underlying layer will result in a high intensity of hair-line cracks, as described above. This scenario of full-friction between layers is most common. However, in exceptional cases, poor inter-layer bonding can occur, resulting in a less intense pattern with wider cracks.

A feature of shrinkage cracks is that they are wider at the top than at the bottom (drying initiates at the surface) and the vertical face is irregular, allowing for the effective transfer of traffic loads across the crack.

Cracks caused by traffic. These cracks occur as a consequence of repeated tensile stresses induced by traffic loads in the cement stabilised layer. Crack initiation occurs at the bottom of the layer where tensile stresses are maximum, causing maximum strain. Being semi-brittle with relatively poor flexural properties, cement treated layers are extremely sensitive to overloading (the proverbial pane of glass on a feather bed).

Even without overloads, damage from continuous trafficking accumulates in the cemented layer, ultimately resulting in fatigue cracking. Once initiated, the cracks take some time to propagate through to the surface of the layer. In its post-cracked state, the layer is still able to support traffic loads, a state that can be modelled by reducing the effective modulus of the cement treated layer. Crack intensity and magnitude increase as the layer deteriorates further under repeated traffic loads. This reduces the effective modulus that, in turn, increases deflection under load, thereby promoting a continuous process of degradation until ultimately the material approaches its pre-stabilised granular state. It is important to note that the rate of degradation accelerates once cracks reach the surface and allow water to penetrate the pavement more freely. The wetter the region, the higher the risk of this moisture accelerated distress.

Cracks caused by traffic

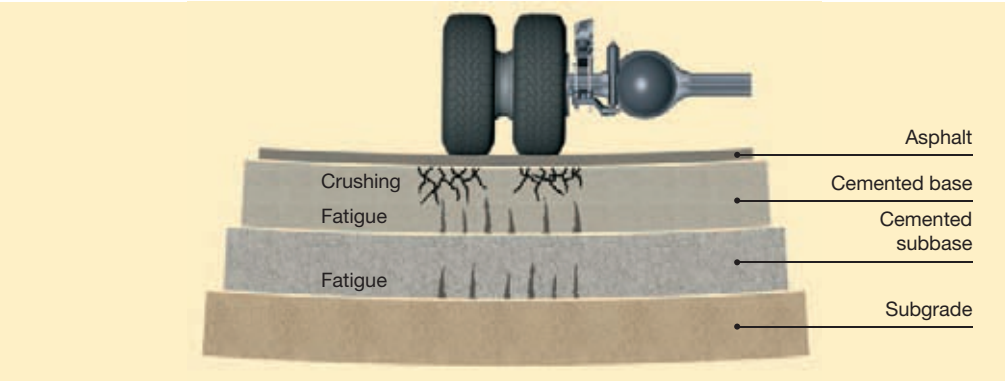
4.2.4 Surface crushing

Crushing occurs in the upper portion of a cement-stabilised base layer when traffic induced stresses exceed the compressive strength of the material. This failure mechanism was initially identified in lightly-cemented base layers with relative thin surface treatments on roads carrying heavy coal traffic in South Africa. The conclusions drawn from research into this mechanism showed that the potential for crushing failure depends on:

- the compressive strength of the stabilised material in the upper portion of the layer;
- the thickness and type of surfacing; and
- the tyre pressure as well as the applied axle load.

Design procedures need to address these conditions. Increasing the compressive strength and thickening both the stabilised and surfacing layers will largely solve the problem. However, overloading remains the major cause of premature pavement failure where axle control is ineffectual. In addition, increasing tyre pressures over the last decade has exacerbated the potential for this failure mechanism.

Furthermore, cement-stabilised base layers are vulnerable to crushing failure if subjected to heavy traffic loading before sufficient strength has developed. This applies especially to traffic that is accommodated on newly constructed cemented layers rather than deviations. The cement normally used for stabilisation and the amount applied results in a relatively slow rate of strength gain after construction, taking some 7 days to achieve 50% of the ultimate strength level (and 90% after 28 days). Thus, when the target UCS for a base layer is 2 MPa, less than 0.5 MPa would have been achieved after 3 days, making the surface particularly vulnerable to slow-moving heavy vehicles with high tyre pressures.



Mechanisms of failure for cemented layers

4.2.5 Durability concerns

The durability of natural material is mainly related to weathering and degradation of the individual particles under the influence of climatic conditions and repeated traffic loads. Such degradation is a slow process and material properties usually remain reasonably constant over the life span of the road, especially if higher-quality materials are used. However, when poorer quality material is stabilised with cement, additional aspects concerning durability need to be considered. Under certain conditions their properties can change over short periods of time due to carbonation and climatic influences.

The CBR (California Bearing Ratio) test is widely used as an indicator of bearing strength for natural materials but is not appropriate for higher strength cemented materials. The UCS test was therefore adopted and relevant limits are used worldwide (e.g. maximum 4 MPa, minimum 2 MPa). However, research has shown that UCS alone is not a reliable indicator of durability; a stabilised material meeting the specified UCS requirements can deteriorate and disintegrate over a short period of time. Additional tests are therefore required to ensure that a cement stabilised material is sufficiently durable, particularly against the potentially destructive effects of carbonation.

Carbonation is the name given to a complex chemical reaction that occurs between a cemented material and carbon dioxide in the presence of water or water vapour (humidity). In layman's terms, this reaction produces calcium carbonate from the free calcium ions. The calcium hydroxide molecules (or free lime, Ca(OH)2, that are always present in a cement treated material) are converted into agricultural lime CaCO3 through the process of carbonation. This allows plasticity (measured by the PI) to return to the material and increase rapidly. Agricultural lime is useful to increase pH but does not assist in reducing PI. The change in molecular structure during carbonation is also associated with a volume change (either decrease or increase, depending on the quality of the aggregates and type of chemical stabilisation). Where the forces emanating from such volume change exceed the strength of the cement treated material, destruction occurs.

This phenomenon is well known in the concrete industry, but is seldom a cause for concern since the tensile strength of concrete is always far in excess of the stresses induced by carbonation. In addition, the aggregate used in the manufacture of concrete is generally crushed stone with excellent durability properties. This is not the case where relatively poor-quality materials are stabilised with a low application rate of cement.

Several durability-type tests can be carried out in the laboratory to determine the potential for carbonation, such as the wet/dry brushing test, determination of the Initial Consumption of Lime (ICL) or cement (ICC) and the ITS value. As a general guideline for minimising the risk of carbonation, sufficient cement must be added to achieve both a minimum ITS value of 250 kPa (regardless of whether the related UCS exceeds the prescribed limit) as well as satisfying the ICC demand.

Note:

- The initial consumption of stabiliser (ICS) needs to be satisfied to avoid premature carbonation (i.e. durability problems).

4.2.6 Working with cement

As described above, one of the main concerns with cement treated material is the inevitable shrinkage cracking that occurs. However, the degree of cracking and the overall quality of stabilised layers are largely contingent on the following key factors:

Mix design. It is of primary importance that a proper mix design is carried out on truly representative samples of the material to be treated with cement. (An example of such a laboratory procedure is included in Appendix 1.) Different materials require different application rates of cement to achieve targets for strength and durability.

Cement quality. Cement has a definite shelf life and, as a rule of thumb, should not be used more than three months after the date of manufacture. Determining the age of cement is difficult, particularly when imported in bulk. If there is any doubt about the age, or any other aspects of quality, samples should be tested to check the strength parameters.

Cement type. Finely-ground cement with rapid-hardening properties should never be used as a stabilising agent.

Uniformity of application. Two different methods are popular for the application of cement as a stabilising agent on recycling projects. The first spreads dry cement powder on top of the existing road surface prior to recycling, the second injects a cement slurry into the mixing chamber whilst recycling:

Bulk spreaders. This is the most widely used method of application. Several different systems are used for discharging the cement onto the road surface at the required spread rate (belt conveyors, auger feeders, pneumatic blowers) and each has its own particular merits, and demerits. The “canvas patch test” is normally conducted to verify the application rate. All bulk spreaders have their limits and care must be exercised when trying to apply very low application rates (< 2%). (Streumaster spreaders can provide application rates of < 2% cement or lime with acceptable accuracy.)

Any form of dry cement spreading is affected by the weather, particularly wind and rain. Being a fine powder (smaller than 0.075 mm), cement is susceptible to wind erosion and readily becomes airborne when fanned by a breeze, whether natural or caused by passing trucks, thereby affecting the rate of application non-uniformly. If rain makes contact with spread cement, it triggers the hydration process. When this occurs, the spread cement must either be mixed immediately or discarded;

Streumaster spreading

Applying cement with the Wirtgen WR 2500 SK (dustless)

- **Recycler equipped with an integrated spreading device.** The WR 2500 SK is a “stretched” version of the standard WR 2500 S recycler with a 4 m³ hopper integrated immediately behind the operator’s cab. Cement or lime is drawn from this hopper by means of a cellular wheel sluice and spread uniformly on the road surface immediately in front of the mixing chamber. This “dustless” system is accurate for spread rates below 2% and up to 6% and addresses all the weather related concerns of spreading cement on the road ahead of the recycler.

- **Slurry injection.** The Wirtgen WM 1000 is specifically designed to premix cement with the amount of water required to achieve the optimum compaction moisture content. The slurry suspension thus formed needs to be sufficiently liquid to be pumped to the recycling machine and injected into the mixing chamber through a spraybar. The water : cement ratio is usually in the region of 1:1, but most recycling applications call for more water than cement to achieve the optimum compaction moisture content.

Applying cement as a slurry using the Wirtgen WM 1000 (dustfree)

Slurry injection is the most efficient means of dispersing the cement throughout the recycled material. This method of application is recommended where high application rates are specified (>4%) for deep recycling (>200 mm) when bulk spreading becomes unmanageable due to the sheer volume of cement required per square metre. (If the thickness of the layer of cement spread on the road exceeds 25 mm, extreme care must be taken to maintain consistency.

In addition, low application rates (1%) that are usually specified when stabilising with a bitumen stabilising agent are best applied by means of cement slurry injection to ensure uniformity of application throughout the recycled material.

Furthermore, the truly dustfree application that is achieved using this method offers significant environmental benefits, both in terms of improved health and safety for the workforce and in the reduction of wind-blown pollution.

Uniformity of mixing. Sufficient tests have been conducted to prove that the mixing capabilities of large recyclers are similar to off-site plant mixers, provided the machine is operated at an advance speed conducive to the specific site conditions (normally between 8 m/min and 12 m/min). It is therefore not necessary to apply a "traditional factor" to the specified application rate of cement as an allowance for site losses and inefficiencies.

Water addition. Cement treated material should be worked as dry as possible, both to minimise shrinkage cracking and to prevent heaving during compaction. Where the addition of water is required, it should always be injected into the mixing chamber and such addition must be judiciously controlled to obtain a moisture content that never exceeds the optimum moisture content of the material.

Curing. Once complete, the surface of a cement stabilised layer must be prevented from drying out for a period of at least seven days. As described above, shrinkage cracks will develop at the surface if the rate of drying exceeds the rate of gain of strength. Drying out can be prevented by frequently spraying the surface with water from a tanker fitted with a full-width spraybar. Other construction traffic should be kept off the layer. Alternatively, a temporary seal can be applied as a curing membrane. As a general rule, cement treated materials should always be covered as soon as possible to minimise the detrimental effects of rapid drying and carbonation.

Temperature. Where ambient temperatures are above 35° C, short sections of road should be treated and finished off as quickly as possible to prevent compacting against the inevitable rapid gain of strength.
Due to the expansion that occurs when water cools below 4° C, no cement stabilisation work should be undertaken if freezing conditions are forecast.

Control tests. The quality of the completed layer is often judged on the strength (UCS, ITS) of specimens manufactured from samples collected behind the recycler. Where this is done, it is important to regularly monitor the time lapse between sampling in the field and compacting the specimens in the laboratory. These tests must simulate field conditions. Any significant delay could result in poor strengths due to the cement hydrating and gaining strength that is subsequently destroyed by compaction.

4.2.7 Early trafficking

Outside normal working hours the full road-width is sometimes opened to traffic. Concerns regarding the early trafficking of cement stabilised material are often expressed. As discussed in Section 4.2.4, such concerns are certainly justified where heavy axle loads can be expected and where the required curing procedures are not followed.

Allowing the surface to dry out can lead to ravelling and loss of strength in the upper portion of the layer, ultimately causing potholes to develop.
The surface should therefore be kept constantly damp by frequent light watering.

4.2.8 Key features of cement stabilised materials

The three most important features of a cement stabilised material are:

Strength. Both the compressive and tensile strength, measured by UCS and ITS tests respectively, are important parameters for evaluating cement-stabilised material.

- **The UCS value.** The UCS test is normally used for evaluating cemented materials. The UCS value is usually determined from prepared specimens that have cured for 7 days at a temperature of 22°C and a humidity of over 95%. Some test methods allow the curing to be accelerated.

The table below shows typical cement application rates (expressed as a percent of the dry density of the recycled material compacted to the target density) for two UCS categories: "lightly-cemented" (less than 4 MPa) and "cemented" (up to 10 MPa).

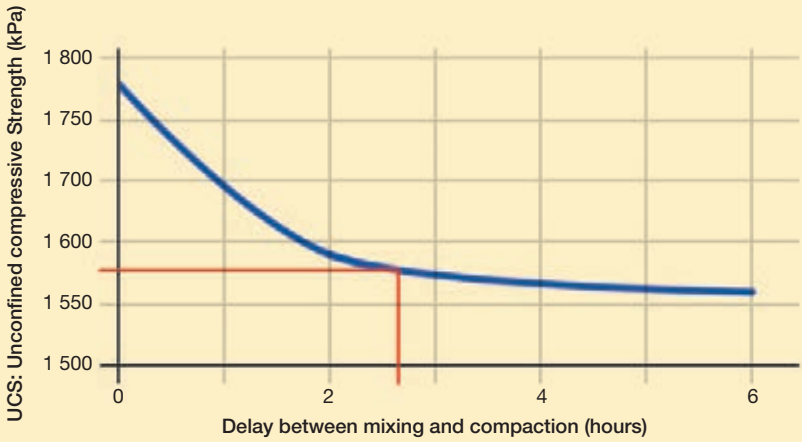
Caution: when working with coarse material. Due to the increased probability of "stone columns" developing within the body of the specimen, a false UCS measurement that approximates the stone strength rather than that of the stabilised mix can result. Unexpectedly high measurements should therefore be investigated by visually inspecting the specimen to determine the extent of aggregate crushing that occurred during the test. Repeat tests may be necessary to achieve a statistically reliable result.

Typical cement application rates (percent by mass)		
Material type	Target UCS value	
	< 4 MPa	Up to 10 MPa
RAP / crushed stone (50 / 50 blend)	2.0 to 3.0	3.5 to 5.0
Graded crushed stone	2.0 to 2.5	3.0 to 4.5
Natural gravel (PI < 10, CBR >30)	2.5 to 4.0	4.0 to 6.0

- **The ITS value.** The ITS test shows greater sensitivity to stabiliser content than the UCS test and is also becoming increasingly important as a measure of long-term durability. As described in Section 4.2.5, recent research has shown that a minimum ITS value of 250 kPa is required to resist the destructive forces generated by carbonation.

Processing time. Mixing, placing, compacting and finishing should be carried out in the shortest possible time. A time limit of 4 hours is normally specified for cement treatment, measured from the time that the cement first comes into contact with the material and moisture, to the time when compaction is complete. This may be generous where there is a potential for rapid strength gain (see Section 4.2.2).

It is strongly recommended that the allowable delay betwen mixing and compaction is checked by analysing the rate of strength gain versus delay time for the material to be stabilised under simulated field conditions (especially temperature).



The example in the figure shows that a time of 2.5 hours is allowable to achieve the required strength. It is important to minimise this time. With proper planning, this time period can be reduced to less than one hour using modern recycling and compaction equipment.

Density. Compaction should always aim to achieve the maximum density possible under the conditions prevailing on site (the so-called "refusal density"). A minimum density is usually specified as a percentage of the modified AASHTO density, normally between 97% and 100% for cement-treated bases. A density gradient is sometimes permitted by specifying an "average" density. This means that the density at the top of the layer may be higher than at the bottom. Where specified, it is normal also to include a maximum deviation of 2% for the density measured in the lowest one-third thickness of the layer. Hence, if the average density specified is 100%, then the density at the bottom of the layer must be more than 98%.

Note:

- The available working time in the laboratory must simulate field conditions

4.3 Stabilising with bitumen

Great strides have been made in recent years with regard to the research of bitumen stabilised materials (BSMs) and the understanding of their key performance parameters. The Technical Guideline TG2 Second Edition (2009) captures the essence of these advancements, incorporating the latest approaches to mix and pavement design of BSMs. This section summarises the important features of these developments.

4.3.1 Overview

Bitumen is a versitile binder that is used in pavement layers in several different forms. However, since bitumen is a highly viscous liquid and therefore unworkable at ambient temperatures, the viscosity must first be reduced to achieve workability. In general, there are three ways of doing this:

- applying heat (increasing the temperature of bitumen and aggregate);
- emulsifying in water to form bitumen emulsion; or
- creating foamed bitumen in a temporary state of low viscosity.

This chapter is focused on the use of bitumen emulsion and foamed bitumen which are the only two viable bitumen stabilising agents. As described in Chapter 6, although bitumen emulsion can also be used as a rejuvenating agent for 100% RAP mixes, this section focuses on stabilisation only.

The following table compares the treatment process for hot mix asphalt with those for the two means of stabilising a material with bitumen: BSM-emulsion (material stabilised with bitumen emulsion) and BSM-foam (material stabilised with foamed bitumen).

Comparison between different types of bitumen treatment			
Factor	Stabilisation process		Hot-mix Asphalt (HMA)
	BSM-emulsion	BSM-foam	
Aggregate types applicable	- Crushed rock - Natural gravel - RAP, stabilised (Cold mix – Chapter 6)	- Crushed rock - Natural gravel - RAP, stabilised - Marginal (sands)	– Crushed rock – 0% to 50% RAP
Bitumen mixing temperature	20° C to 70° C	160° C to 180° C (before foaming)	140° C to 180° C
Aggregate temperature during mixing	Ambient (> 10° C)	Ambient (> 15° C)	Hot only (140° C to 200° C)
Moisture content during mixing	OMC plus 1% minus emulsion addition	"Fluffpoint" 70% to 90% of OMC	Dry
Type of coating of aggregate	Coating of finer particles (and some coarse particles). Increased cohesion from the bitumen/fines mortar	Coating of finest particles only. Increased cohesion from the bitumen/fines mortar	Coating of all aggregate particles with controlled film thickness
Construction and compaction temperature	Ambient (> 5° C)	Ambient (> 10° C)	140° C to 160° C
Air Voids	10% to 15%	10% to 15%	3% to 7%
Rate of initial strength gain	Slow (moisture loss)	Medium (moisture loss)	Fast (cooling)
Modification of bitumen	Yes	No. (Modifiers are generally anti-foamants)	Yes
Important bitumen parameters	- Emulsion type (anionic, cationic) - Residual bitumen - Breaking time	- Foaming properties • Expansion ratio • Half-life	- Penetration - Softening point - Viscosity

The type of material that is produced when treating with either bitumen emulsion or foamed bitumen is similar. This has allowed a common approach to be developed for both types of treatment. There are, however, some nuances, primarily concerned with the moisture/fluid content; these are highlighted in the following sections.

Although BSMs may appear to be "challenging materials" due to the different number and types of possible ingredients, they are stabilised materials, not HMA derivatives. Similar to designing for cement stabilisation, each component in the mix needs to be optimised to formulate a composite product for a specific purpose or application. These components include the material (aggregate) to be treated, water, bitumen and active filler, each with its own variability, availability and cost. To produce a BSM with the necessary quality and consistency to fulfil the intended function, sound procedures need to be followed that assist in identifying optimal formulation, blending and production. This process is the mix design procedure.

The formulation of a BSM requires the consideration of volumetric/compaction characteristics as well as the engineering/durability properties. As with all stabilised materials, the performance of the treated product is largely dictated by the quality of the parent material and its appropriateness for treatment with the selected stabilising agent. The mix design procedure therefore aims to determine the potential of the material in terms of structural performance (resistance to permanent deformation), and durability (resistance to moisture and deterioration). At the same time, economic considerations remain paramount in the selection of BSMs. Since bitumen contributes significantly to the cost of a BSM, the need for effective optimisation of the amount of bitumen added to the mix is of primary importance.

4.3.2 BSM Distress Mechanisms

Project-specific conditions (e.g. available materials, stabilising agents, climate, traffic, supporting layers, construction techniques, etc.) all play a role in the performance of a material and its mode of distress. BSMs need to be appropriately selected for the particular design conditions. By changing the proportions of parent material, blending aggregates, bitumen and active filler, is possible to create a BSM mix that meets specific behavioural characteristics.

There are two fundamental failure mechanisms of BSMs that need to be considered:

- **Permanent Deformation.** This is the accumulation of shear deformation (plastic strain) as a result of repeated loading and is dependent on the material's shear properties and densification achieved. Resistance to permanent deformation (rutting) is enhanced by:
  - Improved material (aggregate) strength, angularity, shape, hardness and roughness;
  - Increased maximum particle size;
  - Improved compaction (field density);
  - Reduced moisture content (curing);
  - Addition of a limited amount of bitumen, usually less than 3.0%. Higher bitumen contents encourage instability due to the lubricating effect of excess bitumen and a consequential lowering of the angle of internal friction; and
  - Addition of active filler, limited to a maximum of 1%. Higher application rates of active filler introduce brittleness which encourages shrinkage and traffic associated cracking

- **Moisture Susceptibility.** The partially coated nature of the aggregate in a BSM makes moisture susceptibility an important consideration. Moisture susceptibility is the damage caused by exposure of a BSM to high moisture contents and the pore-pressures induced by wheel loads. This results in loss of adhesion between the bitumen and the aggregate. Moisture resistance is enhanced by:
  - Increased bitumen content, limited by stability concerns and cost implications;
  - Addition of active filler (maximum 1%);
  - Higher field densities through improved compaction; and
  - stabilising a material with a smooth continuous grading

Note:

- BSM behaviour is similar to unbound granular materials but with greatly improved cohesion and reduced moisture sensitivity

4.3.3 Primary determinants of BSM performance

The performance of a pavement layer constructed from BSM is determined primarily by:

- The quality and consistency of the parent material (together with any blending material) that was stabilised;
- The quality and appropriateness of the bitumen stabilising agent and active filler applied;
- The dosage of bitumen stabilising agent/active filler and the mixing effectiveness;
- The density achieved in the field by applying appropriate compaction effort; and
- The thickness and uniformity of the constructed layer.

In addition, key features of the existing pavement remaining beneath the recycling horizon have a significant effect on the layer of BSM, both in the as-built characteristics (especially density) and in the overall performance of the pavement:

- The quality and uniformity of the underlying supporting layers (pavement composition);
- The prevailing climatic conditions (hot/cold, wet/dry); and
- The effectiveness of drainage provisions that will influence the equilibrium moisture content of all pavement materials, including the BSM.

Furthermore, additional layers placed on top of the new BSM layer provide protection from the environment and high traffic-induced stresses. Ultimately, as was explained in Chapter 2, pavements are designed to accommodate specific levels of traffic loading. Although a layer of BSM is only one of several components that make up the overall pavement structure, each feature influencing performance of the BSM layer needs to be understood to ensure that an appropriate design is ultimately achieved.

4.3.4 Material to be stabilised with bitumen

The quality and composition of material recycled from an existing pavement can vary considerably. Such variations are due to:

- The structure of the existing pavement (materials in the various layers and their thickness);
- Construction variability (material quality and thickness);
- Depth of recycling (additional layers may be encountered in the recycling horizon as the depth increases);
- Age of the pavement (particularly for previously-treated materials and materials prone to weathering);
- Degree of patching and repair of the existing pavement; and
- Thickness and nature of old surfacing materials (e.g. asphalt and/or seals).

The recycled material should be well-graded and comply with the criteria provided below.

On some projects, the required grading may be achieved by incorporating a portion of the underlying layer into the composite recycled material. However, caution must be taken when including an underlying layer comprised of cohesive material. It is preferable to rather incorporate imported material (normally a crushed product) by pre-spreading as a layer on the surface before recycling.

In addition, the maximum particle size and amount of coarse material plays an important role in achieving sufficient density in the field. The maximum particle size should be limited to $^1/_3$ of the layer thickness (i.e. 50 mm for a 150 mm thick layer) and, as a general guideline, the amount of material retained on the 50 mm sieve should not exceed 10 %.

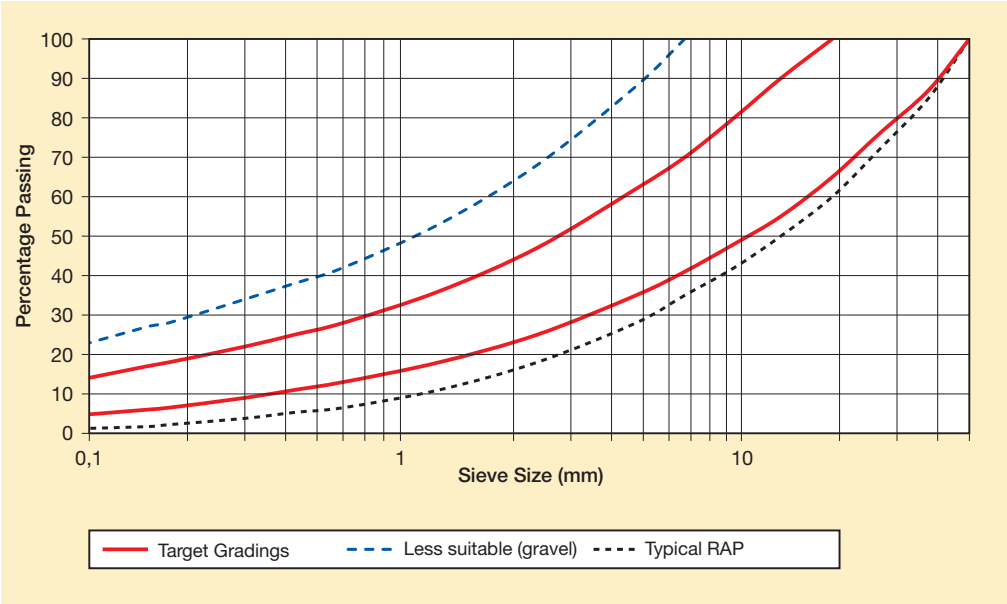
- **RAP material.** Some projects encounter a high proportion of RAP material in the recycling horizon (> 90% of the recovered material). In such cases, the influence of the aged bitumen in the RAP needs to be carefully considered, especially in warm climates where heavy traffic loading is expected. . In particular, the following aspects need attention:

  – Climatic region. In hot climates, the shear properties of the mix should be determined from triaxial tests carried out at representative temperatures;
  – Actual axle loads. Where a BSM layer is intended for use in an area where there is limited axle mass control, higher stresses from overloads may cause accelerated deformation. This needs to be considered when analysing the shear properties.
  – The composition of the RAP material. In hot regions with limited axle mass control, the RAP should always be modified by blending with 15% to 25% crusher dust. This will provide an angular sand skeleton to improve the shear resistance of the mix.

The use of 100% RAP material in BSMs is covered in detail in Chapter 6.

Two key characteristics of the parent (pre-stabilised) material are used as indicators to determine whether or not bitumen stabilisation is likely to be effective: the grading curve and plasticity index (PI). Although a CBR value can be estimated for the material using these two characteristics, it is advisable to carry out the 4-day soaked CBR test when the quality of the material is considered "marginal" for the intended application.

- **Grading.** Sieve analyses carried out on representative samples of material to be stabilised with bitumen provide a good indication of the suitability for such treatment. The following graph and table show the recommended grading envelops for material treated with either foamed bitumen or bitumen emulsion.



Target grading curves for bitumen stabilisation

Recommended grading envelops for bitumen stabilisation								
Sieve size (mm)	Treatment with Foamed Bitumen				Treatment with Bitumen Emulsion			
	Percentage passing each sieve size (%)				Percentage passing each sieve size (%)			
	Recommended gradings		Less suitable (Gravel)	Typical RAP grading	Recommended gradings		Less suitable (Gravel)	Typical RAP grading
	Coarse	Fine			Coarse	Fine		
50	100	100	100	100	100	100	100	100
37.5	87	100	100	85	87	100	100	85
26.5	76	100	100	72	76	100	100	72
19	65	100	100	60	65	100	100	60
13.2	55	90	100	50	55	90	100	50
9.5	48	80	100	42	48	80	100	42
6.7	41	70	100	35	41	70	100	35
4.75	35	62	88	28	35	62	88	28
2.36	25	47	68	18	25	47	68	18
1.18	18	36	53	11	18	36	53	10
0.6	13	28	42	7	12	27	42	6
0.425	11	25	38	5	10	24	38	4
0.3	9	22	34	4	8	21	34	3
0.15	6	17	27	2	3	16	27	1
0.075	4	12	20	1	2	10	20	0

Note. Sieve analyses are always carried out using the washed-fines test method.

The nature of bitumen dispersion is different for foamed bitumen and bitumen emulsion and is the reason for the minor differences between the two recommended grading envelops (confined to the fractions smaller than 2.36 mm).

Foamed bitumen. Unless the sample is comprised mainly of RAP material, foamed bitumen treatment relies on the dust particles (<0.075 mm) to provide a home for the bitumen splinters produced when the bitumen bubbles burst. Hence the minimum requirement for 4% of the material to pass through the 0.075 mm sieve. If there are insufficient dust particles to disperse the added bitumen, individual bitumen splinters will tend to adhere to each other, forming bitumen-rich "blobs" known as "stringers". These stringers are effectively wasted bitumen and are actually detrimental to the mix; they have a negative effect on the angle of internal friction without having a corresponding positive influence on the cohesion of the material.

BSM-foam

The dispersed bitumen splinters in BSM-foam do not coat the larger particles. The mastic (fines, bitumen and water) "spot weld" the coarser aggregate fractions together in BSM-foam. 4% fines are normally required to achieve a satisfactory mix (unless 100% RAP material is treated – see Chapter 6).

As explained in Chapter 6, particles of all sizes in a RAP material were previously coated with bitumen and this old bitumen appears to offer the bitumen splinters a home. The minimum dust requirement may therefore be reduced as the amount of RAP in the mix increases. (Recycled chip seal material has the same effect on the mix as RAP.)

Bitumen emulsion. Mixing a material with bitumen emulsion is essentially a wetting process. However, the charge on the individual emulsified bitumen droplets will tend to see a selective attraction to those particles with higher opposite charge concentrations (the finer fractions).

BSM-emulsion

Bitumen emulsion coats some of the coarse particles, not only the fines.
A minimum fines content of 2% is sufficient.

Blending. The grading curve for the material that is to be bitumen stabilised should always be plotted on a graph that includes the target gradings (taken from the above table). The plot will highlight any glaring deficiencies and suggest the need for blending as well as the type/size of blending material. Material with a grading curve that falls outside the recommended grading envelope (target range) can normally be successfully stabilised without blending, provided the following conditions are satisfied:

- Finer material. The plasticity index (PI) is below 10.
- Coarser material. The material is comprised mainly of RAP or recycled chip seal material.

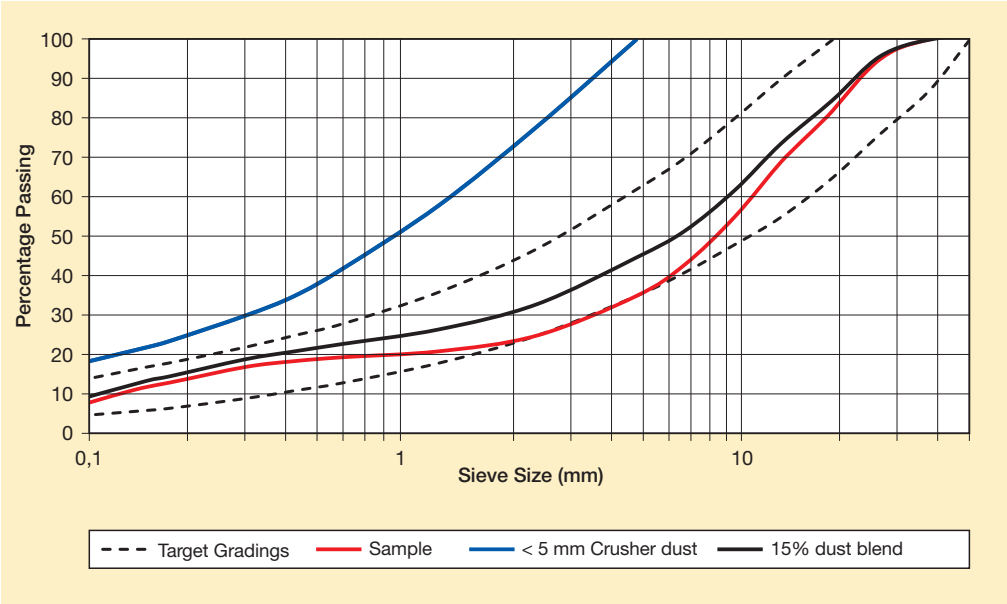
It must be appreciated that materials falling outside the target grading envelope have limitations and should therefore only be utilised where alternatives are severely limited. The finer the material, the higher the bitumen demand whilst coarser materials are prone to segregation and can be extremely difficult to work. Blending often offers a technical solution whilst reducing the overall cost.

Note:

- Under no circumstance should active filler be used as a blending material. Only inert materials (e.g. crusher dust) are to be used for blending. The maximum amount of active filler that is added to a BSM is 1.0%.
- A deficiency in fines should never be corrected by increasing the active filler to > 1%

Since bitumen stabilised materials are reliant on the sand skeleton for their performance, the portion of the plot between 0.075 mm and 2.0 mm is of primary importance. All available sieve sizes in this range for both asphalt and soils testing are used in the sieve analysis to obtain as much information on this lower portion of the grading curve and the curve should always be plotted to determine whether the curve is continuous or "bulging" (i.e. gap graded). Blending is advised where there is a marked bulge in the 0.075 mm to 2.0 mm portion of the curve, as shown in the example opposite.

In this example, blending the material with 15% minus 5 mm crusher dust largely eliminates the bulge in the curve below 2 mm. This will improve the overall strength of the material (sand skeleton) and allow a higher level of density to be achieved which will reduce moisture susceptibility as well as adding to the strength achieved in the field.



Example: Blending to correct poor grading

Alternatively, when selecting the various component materials in a blending exercise (e.g. when using a KMA 220 for in-plant mixing), the following equation can be useful for determining the required blend proportions. This allows the best particle packing and minimum voids (after compaction) to be achieved by calculating the amount required for each individual particle size.

$$P = \left[\frac{d}{D}\right]^n \times 100$$

where
- d = selected sieve size (mm)
- P = percentage by mass passing a sieve of size d (%)
- D = maximum particle size (mm)
- n = grading coefficient dependent on particle packing characteristics (a value of 0.45 is recommended)

- **Plasticity.** Plasticity in a material is attributed to the presence of cohesive clay particles in the fines fraction. Following standard laboratory test procedures, the Plasticity Index (PI) of a material is determined on the fraction passing the 0.425 mm sieve and is a primary indicator of the material's moisture susceptibility. The higher the PI value, the greater is the amount of clay in the material. Due to the shape and size of the individual particles, clay has an ability to retain relatively high levels of moisture. In such a moist state, clay is highly cohesive, implying that the particles bind together in "lumps". Recycling a high PI material will not necessarily break these lumps apart and they will remain in the new layer as localised weak spots that retain their moisture susceptible nature.

In addition, foamed bitumen stabilisation relies on the fines fraction to disperse the bitumen. If the fines include a significant amount of clay, they will not be available to do so because they will be locked together in lumps. The gradings obtained from standard tests may show that there are plenty of fines present in a material but the grading always needs to be viewed in light of the PI value; if the PI is in excess of 10, the fines are likely to be "lumped" and therefore unable to act as a dispersing agent for the bitumen.

As a general rule, if test results show the PI of the material is in excess of 10 then pre-treatment with hydrated lime should be carried out. Such treatment modifies the material by separating the clay particles, thereby eliminating plasticity.

Durability. Durability is primarily concerned with the properties of the pre-stabilised material. Where a poorer quality "marginal material" is to be stabilised with bitumen, it is advisable to check the untreated material's susceptibility to moisture change and weathering. Since the dispersion of bitumen in a BSM is selective, most of the coarse fractions will not be coated with bitumen and will remain unprotected in the mix. Determining susceptibility to moisture change and rapid weathering is therefore advisable using one of the many different methods for testing a material's durability.

Material temperature. At the time of mixing with a bitumen stabilising agent, the temperature of the material must be sufficiently high as it plays an important role in determining the quality of mix achieved.

- **BSM-emulsion**
  Typically, materials with a temperature of 10° C or higher can be treated with bitumen emulsion without compromising the bitumen distribution in the mix.

- **BSM-foam**
  The temperature of the material has a significant influence on the degree of dispersion and the properties of the mix. Higher material temperatures increase the size of the particle that can be coated whereas low temperatures can result in little or no dispersion of the bitumen. Temperature measurements of the material are therefore essential before laboratory or field production commences.

Mixing should not be attempted with material temperatures less than 10° C. Where the temperature of the material ranges between 10 and 15° C, mixes should only be produced with superior quality foamed bitumen (especially the half-life). Where such conditions are expected, the quality of the mix should be checked in the laboratory at the anticipated mixing temperature before commencing construction.

4.3.5 Bitumen stabilising agents

Penetration grade bitumen is used to produce both the foamed bitumen and bitumen emulsion that is used as a stabilising agent to manufacture BSMs. The types of bitumen and specific bitumen requirements are outlined below.

Bitumen Emulsion

Base bitumens with penetration values between 50 and 100 are generally selected for bitumen emulsion production, although softer and harder bitumen has been successfully used. The selection of the correct grade or category of bitumen emulsion for each application is essential, as outlined in the table below.

Slow set stable grade cationic bitumen emulsions are almost exclusively used worldwide for BSMs as they generally work well with dense graded materials, regardless of the parent rock, as well as with material with high fines contents. These bitumen emulsions have long workability times to ensure good dispersion and are formulated for mix stability. (Anionic stable grade emulsion are used in a few warm and dry climates.)

Breaking rate. There have been many recent developments in bitumen emulsion technology to improve stability without prolonging the break time. These emulsions are typically slower setting than the standard products, and should be used on projects where the treated layer can be cured for a period before opening to traffic. During the mix design phase, and on site before full-scale application begins, the breaking rate should be tested with representative samples of material, active filler and water, at realistic temperatures.

Compatibility of bitumen emulsion and the parent material. The selection of the bitumen emulsion type is influenced by the type of material that is to be treated. Certain material types are not suitable for treatment with anionic bitumen emulsions. These are the acidic rocks with silica contents above 65% and alkali contents below 35% and include quartzite, granite, rhyolite, sandstone, syenite and felsites. Treatment of such materials requires a cationic bitumen emulsion, as outlined in the table opposite.

Manufacturers normally recommend that undiluted bitumen emulsion is **heated** to between 50 and 60° C to prevent premature breaking due to the increase in pressure and shearing action while pumping and injecting through the spraybar on the recycler.

Classes of Bitumen Emulsion		
Bitumen Emulsion Type	**Anionic**	**Cationic**
Emulsifier type	Fatty acid or resin acid	Amine
Bitumen emulsion charge	Negative	Positive
pH	High (alkali)	Low (acid)
Grades	Stable mix (slow set) for recycling / stabilising	

Bitumen emulsion type / aggregate type compatibility			
Emulsion Type	**Aggregate (Rock) Type**	**Trends**	
		Breaking rate	**Adhesion**
Anionic	Acidic	Slow	Poor
Anionic	Alkaline	Medium	Good
Cationic	Acidic	Fast	Excellent
Cationic	Alkaline	Fast	Good

Foamed Bitumen

Bitumen grades with penetration values between 60 and 200 are generally selected for BSM-foam, although harder bitumen has been successfully used in the past without compromising the quality of the mix (harder bitumen is generally avoided due to poor quality foam, leading to poorer dispersion of the bitumen in the mix).

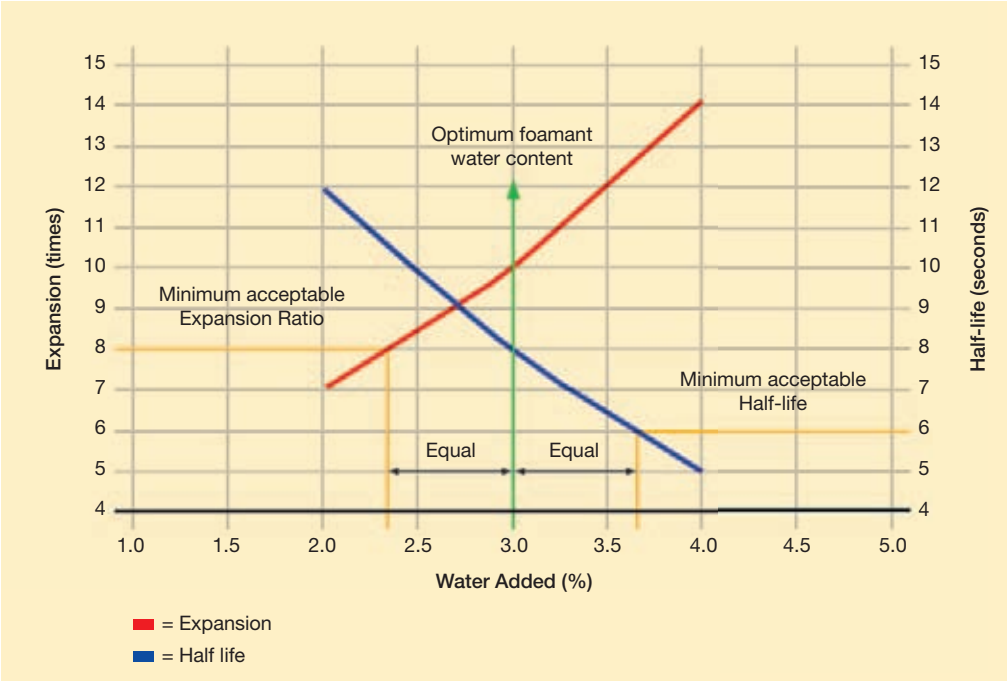
The penetration value alone does not qualify bitumen for use in a BSM-foam. The foaming characteristics of each bitumen type needs to be tested. Two properties form the basis of a bitumen's suitability for use, namely the Expansion Ratio (ER) and Half-life ($\tau_{1/2}$):

- The expansion ratio is a measure of the viscosity of the foam and determines how well the bitumen will disperse in the mix. It is calculated as the ratio of the maximum volume of foam relative to the original volume of bitumen.
- The half-life is a measure of the stability of the foam and provides an indication of the rate of collapse of the foam. It is calculated as the time taken in seconds for the foam to collapse to half of its maximum volume.

One of the dominant factors influencing foam characteristics is the amount of water that is injected into the expansion chamber to create the foam, the foamant water. Increasing the application rate of water creates greater expansion (higher ER) but leads to more rapid subsidence or decay, a shorter half-life ($\tau_{1/2}$), as illustrated in the graph opposite.

The water application rate and bitumen temperature are the most important factors influencing foam quality. A higher bitumen temperature usually creates better foam. A sensitivity analysis in the laboratory is recommended to identify a target bitumen temperature for foaming. (As with HMA production, temperature limits should be implemented to prevent damage to the bitumen.)

The variability of the foam characteristics measured in a laboratory, both in terms of repeatability and reproducibility, are significant. To obtain an acceptable level of statistical reliability, at least three tests are recommended for each set of conditions. In addition, potential variability in the bitumen composition from the same source necessitates checking the foam characteristics of each tanker load of bitumen.



Determining the Optimum Foamant Water Content

Foamed Bitumen Characteristics (Minimum Limits)		
Aggregate Temperature	10 ° C to 15 ° C	Greater than 15 ° C
Expansion Ratio, ER (times)	10	8
Half-life, $\tau_{1/2}$ (seconds)	8	6

4.3.6 Active filler

For the purpose of this manual, the term active filler is used to define fillers that chemically alter the mix properties. The types of active filler used with BSMs are: cement (various types, but not rapid hardening cements), lime, fly ash and slagment but excludes natural fillers such as rock flour. In addition, the term "lime" always refers to hydrated lime in this manual.

The purpose of incorporating active filler in BSM is to:

- **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.

BSM-emulsion

- Control the **breaking time**.
- Improve the **workability** (in some cases).

BSM-foam

- To assist in **dispersing the bitumen** splinters.

Note:

- The maximum allowable cement addition for BSMs is 1%

Various types of active filler can be used, separately or in combination. The type selected will depend on availability, cost and efficacy with the actual component materials. Research has shown that it is almost impossible to predict which active filler will prove to be the most effective without experimentation during mix design. (Testing 100 mm diameter specimens for Indirect Tensile Strength is the most useful guide for active filler selection, as described in Appendix 1)

When cement is used, the application rate must be limited to a maximum of 1% by mass of dry material. When using hydrated lime, the application rate may be increased to 1.5% (or more) where the lime is required to modify plasticity. However, it should be noted that above these application rates, the increase in mix stiffness is compromised significantly by a loss in flexibility of the material and the benefit of the bitumen is hardly realized.

Where active fillers are applied, the time delay between mixing the active filler with the material and application of the foamed bitumen or bitumen emulsion should be reduced to a minimum (both in the laboratory and the field). The active filler reaction begins immediately upon contact with moist material, promoting adhesion between the fine particles. The longer the delay between premixing with active filler and applying the bitumen, the lower the percentage of fines available for dispersion of the bitumen in the BSM mix.

Where materials with unacceptably high PI values are encountered, they can be treated with hydrated lime to modify the plasticity, thereby rendering them acceptable for treatment with foamed bitumen or bitumen emulsion. Pre-treating with lime must allow for sufficient time for modification to take place before bitumen treatment (normally 4 hours is sufficient).

4.3.7 Water quality

The quality of the water used to create the foamed bitumen and to dilute a bitumen emulsion is important. The standard requirements of water quality for concrete and other road materials should be followed in this regard.

BSM-emulsion

The pH levels of the water must be checked, as must the compatibility of the bitumen emulsion and the water.

Note:

- When diluting bitumen emulsion, always add the water to the bitumen emulsion to avoid a premature break.

BSM-foam

Although acceptable foam may be achieved using water containing impurities, such practice should be avoided. Impurities often lead to scales forming on the walls of the feed pipes and these eventually dislodge and block the water injection jets, preventing the bitumen from foaming.

4.3.8 Mix design procedure

The mix design procedure involves several steps and one or more series of tests, depending on the importance of the road and the magnitude of design traffic. The mix design procedure always starts by testing samples of the material that will be stabilised (standard laboratory tests) to determine whether they are suitable for treating with bitumen and, if not, the type of pre-treatment or blending required to make them suitable.

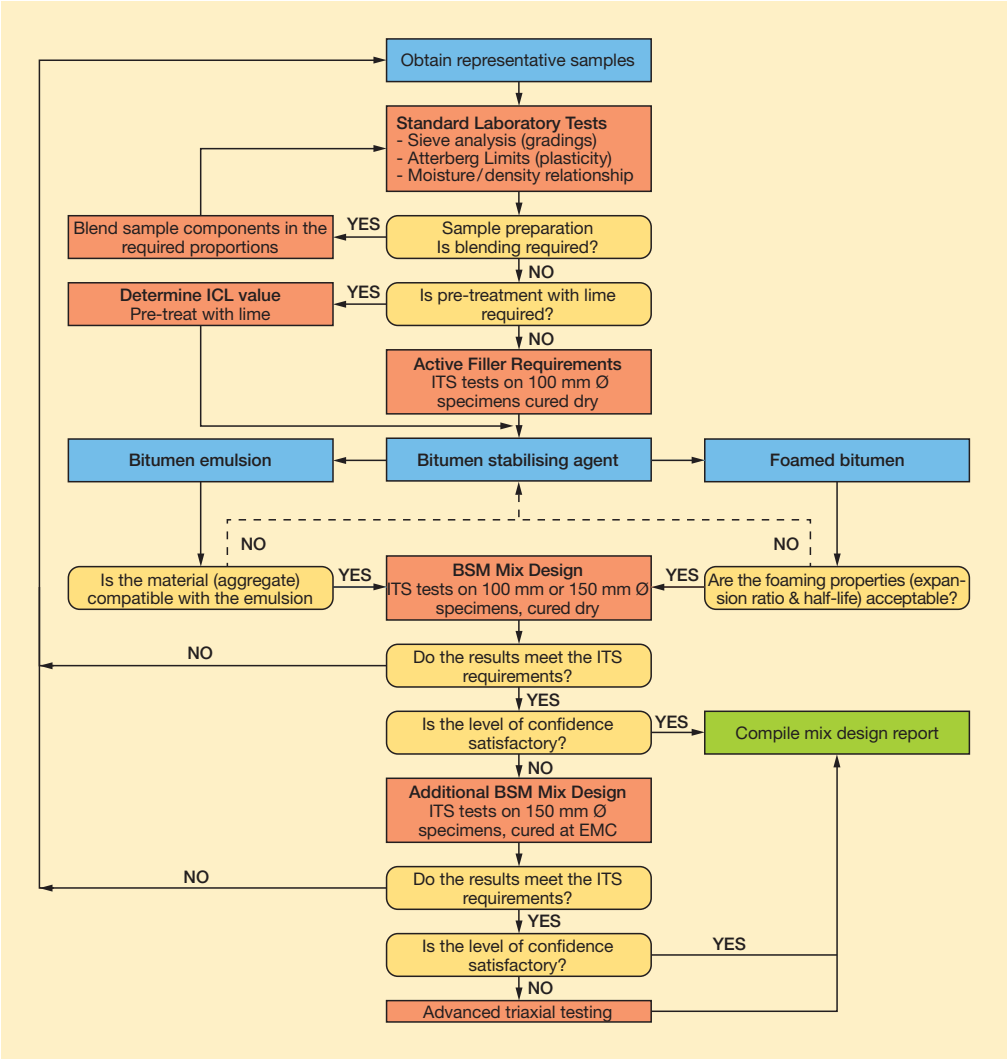
Once the suitability of the material has been proven, the actual mix design procedure commences with a series of preliminary tests to determine the need for the addition of an active filler and whether the material exhibits a preference for either cement or hydrated lime. All further testing is then undertaken in accordance with these results (e.g. if cement is shown to be the preferred active filler then all mixes will include a nominal 1% cement).

The mix design is then undertaken by mixing a series of samples, each with a different amount of bitumen added. The mixed materials are then used to manufacture several 100 mm or 150 mm diameter specimens which are cured dry and tested to determine their respective indirect tensile strengths (ITS) under both unsoaked and soaked conditions. These results provide an indication of the optimum amount of bitumen that should be added to achieve a specific level of strength.

Additional mix designs using 150 mm diameter specimens cured under a different regime may then be carried out if a higher level of confidence is required. The interval of added bitumen for such mixes is normally less than that applied for the initial mix design, thereby allowing a more precise assessment of the optimum added bitumen content to be made. Other sensitivity tests can also be undertaken using the same procedure for specimen manufacture and curing (e.g. the effect of reducing the amount of active filler to 0.75%).

Appendix 1 includes the detailed laboratory procedures for carrying out a BSM mix design using either bitumen emulsion or foamed bitumen as the stabilising agent.

Where even higher levels of confidence are required (e.g. for the design of heavy-duty or strategic pavements, such as runways at major airports), a series of triaxial tests may be carried out on large specimens (150 mm diameter x 300 mm high) manufactured at the optimum added bitumen content. The shear properties determined from such a testing programme feed directly into pavement design models. These tests are considered “specialised” and are not covered in this manual; they are adequately described in the Technical Guideline TG2 (2009).



The complete mix design procedure is explained in the flow chart above.

4.3.9 Classification of BSMs

The results obtained from the various mix design strength tests are used to judge the anticipated performance of the BSM. Several different classification systems have been developed for these materials, as well as methods to relate the BSM to a standard material (e.g. the equivalency method adopted in California). Structural layer coefficients derived from these strength tests have also been for developed for input into the AASHTO Structural Number design method (see Section 4.3.12).

To date, the most comprehensive research programme on BSMs was carried out in South Africa between 2004 and 2009. Involving both laboratory testing (including monotonic and repeated-load triaxial tests on large specimens) together with a detailed long-term pavement performance (LTPP) exercise of 23 different pavements (including several HVS trial sections), this work culminated in the publication of Asphalt Academy's Technical Guidelines TG2 (2009). The design methodology emanating from this work uses the results from strength tests (ITS and triaxial) to classify the BSM in to one of three classes (BSM1 being a high quality material with excellent shear properties whilst a BSM3 has poor shear properties, suitable only for lightly trafficked roads). Such classification feeds directly into the empirical Pavement Number pavement design method (described in Section 2.6.4).

To simplify matters and to focus on the more practical aspects of pavement design, two classes of BSMs have been adopted:

Class 1 BSM:
Materials with high shear strength. These materials are suitable for base layer construction on pavements with a structural capacity in excess of 3 million ESALs (where analytical modelling is recommended for pavement design).

Class 2 BSM:
Materials with moderate shear strength. These materials are suitable for base layer construction for pavements with a structural capacity less than 3 million ESALs (where the empirical methods (SN and PN methods) are adequate for pavement design).

The following table summarises the requirements for classifying a BSM into one of two classes.

Recommended BSM classes based on ITS test results			
	Class 1	**Class 2**	**Not suitable**
Parent material: Test result from Mix Design	RAP and GCS RAP / GCS blend RAP / GCS / gravel	Material blends Natural gravel Marginal materials	Poor gravels Plastic materials Soils
ITS_{DRY} Both 100 mm and 150 mm Ø specimens	**> 225 kPa**	125 to 225 kPa	< 125 kPa
ITS_{WET} & ITS_{SOAK} Both 100 mm and 150 mm Ø specimens	**> 100 kPa**	50 to 100 kPa	< 50 kPa
ITS_{EQUIL} 150 mm Ø specimens only	**> 175 kPa**	95 to 175 kPa	< 95 kPa
Implied shear properties			
Cohesion	> 250 kPa	> 50	< 50
Angle of internal friction	> 40°	> 25°	< 25°
Implied parent material properties			
California Bearing Ratio (CBR)	> 80%	>20%	< 20%
Plasticity Index (PI)	< 10	< 15	> 15

4.3.10 Working with BSMs

Safety aspects for BSM-foam
Bitumen temperatures need to be high (typically >160° C) for the water reaction to produce an acceptable foamed. At such high temperatures, bitumen must be treated with respect and adequate safety procedures established, similar to those adopted for hot mixed asphalt production. This is well known by asphalt manufacturers who work with hot bitumen on a daily basis, but the recycling contractor who undertakes a foamed bitumen project for the first time needs to ensure that his personnel receive appropriate training. The same safety rules as those documented for hot mix asphalt, are applicable to foamed bitumen.

Fluid Considerations
The role of moisture in the stabilised material is similar for BSM-emulsion and BSM-foam in many respects, but there are some differences. The entire fluid content in the mix (moisture and bitumen) needs to be considered. The role of fluid in the two types of BSM is explained in the table below.

Role of fluids in BSM		
Component	**BSM-emulsion**	**BSM-foam**
Bitumen	Contributes to fluids for compaction	Negligible contribution to fluids for compaction
Moisture in aggregate	Reduces absorption of bitumen emulsion water into aggregate	Separates and suspends fines making them available to bitumen during mixing
	Prevents premature breaking	Acts as carrier for bitumen splinters during mixing
	Extends curing time and reduces early strength	Reduces early strength
	Provides workability of BSM at ambient temperatures	
	Reduces friction angle and lubricates for compaction	
	Provides shelf-life for the mix	

BSM-emulsion

Changes in moisture content occur in two distinct phases, namely:

- **Breaking** is the separation of the bitumen from the water phase through flocculation and the coalescence of the bitumen droplets to produce films of bitumen on the individual particles of the material. The rate at which the bitumen droplets separate from the water phase is referred to as the breaking time (Also known as the setting or settling time.)

The breaking process with anionic bitumen emulsions is a mechanical process (evaporation), whereas cationic bitumen emulsions produce a chemical break. For dense mixtures, more time is needed to allow for mixing and placement and slower breaking times are required. As the bitumen emulsion

breaks, the colour changes from dirty brown to black. Although this can be observed with the naked eye, it is recommended that a magnifying glass is used.

- **Curing** is the displacement of water and the resultant increase in stiffness and tensile strength of the BSM. This is important as a mix needs to acquire sufficient stiffness and cohesion between particles before carrying traffic.

Some of the factors which influence the breaking and curing process (known as the **"setting" process** of bitumen emulsions), include:

- **Rate of absorption** of water by the material. Rough-textured and porous materials reduce the breaking and setting time by absorbing water contained in the bitumen emulsion.
- **Moisture content** of the mix prior to mixing influences breaking time.
- **Moisture content** of the mix after compaction influences curing rate.
- **Grading** of the material and voids content of the mix.
- **Type, grade** and **quantity** of the bitumen emulsion.
- **Mechanical forces** caused by compaction and traffic.
- **Mineral composition of the material.** The rate of cure may be affected by physico-chemical interactions between the bitumen emulsion and the surface of the individual material particles.
- **Intensity of electrical charge** on the material particles in relation to that of the bitumen emulsion.
- **Active filler** addition, the amount of cement or lime.
- **Temperature** of material and air. The higher the temperature, the quicker the bitumen emulsion breaks and cures.

BSM-foam

The moisture content of the material reduces due to evaporation and repulsion by the bitumen. This process is known as "curing".

As the moisture content reduces, the tensile strength and stiffness of the material increases. This is important because the completed layer of BSM-foam often needs to acquire sufficient stiffness and cohesion between the particles before carrying heavy loads.

Mixing Moisture. The moisture content that will provide the best BSM mix is termed the optimum mixing moisture content (OMMC). This is the moisture in the material plus, for BSM-emulsions, any additional moisture in the bitumen emulsion. OMMC varies with gradation of the material and, in particular, the size of the fraction smaller than 0.075 mm.

BSM-emulsion

A minimum of 1 to 2% moisture is required in the material prior to adding the bitumen emulsion.

The water and bitumen in the bitumen emulsion act as lubricants in BSM-emulsion mixes. The optimum moisture content (OMC) determined from modified AASHTO compaction should be used for the total mixing fluid content. This is explained in the equation below:

$$OFC = OMC_{MOD\ U} = FMC + EWC + RBC$$

Where

OFC	=	optimum fluids content (%)
OMC_{MOD-U}	=	optimum moisture content using Mod. AASHTO compaction on untreated material (%)
FMC	=	moisture content of aggregate (%)
EWC	=	bitumen emulsion water content including water used for dilution as percentage of dry aggregate (%)
RBC	=	residual bitumen content as percentage of dry aggregate (%)

BSM-foam

Fluffpoint moisture (the moisture content that results in the maximum bulk volume of loose mineral aggregate during agitation) should be used as a target. This value ranges from 70 to 90% of the optimum moisture content (OMC) (determined from modified AASHTO compaction).

The targeted mixing moisture content when adding foamed bitumen is 75% of OMC.

Bitumen Supply

When coupling a new tanker load of bitumen stabilising agent to the recycler, some basic checks should be conducted to ensure that the bitumen is acceptable for use. Regardless of whether the tanker contains bitumen emulsion or Penetration-grade bitumen, the temperature of the contents should be checked using a calibrated thermometer (gauges fitted to tankers are notoriously unreliable).

BSM-emulsion

- Pressure gauge on the spraybar should be checked to ensure that the bitumen emulsion is flowing freely and not creating excessive backpressure due to a blockage.

BSM-foam

- the foaming characteristics of each tanker load of bitumen must be checked using the test nozzle on the recycler.
- The quality of foam is a function of bitumen operating pressure. The higher the pressure, the more the stream of bitumen will tend to "atomise" as it passes through the jet into the expansion chamber, thereby promoting uniformity of foam. If the bitumen were to enter the expansion chamber as a stream (as it does under low pressures) the water would impact on only one side of the stream, creating foam, but the other side would remain as unfoamed hot bitumen. It is therefore imperative to set the recycler up and to operate at an advance speed that will maintain a minimum operating pressure above 3 bars.

Bitumen delivered to site by tankers that are fitted with fire-heated flues is sometimes contaminated with small pieces of carbon that form on the sides of the flues whilst heating. Draining the last few tons from the tanker tends to draw these unwanted particles into the recycler's system and can cause blockages. This problem is easily resolved by ensuring the effectiveness of the filter in the delivery line. Any unusual increase in pressure will indicate that the filter requires cleaning, a procedure that should anyway be undertaken on a regular basis (e.g. at the end of every shift).

Applying Active Filler
See Section 4.2.6, Uniformity of application.

Mixing
Mixing the BSM involves the addition of all additives to the parent material and providing sufficient agitation energy within a relatively short time period to achieve a homogenous blend. This process is adequately described in Chapter 3 for both in-place and in-plant treatment.

Compaction
Special attention needs to be paid to compaction, as it improves particle contacts and reduces voids. The density achieved is critical to the ultimate performance of the mix.

BSM-emulsion

The inclusion of bitumen emulsion typically improves compactibility of the mix.

BSM-foam

Compaction promotes adhesion of the bitumen mastic to the coarse particles.

Vibrating padfoot rollers are normally used to compact layers of BSM in the field. Such rollers impart very high energy and therefore, in order to achieve a specific level of density, require a lower effective optimum fluids content relative to that determined in the laboratory. For this reason it is normally possible to compact in the field at the (relatively low) mixing moisture content. (Roller selection, in terms of type and capacity, is described in the Wirtgen Cold Recycling Application manual.)

As with cement stabilised materials, compaction of BSM's should always aim to achieve the maximum density possible under the conditions prevailing on

site (the so-called “refusal density”). A minimum density is usually specified as a percentage of the modified AASHTO density, normally between 98% and 102% for bitumen stabilised bases. A density gradient is sometimes permitted by specifying an “average” density. This means that the density at the top of the layer may be higher than at the bottom. Where specified, it is normal also to include a maximum deviation of 2% for the density measured in the lowest one-third thickness of the layer.

Hence, if the average density specified is 100%, then the density at the bottom of the layer must be more than 98%.

Curing

Curing of BSMs is the process where the mixed and compacted layer looses moisture through evaporation, particle charge repulsion and pore-pressure induced flow paths.

The reduction in moisture content leads to an increase in the tensile and compressive strength, as well as stiffness of the material.

The rate of moisture loss from newly constructed BSM layers plays a significant role in determining the performance of the layer. It is in the early period of repeated loading that the majority of the permanent deformation takes place in BSM layers. Where a new BSM layer is to be trafficked immediately after construction, it is important to keep the compaction moisture content to a minimum. The lower the degree of saturation (moisture content) of the BSM, the greater the resistance to permanent deformation.

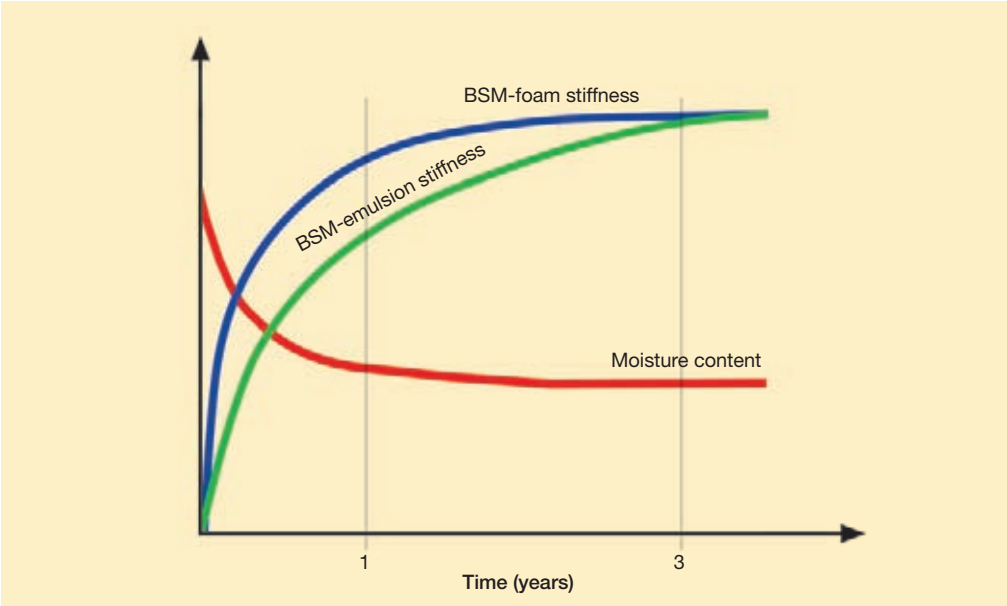
BSM-emulsion

Chemistry plays a significant role in the curing of BSM-emulsions. Water is an intrinsic component of bitumen emulsions. Breaking of the bitumen emulsion needs to take place before curing as a result of migration and moisture loss by evaporation.

BSM-emulsion usually requires longer curing times than BSM-foam because of the higher moisture contents.

BSM-foam

Curing takes place as a result of migration of water during compaction and continues with repulsion of moisture by the bitumen and loss by evaporation.



Concept of Curing and Influence on Mix Stiffness

Curing rates are dependent on the type of treatment (BSM-emulsion or BSM-foam). Although the use of active filler also has an impact on curing, its inclusion in a BSM does not justify extensions in the curing time as cementation is not one of the desired properties of these materials.

Although a BSM should have sufficient stiffness and strength to withstand moderate levels of early traffic, the layer will continue to gain strength over several years (i.e. improve its resistance to permanent deformation).

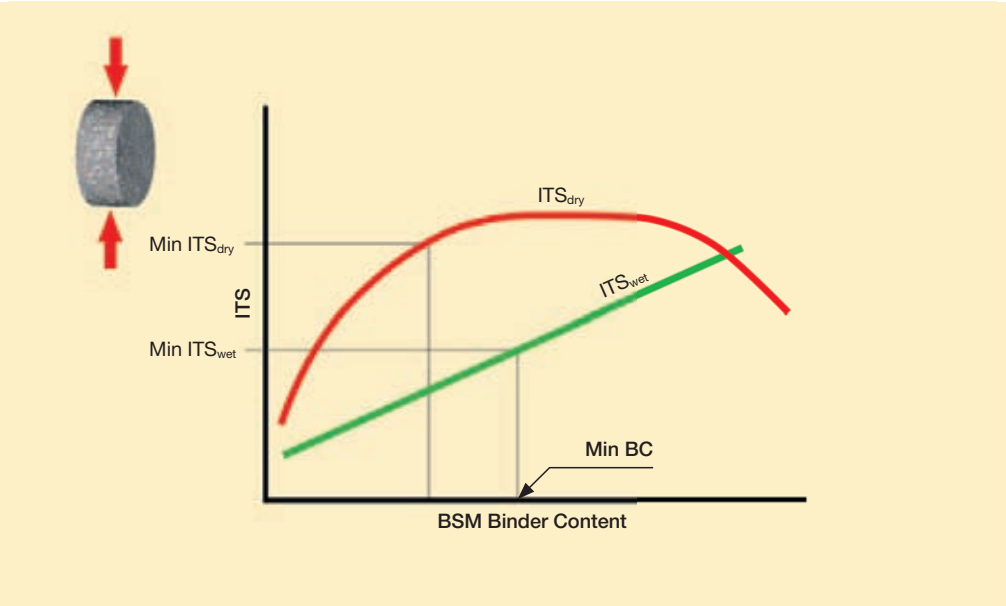
4.3.11 Mechanical tests

Indirect Tensile Strength (ITS)
The ITS test is an indirect measure of the tensile strength and reflects the flexibility and flexural characteristics of the BSM. Although this test does not produce highly repeatable results, it is the most economical method available for investigating the effectiveness of the bitumen in a BSM. In addition, a background of historical data is available.

100 mm or 150 mm diameter specimens are used to indicate the optimum bitumen content, the need for an active filler and, if active filler is required, at what content.

The specimens are cured for 72 hours at 40° C to reach a constant mass (see Appendix 1). ITS_{DRY} values are determined from these specimens. The results obtained after soaking specimens for 24 hours at 25° C are termed ITS_{WET}. The ratio of ITS_{WET} and ITS_{DRY}, expressed as a percentage, is the Tensile Strength Retained (TSR). An example of the typical trends of ITS test results is shown in the figure below, indicating how the actual results are analysed against the limits for a specific BSM classification.

150 mm diameter and 95 mm high specimens can also be cured to simulate the field moisture conditions (see Appendix 1). The ITS results from



ITS interpretation for bitumen type and content (Level 1)

specimens tested after this curing are termed ITS_{EQUIL}. The results after soaking for 24 hours at 25° C are termed ITS_{SOAK}.

The limits for interpreting the various ITS tests are shown in the table in section 4.3.9.

Note: The TSR value is useful for identifying problem materials. If the TSR is less than 50%, it is recommended that active filler be included in the mix. If such treatment does not increase the ITS_{WET} value with a maximum application of 1% lime or cement, then the nature of the material being stabilised should be investigated:

- Granular material. A combination of TSR < 50% and ITS_{DRY} > 400 kPa suggests contamination (normally attributed to clay or deleterious materials). In this situation, it is suggested that the material is pre-treated with hydrated lime and the tests repeated.
- 100% RAP material. A combination of TSR < 50% and ITS_{DRY} > 500 kPa indicates that the treated material is partly stabilised and partly asphaltic (continuously bound). In this situation, the material probably requires blending with crusher dust to ensure that the stabilisation process dominates the mix (see Chapter 6).

In addition, tests using 150 mm diameter specimens should be undertaken to verify the relevant ITS_{EQUIL} values for BSM classification.

4.3.12 Pavement design approaches for BSMs

There are three accepted structural design methods for BSMs and these are discussed below.

Structural Number design method
The AASHTO (1993) design method using Structural Numbers was described in Chapter 2, Section 2.6.3. This design method is popular worldwide and, due largely to its simplicity and user-friendliness, it is used extensively for the design of all types of pavements.

As with all design routines, the reliability of the results obtained using this method is dependent on the accuracy of the input parameters. With Structural Numbers, the primary input that affects the result is the selection of an appropriate structural layer coefficient for each layer of material. These coefficients reflect the engineering properties of the in situ material in the layer.

The table below includes structural layer coefficients for layers that are constructed using conventional materials.

Typical structural layer coefficients (from AASHTO)		
Material type	Characteristic	Structural layer coefficient (per inch)
Asphalt surfacing	Elastic modulus 2,500 to > 10,000 MPa	0.30 to 0.44
Asphalt base	Continiously graded (6% voids)	0.20 to 0.38
Graded crushed stone	CBR > 80%	0.14
Natural gravel, type 1	CBR 65 to 80%	0.12
Natural gravel, type 2	CBR 40 to 65%	0.10
Soil, type 1	CBR 15 to 40%	0.08
Soil, type 2	CBR 7 to 15%	0.06
Cohesionless sand	PI = 0	0.04 to 0.05
Cement-treated crushed stone	1.0 < UCS < 3.0 MPa	0.17
Cement-treated gravel	UCS < 1.0 MPa	0.12

Note:

- The AASHTO design method originated from America. Structural layer coefficients are normally quoted in terms of “per inch” of layer thickness. The various values assigned to different materials and the magnitude of the coefficient in “per inch” terms is well known, worldwide. It is therefore preferable for countries using the metric system to retain the coefficients in “per inch” terms and convert the layer thickness from centimetres to inches (1 inch = 2.54 cm).

Since bitumen stabilisation is a relatively new technology, no relevant structural layer coefficients are available for these materials in the AASHTO design method. The following table shows how the structural layer coefficient of a BSM can be estimated from ITS test results. In addition, this chart can be used as an indicator of the shear properties of a BSM as well as gaining a rough estimate of the anticipated structural layer coefficient based on the CBR value of the untreated material.

Suggested structural layer coefficients for bitumen stabilised material (BSM)

Structural layer coefficient (per inch)	0.18	0.23			0.28	max 0.35
Indirect tensile strength (ITS) after stabilisation						
100/150 mm Ø specimens						
ITS_{DRY} (kPa)	125	175			225	
ITS_{WET} & ITS_{SOAK} (kPa)	50	75			100	
150 mm Ø specimens						
ITS_{EQUIL} (kPa)	95	135			175	
Indicated shear properties						
Cohesion (kPa)	50	100			250	
Angle of Friction (°)	25	30			40	
Material CBR value before stabilisation (at field density)						
(Materials with CBR < 20% not recommended)	20	40			80	
Anticipated application rate of bitumen for stabilisation (% by mass)						
	2.5–4.0		2.0–3.0		1.8–2.3	

Pavement Number design method
The PN design method was developed as part of TG2 (2009) guidelines for the design of BSMs.

The PN design method is a knowledge-based (or heuristic) approach and can be used for design with a reliability of 90% to 95%. As was described in Chapter 2, Section 2.6.4, the PN method is similar to the well known Structural Number method (AASHTO 1993) and has the following advantages:

- Data from numerous in-service pavements were used to develop the method. The type and detail of the data suggests the use of a relatively simple method and precludes the use of a Mechanistic-Empirical design method.
- The method gives a good fit to the available field data.
- The method is robust, and cannot easily be manipulated to produce inappropriate designs

The PN design method has been verified for design traffic of up to 30 million ESALs based on long-term pavement performance (LTPP) data from pavements with BSM base layers. The pavements being monitored continue to be trafficked and the maximum design traffic will increase with time. Pavements modelled using this method require a minimum subgrade CBR value of 3%. In addition, the LTPP exercise included roads with only an 80 kN legal axle limit. The method has therefore not been calibrated for higher axle loads (e.g. the 13 ton legal limit in Greece).

The PN design method uses a materials classification system for each layer in the pavement structure and the design approach is, in effect, an intelligent Structural Number method. The Pavement Number is the sum of the products of layer thicknesses and their respective Effective Long Term Stiffness (ELTS) values, as described in Section 2.6.4. Some rules relating to the pavement layers are:

- The load spreading potential of an individual layer is a product of its thickness and its ELTS value under loading.
- The ELTS value of a layer depends on the material type and class and on its location in the pavement structure.
- Fine-grained subgrade materials act in a stress-softening manner. For these materials, the ELTS is determined mainly by the material quality and by the climatic region. Owing to the stress softening behaviour, subgrade materials will generally soften with decreased cover thickness.
- Coarse-grained, unbound materials act in a stress-stiffening manner. For these materials, the ELTS is determined mainly by the material quality and the relative stiffness of the supporting layer. The ELTS of these materials will increase with increasing support stiffness, by means of the modular ratio limit, up to a maximum stiffness which is determined by the material quality.
- BSMs are assumed to act in a similar way to coarse granular materials, but with a higher cohesive strength. The cohesive strength is subject to breakdown during loading and thus some softening over time can occur. The rate of softening is mainly determined by the stiffness of the support, which determines the degree of shear in the layer. However, owing to the higher cohesive strength in BSMs, these layers are less sensitive to the support stiffness than unbound granular materials and can therefore sustain higher modular ratio limits. (If, however, the cement content of a BSM mix exceeds 1 %, the material is assumed to behave as a cemented material.)

The above rules-of-thumb introduce several concepts such as the ELTS, modular ratio limit, maximum stiffness and stress-stiffening (or softening) behaviour. These need to be understood in order to use the method. (Refer TG2 (2009) for more details.)

Deviator Stress Ratio Design Method

Pavement rehabilitation projects that are designed to carry > 30 million ESALs design traffic, require more advanced structural analysis than the Pavement Number Design Method alone. In such cases the Deviator Stress Ratio Method should be adopted. The Deviator Stress Ratio Method is based on the premise, proven through research, that the rate of permanent deformation in a BSM layer is a function of the ratio of the applied deviator stress relative to the maximum deviator stress of the BSM (at failure).

This is shown in the equation below.

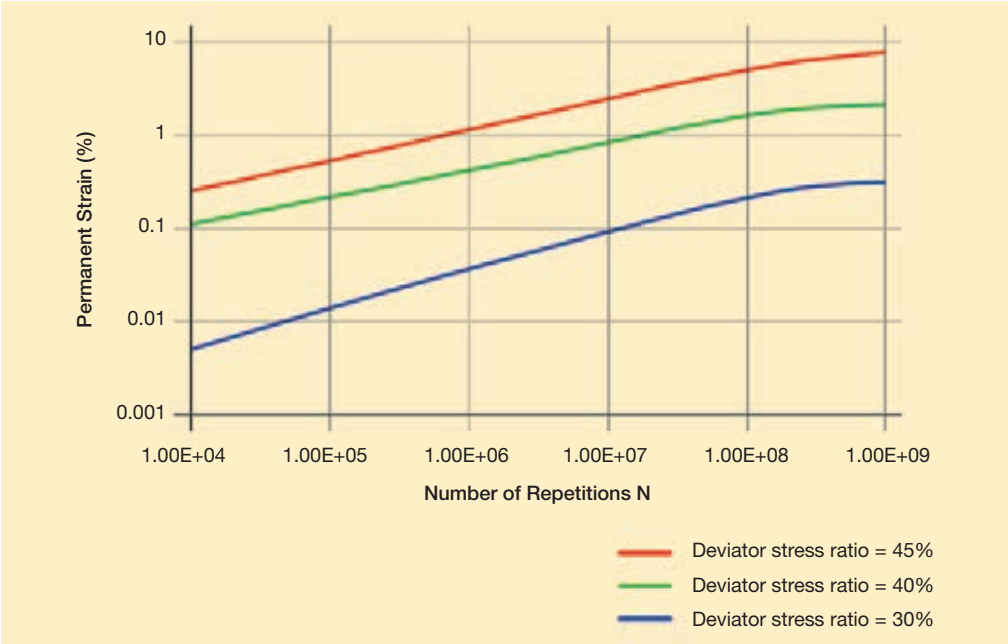
$$\text{Deviator Stress Ratio} = \frac{\sigma_1 - \sigma_3}{\sigma_{1.f} - \sigma_3}$$

Where σ_1 = major principal stress applied to BSM layer from M-E analysis
σ_3 = minor principal stress applied to BSM layer from M-E analysis
$\sigma_{1,f}$ = major principal stress at failure from triaxial testing of BSM

In order to carry out a Mechanistic-Empirical (M-E) analysis of the pavement structure, a Resilient Modulus value for the BSM is required. Based on triaxial testing and pavement monitoring, the following values are considered to be reasonable for this purpose.

Resilient Moduli ranges for BSMs		
Material type	**Bitumen Content of BSM (%)**	**Resilient Modulus MR(MPa)**
100% RAP	1.6 to 2.0	1,000 to 2,000
RAP/crushed stone (50:50 blend)	1.8 to 2.5	800 to 1,500
Graded crushed stone	2.0 to 3.0	600 to 1,200
Natural gravel (PI < 10, CBR>45)	2.2 to 3.5	400 to 800
Natural gravel (PI < 10, CBR>25)	2.5 to 4.0	300 to 600

The preferred way to approach advanced pavement design is to select a deviator stress ratio limit depending on the importance of the road being analysed and the design traffic. The following figure illustrates how the pavement life decreases as the deviator stress ratio increases.



Recommended deviator stress ratio limits, based on the relationships outlined above and taking account of typical recycling depths, are provided in the table below.

Deviator Stress Ratio Limit for 10 mm maximum deformation in BSM layer	
Design Reliability 95% (Highways with heavy traffic)	Design Reliability 80 – 90% (Arterials with moderate traffic)
35%	40%

It is recognised that BSMs have stress-dependent properties. Where dynamic triaxial testing has provided the data to develop relationships between Resilient Modulus and applied stress for a particular BSM, then iterative multi-layer stress-dependent analyses can be followed. This requires the BSM layer to be subdivided into 25 mm to 50 mm sub-layers and each one is then analysed for convergence of Resilient Modulus. Such analyses will yield more realistic stress distributions in the BSM layer that are, in turn, used to determine the deviator stress ratio. The calculated deviator stress ratio can be used to establish whether or not the BSM layer is the critical layer in the pavement (i.e. the layer that reaches a terminal condition first and precipitates failure in the other layers).

The deviator stress ratio approach provides reasonable designs with sufficient conservatism. It does, however, require sound pavement engineering knowledge and should only be used by experienced practitioners.

4.4 Summary: Advantages and disadvantages of cement and bitumen stabilising agents

Cement Stabilisation	
Advantages	**Disadvantages**
• Availability. Cement can be obtained worldwide, always in bags, often in bulk. • Cost. Relative to bitumen, cement is inexpensive. • Ease of application. Cement can always be spread by hand in the absence of bulk spreaders or slurry units. • Acceptance. Cement is well-known in the construction industry. Standard test methods and specifications are usually available. • Significant improvement of compressive strength and durability properties of most materials.	• Shrinkage cracking is unavoidable. However, it can be minimised. • Increases rigidity in flexible pavements. • Requires proper curing and protection from early trafficking, particularly heavy slow-moving vehicles.

Stabilising with Bitumen (Emulsion and Foamed)	
Advantages	**Disadvantages**
• Flexibility. Stabilising with bitumen creates a visco-elasto-plastic type of material with improved shear properties, (cohesion and resistance to deformation). • Ease of application. A bulk tanker is coupled to the recycler and the bitumen is injected through a spraybar (special type of spraybar for foamed bitumen). • Acceptance. Bitumen emulsions are relatively well-known in the construction industry. Standard test methods and specifications are available. • Rate of gain of strength. Material can be trafficked soon after placing and compaction, especially with BSM-foam • Durability. BSMs tend to lock-up the finer particles by encasulating them in bitumen. This prevents them from reacting to water and from any pumping potential.	• Cost. Bitumen is relatively expensive. • Bitumen emulsions are not normally manufactured on site. The manufacturing process requires strict quality control. Emulsifiers are expensive. Transport costs inflated by hauling the water component, not only bitumen. (Foamed bitumen uses standard penetration-grade bitumen. There are no additional manufacturing costs and is therefore less expensive than bitumen emulsion.) • Foamed bitumen demands that the bitumen is hot, usually above 160° C. This often requires special heating facilities and additional safety precautions. • BSM-foam requires strict adherence to grading requirements, especially the fraction < 0.075 mm. (BSM-emulsion is more forgiving in this respect.) • Where the moisture content of the material in the existing pavement is close to OMC, saturation often occurs when emulsion is added. • Curing can take a long time for BSM-emulsion. Strength development is dictated by moisture loss. • Availability. The required formulation for an emulsion that is appropriate for a specific recycling application may not always be available.

5 Recycling Solutions

All pavement rehabilitation solutions are project specific. As was described in Chapter 2, each construction site is different; the depth of recycling and type of stabilisation that is appropriate for a specific road is dictated by:

- the anticipated traffic over the design period (the structural capacity requirement),
- the composition of the existing pavement structure,
- the materials in the various layers, particularly those at the top of the pavement,
- the number of layers overlying the subgrade (the total cover thickness), and
- the strength of the underlying subgrade.

This chapter provides guidelines to assist design engineers visualising the type of pavement structures that can be achieved by recycling. Typical solutions for different traffic categories are illustrated, ranging from low volume roads (< 1 million ESALs) to heavily trafficked highways (> 100 million ESALs), each with a selection of realistic conditions in the existing pavement that are in need of rehabilitation.

This is followed by an example showing different solutions for rehabilitating a specific road, two of which require the existing pavement to be recycled. This exercise includes analyses of the costs involved in the initial rehabilitation exercise, maintenance requirements during the service life and the cost of rehabilitating the pavement at the end of the service life (whole-of-life cost analysis). Also included in this exercise is a section on the energy consumed by the various construction and maintenance activities during the entire service life. Combining the whole-of-life costs with energy consumed is useful for decision making.

5.1 Guidelines for recycling different pavements

The guidelines presented in this section are focused on recycling. Six different traffic categories are shown:

- light pavements with a structural capacity of approximately 300,000 ESLAs that would normally be applicable to rural access roads;
- low volume roads with a structural capacity of 1 million ESALs, typical for farm-to-market roads;
- secondary roads with a structural capacity of 3 million ESALs;
- main rural roads with a structural capacity of 10 million ESALs;
- interurban highways with a structural capacity of 30 million ESALs; and
- major multi-lane highways with a structural capacity of 100 million ESALs.

Three different recycling applications are considered for each category:

- recycling the existing pavement without the addition of stabilising agents (mechanical modification or reworking),
- recycling with a cementitious stabilising agent (a minimum thickness of 150 mm has been adopted for such layers), and
- recycling with a bitumen stabilising agent (the minimum thickness for such layers when recycled in-place is 100 mm).

For traffic categories below 30 million ESALs, three different existing pavement scenarios are considered, essentially dictated by subgrade strength, depth of cover and the quality of material in the existing layers. The three different subgrade conditions assumed are:

- "Good" with an in situ CBR value between 10% and 25%,
- "Fair" with an in situ CBR value between 3% and 10%. And
- "Poor" with a CBR value of 3% or less.

Each scenario has been carefully selected to be relevant for the type of pavement that is to be rehabilitated and/or upgraded. The same realistic approach has been adopted for traffic categories in excess of 30 million ESALs; existing pavements for such heavy traffic are likely to be reasonably substantial with sufficient cover.

5.1.1 Lightly trafficked roads (structural capacity: 0.3 million ESALs)

Existing roads falling into this category are normally gravel roads or roads sealed with a light surface treatment. As a minimum, the existing road will include a layer of suitable gravel wearing course material with a CBR in excess of 25%. (Should this not be the case, then the road needs to be constructed; it cannot simply be recycled because there is no structural layer to recycle.)

Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these lightly trafficked pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 1 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 125 kPa, ITS_{WET} > 50 kPa

	Existing Pavement			Recycle /
		Thick mm	CBR %	
Good support				2nd: Import 125 mm (new layer) 1st: Rework 100 mm
		150	30	
		∞	10	
Fair support				2nd: Import 150 mm (new layer) 1st: Rework 100 mm
		150	30	
		∞	7	
Poor support				2nd: Import 150 mm (new layer) 1st: Import 100 mm Recycle 125 mm
		150	30	
		∞	3	

- Chip seal
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel / soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm

Mechanical modify			Recycle with cement or lime			Recycle with bitumen		
	Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %
	125	50	Recycle 150 mm with cement			Recycle 100 mm with bitumen		
	100	30		150	CTB		100	BSM
	50						50	30
	∞	10		∞	10		∞	10
	150	50	Import 125 mm Recycle 150 mm with cement			Import 50 mm Recycle 100 mm with bitumen		
	100	30		150	CTB		100	BSM
	50			125	30		100	30
	∞	7		∞	7		∞	7
	150	50	Import 175 mm Recycle 200 mm with cement			Import 100 mm Recycle 125 mm with bitumen		
	125	30		200	CTB		125	BSM
	125			125	30		125	30
	∞	3		∞	3		∞	3

Chip seals (normally single seals) are an appropriate surface treatment for these pavements

5.1.2 Low volume roads (structural capacity: 1 million ESALs)

Existing low volume roads will normally include a gravel wearing course with a surface treatment (chip seal) overlying a relatively light pavement structure constructed from natural gravels.

Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these low volume pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 1 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 175 kPa, ITS_{WET} > 75 kPa

- Chip seal
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel/soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm

	Existing Pavement		Recycle
	Thick mm	CBR %	
Good support			Import 75 mm Recycle 125 mm
	150	30	
	150	10	
	∞	10	
Fair support			Import 100 mm Recycle 150 mm
	150	30	
	150	10	
	∞	7	
Poor support			2nd: Import 150 mm (new layer) 1st: Rework 150 mm
	150	30	
	150	10	
	∞	3	

	Existing Pavement		Two-part
			Recycle in
Poor support	Thick mm	CBR %	Step 1
	150	30	Recycle 200 mm with lime
	150	10	
	∞	3	

/Mechanical modify		Recycle with cement or lime			Recycle with bitumen		
	Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %
	125	50						
	100	30	Recycle 150 mm with cement	150	CTB	Recycle 100 mm with bitumen	100	BSM
	150	10		150	10		150	10
	∞	10		∞	10		∞	10
	150	50						
	100	30	Recycle 150 mm with cement	150	CTB	Recycle 100 mm with bitumen	100	BSM
	150	10		150	10		150	10
	∞	7		∞	7		∞	7
	150	50						
	150	30	Recycle 250 mm with cement	250	CTB	Recycle 150 mm with bitumen	150	BSM
	150	10		50	10		150	10
	∞	3		∞	3		∞	3

recycling: Lime to modify plasticity and bitumen for strength						
place with lime			Re-recycle with bitumen			
	Thick mm	CBR %	Step 2	Thick mm	CBR %	
	200	CTB	Recycle 100 mm with bitumen	100	BSM	
				100	CTB	
	100	10		100	10	
	∞	3		∞	3	

Chip seals (normally double seals) are an appropriate surface treatment for these pavements

5.1.3 Secondary rural roads (structural capacity: 3 million ESALs)

Existing roads falling into this category can be expected to have a formal pavement structure with a light surfacing (either a thin asphalt layer or competent chip seal). These pavements would invariable have been constructed using a reasonably good quality material in the base layer, either a crushed stone or a natural gravel with a CBR value in excess of 80%.

Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these secondary road pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 2 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 225 kPa, ITS_{WET} > 100 kPa, ITS_{EQUIL} > 175 kPa

- Chip seal
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel/soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm

	Existing Pavement		Recycle
	Thick mm	CBR %	
Good support			Import 100 mm Recycle 125 mm
	150	80	
	150	50	
	∞	20	
Fair support			2nd: Import 150 mm (new layer) 1st: Rework 100 mm
	150	80	
	150	50	
	∞	7	
Poor support			2nd: Import 150 mm (new layer) 1st: Import 50 mm Recycle 200 mm
	150	80	
	150	50	
	150	10	
	∞	3	

/ Mechanical modify		Recycle with cement or lime		Recycle with bitumen	
Thick mm	CBR %	Thick mm	CBR %	Thick mm	CBR %
125	100	Recycle 250 mm with cement		Recycle 125 mm with bitumen	
125	80	250	CTB	125	BSM
150	50			150	50
∞	20	∞	20	∞	20
150	100	Import 75 mm Recycle 275 mm with cement		Recycle 150 mm with bitumen	
100	80	275	CTB	150	BSM
50					
150	50	100	50	150	50
∞	7	∞	7	∞	7
150	100	Import 100 mm Recycle 300 mm with cement		Recycle 200 mm with bitumen	
200	80	300	CTB	200	BSM
150	50	100	50	100	50
150	10	150	10	150	10
∞	3	∞	3	∞	3

A competent chip seal (Cape Seal or triple seal) is an appropriate surface treatment for these pavements although the application of a thin asphalt (30 mm) is often preferred, especially in wet and / or cold regions

5.1.4 Main rural roads (structural capacity: 10 million ESALs)

Existing roads falling into this category would normally have been constructed with a reasonable pavement structure, surfaced with either an thin asphalt surfacing or multiple chip seals. These pavements would invariable have been constructed using a crushed stone or good quality natural gravel (CBR > 80%) but may include a layer of cement stabilised material.

Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these main road pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 2 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 225 kPa, ITS_{WET} > 100 kPa, ITS_{EQUIL} > 175 kPa, cohesion > 250 kPa, angle of friction > 40°

- Asphalt
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel/soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm

	Existing Pavement			Recycle
		Thick mm	CBR %	
Good support		40	AC	2nd: Import 125 mm (new layer) 1st: Rework 125 mm
		150	80	
		150	50	
		∞	20	
Fair support		40	AC	2nd: Import 150 mm (new layer) 1st: Import 60 mm Recycle 125 mm
		150	80	
		150	50	
		∞	7	
Poor support		40	AC	2nd: Import 150 mm (new layer) 1st: Import 185 mm Recycle 250 mm
		150	80	
		150	50	
		150	10	
		∞	3	

/Mechanical modify		Recycle with cement or lime		Recycle with bitumen	
Thick mm	CBR %	Thick mm	CBR %	Thick mm	CBR %
40	HMA	40	HMA		
125	100	125	100	40	HMA
125	80	250	CTB	125	BSM
150	50	90	50	150	50
∞	20	∞	20	∞	20
40	HMA	40	HMA		
150	100	150	100	40	HMA
125	80	275	CTB	200	BSM
125					
150	50			140	50
∞	7	∞	7	∞	7
40	HMA	40	HMA		
150	100	150	100	40	HMA
250	80	300	CTB	250	BSM
125					
150	50			140	50
150	10	150	10	150	10
∞	3	∞	3	∞	3

2nd: Import 125 mm New layer
1st: Recycle 250 mm with cement

Recycle 125 mm with bitumen

2nd: Import 150 mm New layer
1st: Recycle 275 mm with cement

Recycle 200 mm with bitumen

2nd: Import 150 mm New layer
1st: Recycle 300 mm with cement

Import 50 mm Recycle 250 mm with bitumen

A 40 mm thick asphalt surfacing is appropriate for these pavements although, in dry regions, a competent surface treatment (e.g. Cape Seal) is often preferred

5.1.5 Interurban highways (structural capacity: 30 million ESALs)

Existing roads falling into this category would normally have been constructed with a deep pavement structure and would usually include an asphalt base layer and surfacing. A good quality crushed stone or good natural gravel (CBR > 80%) would invariably lie beneath the asphalt layers, or a

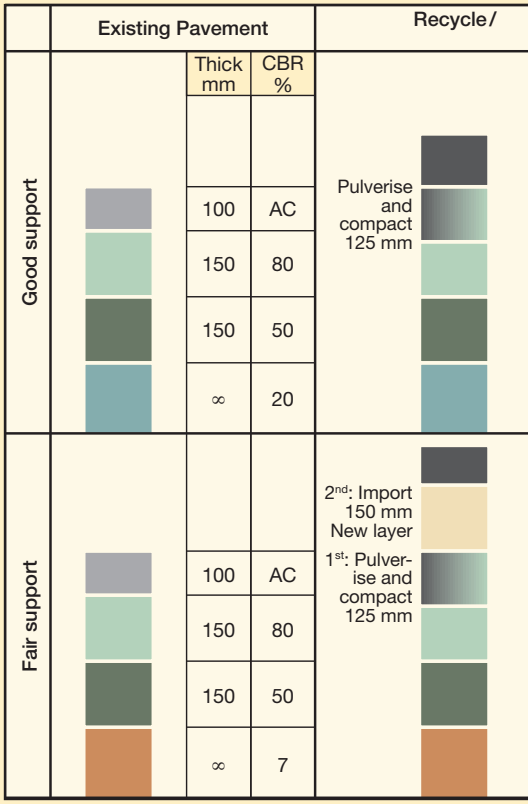
Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these main road pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 2 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 225 kPa, ITS_{WET} > 100 kPa, ITS_{EQUIL} > 175 kPa, cohesion > 250 kPa, angle of friction > 40°

- Asphalt
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel/soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm



layer of cement stabilised material.
Two different options are shown for the pavement that is recycled with cement or lime, one with a layer of crushed stone material on top of the recycled / cement stabilised layer, the other with asphalt.

Mechanical modify		Recycle with cement or lime						Recycle with bitumen		
		Crushed stone overlay			HMA overlay					
Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %
			50	HMA						
90	HMA	2nd: Import 125 mm (new layer)	125	100		80	HMA		50	HMA
125	100	1st: Recycle 275 mm with cement	275	CTB	Recycle 275 mm with cement	275	CTB	Recycle 250 mm with bitumen	250	BSM
125	80									
150	50		125	50		125	50		150	50
∞	20		∞	20		∞	20		∞	20
90	HMA		50	HMA					50	HMA
150	100	2nd: Import 150 mm New layer	150	100		90	HMA	Import 150 mm Recycle 275 mm with bitumen	275	BSM
125	100	1st: Recycle 300 mm with cement	300	CTB	Recycle 300 mm with cement	300	CTB			
125	80								125	80
150	50		100	50		100	50		150	50
∞	7		∞	7		∞	7		∞	7

These pavements always receive an asphalt surfacing with a minimum thickness of 40 mm

5.1.6 Major multi-lane highways (structural capacity: 100 million ESALs)

Existing roads falling into this category would generally have been constructed with a deep pavement structure that includes an asphalt base layer and surfacing. A good quality crushed stone or good natural gravel (CBR > 80%) would invariably lie beneath the asphalt layers, or a layer of cement stabilised material. Normally the asphalt will be in a distressed state with cracks affecting the full depth

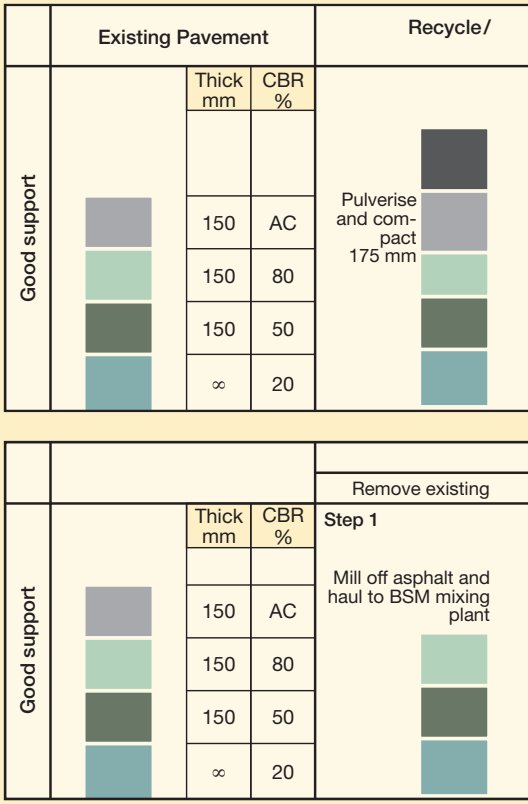
Mix designs are required to ensure that sufficient strength is achieved by adding stabilising agents. The minimum requirements for these main road pavements are:

- Cement stabilisation (denoted as CTB in the chart): UCS > 2 MPa
- Bitumen stabilisation (Denoted as BSM in the chart): ITS_{DRY} > 225 kPa, ITS_{WET} > 100 kPa, ITS_{EQUIL} > 175 kPa, cohesion > 250 kPa, angle of friction > 40°

- Asphalt
- Bitumen stabilised material (BSM)
- Cement stabilised material (CTB)
- CBR > 100 New GCS
- CBR > 80 GCS (graded crushed stone)
- 45 < CBR < 80 (coarse gravel)
- 25 < CBR < 45 (natural gravel)
- 15 < CBR < 25 (gravel / soil)
- 10 < CBR < 15 (sandy silty soil)
- 7 < CBR < 10 (silty soil)
- 3 < CBR < 7 (clayey silty soil)

Rules

1. **Recycling with cement:**
   minimum thickness: 150 mm
2. **Recycling with bitumen:**
   minimum thickness: 100 mm



of the layer. Two different options are shown for the pavement that is recycled with cement or lime, one with a layer of crushed stone material on top of the recycled / cement stabilised layer, the other with asphalt. In addition, an option that involves “two-part recycling” is included.

Mechanical modify		Recycle with cement or lime						Recycle with bitumen		
		Crushed stone overlay			HMA overlay					
Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %		Thick mm	CBR %
			50	HMA						
180	HMA	Import 150 mm (new layer)	150	100		90	HMA		50	HMA
175	100	Recycle 300 mm with cement	300	CTB	Recycle 300 mm with cement	300	CTB	Recycle 250 mm with bitumen	250	BSM
125	80								50	80
150	50		150	50		150	50		150	50
∞	20		∞	20		∞	20		∞	20

Two-part recycling to achieve deep composite pavement							
asphalt		Recycle in place with cement			Import recycled BSM & overlay		
Thick mm	CBR %	Step 2	Thick mm	CBR %	Step 3	Thick mm	CBR %
						50	HMA
					Import & pave 150 mm BSM	150	BSM
150	80	Recycle 250 mm with cement	250	CTB		250	CTB
150	50						
			50	50		50	50
∞	20		∞	20		∞	20

These pavements always receive an asphalt surfacing with a minimum thickness of 50 mm

5.2 Alternatives for pavement rehabilitation

The real challenge for a pavement engineer tasked with a rehabilitation design is selecting the alternative options for treating a distressed pavement. Often such comparisons do not go further than simplistic costing exercises, normally confined to initial construction costs. In addition, the alternative options for rehabilitation are often not properly formulated. This results in different options having different life expectations (structural capacity), making any comparison a futile exercise of subjectivity. Furthermore, the current increasing awareness of the need to reduce energy consumption in all walks of life brings a new dimension to evaluating construction activities.

This section aims to illustrate the importance of three primary aspects that should be addressed in comparing different pavements:

- the pavement structures being compared must have similar service lives. The pavement can be constructed initially to provide sufficient structural capacity for the duration of the service life with timeous interventions to meet functional requirements. Alternatively, a phased construction approach can be adopted with strengthening interventions that will allow the service life to be achieved;
- all costs incurred in achieving the required service life must be considered, not solely the initial cost of construction i.e. cradle to cradle. Provided the different pavements are properly conceived to provide similar service lives, such an exercise is relatively straightforward; and
- to provide an indication of the impact each different pavement has on the environment, the energy consumed by construction activities in achieving the required service life for each different pavement can be estimated and aggregated.

Several other aspects could also be incorporated in comparing different pavements over a defined service life, the most important being road-user costs. However, by tailoring maintenance and/or phased interventions aimed at keeping the functional properties of the pavement at similar levels, these may be ignored since they will all make similar contributions. Hence, including only the cost of all construction activities and energy consumed in the initial provision of an adequate pavement structure, the various interventions required during the service life, as well as rehabilitation at the end of the service life are considered adequate for comparing different pavement structures and portraying the “real picture” that allows a realistic whole-of-life comparison to be made.

Selection of pavement rehabilitation options can best be explained by way of an example. This section uses the comparison of four different options for rehabilitating a hypothetical pavement to demonstrate a proposed method for incorporating cost and energy in determining which option is optimal. A typical pavement structure (for many countries) consisting of thick asphalt concrete constructed on layers of granular material has been assumed. The four rehabilitation options selected for comparison include conventional construction methods as well as those that incorporate recycling material from the existing pavement. The key requirements are a structural capacity of 20 million equivalent 80 kN standard axle loads (ESALs) over a 20-year service life.

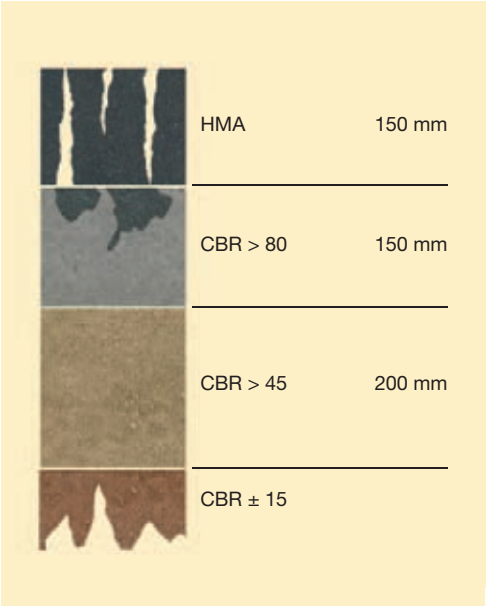
The envisaged maintenance / strengthening interventions and rehabilitation requirements after 20-years are then defined and all construction activities quantified. These quantities allow a whole-of-life cost estimate to be calculated, based on unit rates prevailing in the country's construction industry.

The Present Worth of Costs based on the whole-of-life cost estimate and different discount rates are then used to evaluate the relative economic differences.

Finally, the same quantities are used to determine the amount of energy consumed during the service life, including rehabilitation after 20 years.

5.2.1 Existing pavement

The figure adjacent shows the structure of a pavement that is typical for a heavily trafficked road. Full-depth cracking in the asphalt layers indicates that the pavement has reached the end of its service life. The various pavement layers are 150 mm of thick asphalt concrete overlying 350 mm of granular material consisting of two layers; a 150 mm layer of good quality graded crushed stone (CBR > 80%) above a 200 mm thick layer of natural gravel (CBR > 45%). The total cover to the underlying subgrade is therefore 500 mm. The subgrade is assumed to have an in situ resilient modulus of 85 MPa.



Distress symptoms are typical for such a pavement structure at the end of the service life. The asphalt material has suffered fatigue cracking with cracks propagating thorough the full thickness of asphalt. Such cracks allow water to enter the underlying crushed stone material causing saturation and leading to the hydraulic displacement of fines (pumping) when subjected to heavy traffic loads. The consequence of pumping is degradation of the layer and pothole development. This pavement has reached its terminal state and requires major rehabilitation.

5.2.2 Rehabilitation requirements

Rehabilitation objectives call for a 20 year service life. Anticipated traffic over this period indicates a structural capacity requirement of 20 million ESALs. To meet the normal requirements for ride quality and skid resistance, an ultra-thin friction course (UTFC) surfacing is required. Such a surfacing is expected to provide a service life of between six and eight years. At the end of the 20 year service life, rehabilitation will be required to restore structural capacity.

5.2.3 Rehabilitation options

Four alternative design options are considered below, each meeting the structural capacity requirement of 20 million ESALs. The AASHTO 1993 Design Method (Structural Numbers) provides a simple means of evaluating the alternative pavement structures, using the following input data to determine the Structural Number requirer (SN_{REQ}):

- Subgrade support conditions: CBR 15% (average 85 MPa/12 392 psi)
- Reliability: 90%
- Standard deviation: 0.45
- Initial serviceability: 4.2
- Terminal serviceability: 2.5

A SN_{REQ} value of 4.63 is obtained by using the above data as input for an appropriate SN computer programme.

Option 1. Patch and overlay
This option is popular in many first-world countries, primarily due to the speed and simplicity of construction. Severely cracked sections that fall within the wheel paths are milled out and replaced with fresh asphalt before an overlay is applied. A milling machine with a 1 m cut width can be utilised to cut a 75 mm deep strip following the wheel path. Continuously graded hot mixed asphalt (HMA) is used as backfilled material, placed by paver and compacted.

To minimise the thickness of asphalt overlay, a phased construction approach is adopted. A 60 mm thick asphalt base surfaced with a 30 mm thick UTFC surfacing is estimated to provide a 7 year life before cracks reflecting from the underlying fatigued structure demand intervention. This is timed to coincide with the requirement to replace the UTFC surfacing as it would reach the end of its functional life after 7 years. It is optimistically estimated that milling off and replacing the UTFC layer together with 35 mm of underlying asphalt will carry the design traffic for a further 7 years when the same treatment will be required to achieve the overall 20 year service life. At that stage, advanced distress (in the form of bitumen stripping and “washboarding’) can be expected in the body of the asphalt, requiring deep milling to address the problem. Rehabilitation requirements are expected to be the same as those described under Option 2 below.

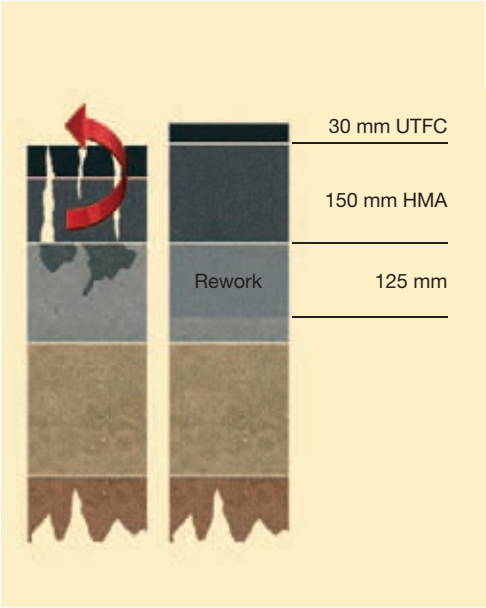
Option 1. Patch and overlay. Structural Number determination				
Layer	Layer coefficient (C_L per inch)	Drainage coefficient (C_D)	Layer thickness (t) (mm / inch)	Layer contribution (C_L x C_D x t)
New UTFC	0.44	1	30 / 1.2	0.53
New asphalt binder	0.42	1	60 / 2.4	1.01
Patched asphalt	0.33 *	1	75 / 3	0.99
Aged asphalt	0.22 **	1	75 / 3	0.66
Old GCS base	0.12 ***	1	150 / 6	0.72
Gravel subbase	0.10	0.9	200 / 8	0.72
			SN_{ACT}	**4.63**

* Structural layer coefficient of 0.33 reflects the part new / part aged composition of the layer
** Structural layer coefficient of 0.22 reflects the aged (brittle) nature of the asphalt
*** Structural layer coefficient of GCS reduced from 0.14 to 0.12 since pumping has occurred

Option 2. Mill and replace
This rehabilitation method calls for the entire thickness of distressed asphalt affected by full-depth cracking to be milled off and removed from site. The underlying crushed stone base will then require repairing by in situ reworking (to a nominal depth of 125 mm) before paving a 150 mm thick asphalt concrete base, followed by the 30 mm UTFC surfacing.

The figure illustrates the operations required. Reworking the crushed stone base implies that traffic will have to be diverted for sufficient time to allow the base material to dry back before the asphalt base can be paved, followed by the UTFC surfacing.

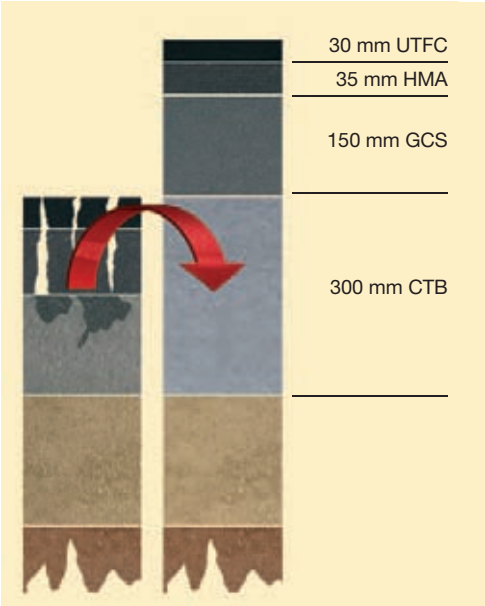
The critical layer in this pavement is the combined asphalt base and surfacing layers that will suffer fatigue cracking due to tensile strain level developing at the bottom of the asphalt concrete. Two maintenance interventions are envisaged to coincide with the anticipated life of the UTFC surfacing. After 7 and 14 year intervals, only the UTFC will need replacing. At the end of the service life, fatigue cracks would have reached the surface, allowing water to ingress through to the underlying granular layers, causing the same distress and failure mechanism that the pre-rehabilitated pavement has suffered. At that stage, rehabilitation requirements are expected to be the same as those described under Option 1 above.



Option 2. Mill and replace. Structural Number determination				
Layer	Layer coefficient C_L (per inch)	Drainage coefficient C_D	Layer thickness t (mm / inch)	Layer contribution C_L x C_D x t
New UTFC	0.44	1	30 / 1.25	0.53
New asphalt base	0.42	1	150 / 6	2.52
Reworked GCS base	0.14	1	150 / 6	0.84
Gravel subbase	0.1	0.9	200 / 8	0.72
			SN_{ACT}	**4.61**

Option 3. Recycle/cement stabilise and overlay
A standard rehabilitation approach that is popular in several parts of the world is shown in the figure below. It calls for the upper 300 mm of existing pavement to be recycled in situ, stabilised with cement. Such a blend of recovered asphalt pavement material and crushed stone would normally require the addition of some 2.5% (by mass) of cement to achieve an unconfined compressive strength (UCS) of 2 MPa.

After a curing period of 7 days, a new 150 mm thick highly densified graded crushed stone base is constructed from on top of the new subbase. Achieving high levels of density requires the layer to be "slushed" that, in turn, calls for a drying out period before the asphalt binder layer and UTFC surfacing can be applied.



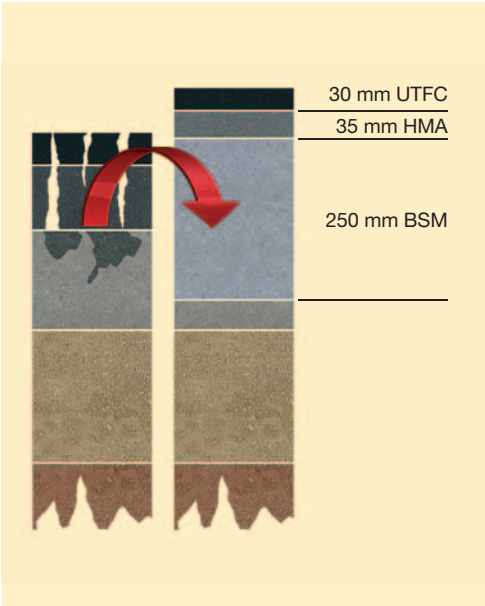
Since a cohesionless crushed stone base cannot tolerate the action of traffic without ravelling, this method of rehabilitation calls for all traffic to be diverted away from the works until the asphalt has been applied.

The critical layer in this pavement is the crushed stone base. The failure condition assumed is 20 mm of permanent deformation followed by degradation due to moisture-activated distress.

Two maintenance interventions are envisaged to coincide with the anticipated life of the UTFC surfacing. After 7 years, only the UTFC will need replacing. After a further 7 years (i.e. 14 years after the initial rehabilitation), both layers of asphalt will need replacing to ensure that the 20 year service life is achieved. At the end of the service life, deformation in the wheel paths can be expected to be in the order of 20 mm. At that stage, rehabilitation requirements are expected to be the same as those described under Option 4 below.

Option 3. Recycle/cement stabilise and overlay. Structural Number determination				
Layer	Layer coefficient C_L (per inch)	Drainage coefficient C_D	Layer thickness t (mm/inch)	Layer contribution C_L x C_D x t
New UTFC	0.44	1	30/1.2	0.53
New asphalt binder	0.42	1	35/1.3	0.54
New GCS base	0.14	1	150/6	0.84
Recycled CTB	0.17	1	300/12	2.04
Gravel subbase	0.1	0.9	200/8	0.72
			SN_{ACT}	**4.67**

Option 4. Recycle/bitumen stabilise
This rehabilitation method calls for the upper 250 mm of the pavement to be recycled in situ with the addition of a bitumen stabilising agent, as illustrated in the figure below. (Assumed application rates are 2.2% residual bitumen and 1% cement (by mass.)) One of the reasons for this method becoming popular is the increase in cohesion of the stabilised material that allows the completed layer to be opened to traffic soon after it has been compacted (normally to a density in excess of 100% of the modified AASHTO T-180 density) and finished off. When a properly-formulated bitumen emulsion is used as the stabilising agent, a delay of between 2 and 4 hours is required to allow the emulsion to break sufficiently. Instant cohesion is, however, achieved on compaction when foamed bitumen is used as the stabilising agent.



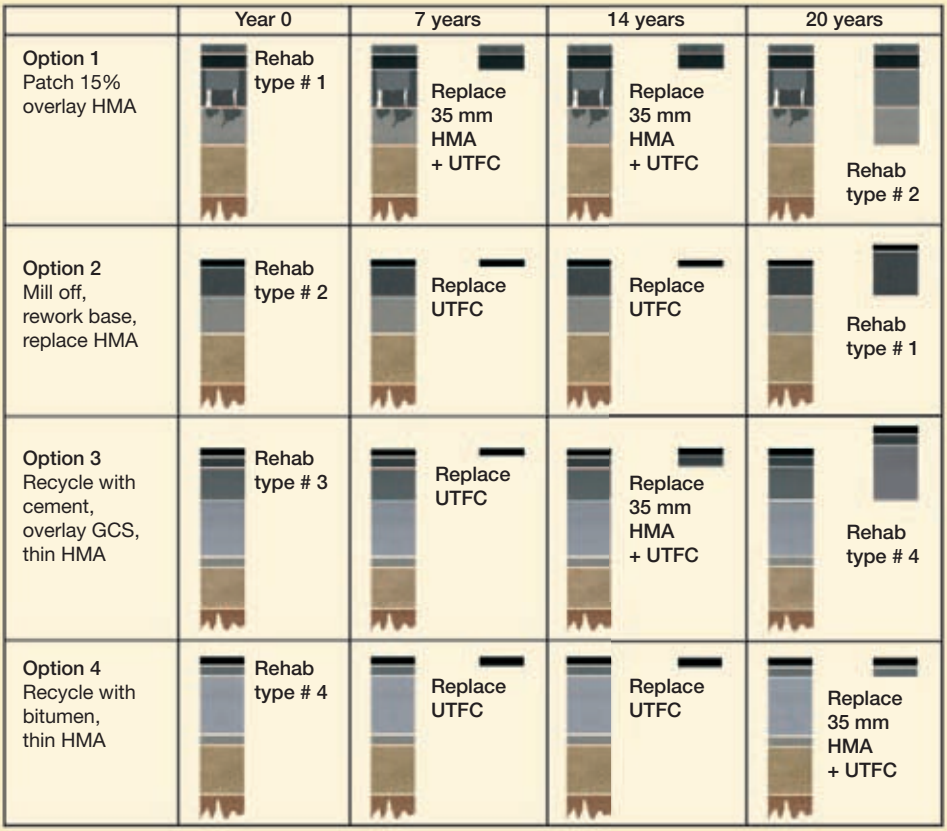
Since the asphalt concrete surfacing cannot be applied until the moisture content of the recycled base has reduced (< 50% of the optimum is normally specified), a fog spray of dilute emulsion is usually applied to prevent the finished surface from ravelling under traffic action.

The critical layer in this pavement is the bitumen stabilised base. However, the deviator stress ratio is below 30% implying that the failure condition assumed (20 mm of permanent deformation in the wheel paths) will not be reached after the repeated loading of 20 million ESALs. (Note: Permanent deformation will occur, but the total amount will be less than 20 mm.) Two maintenance interventions are envisaged to coincide with the anticipated life of the UTFC surfacing. After 7 and 14 year intervals, only the UTFC will need replacing. At the end of the service life, permanent deformation will be evident in the wheel paths, and this can be addressed by milling off and replacing the asphalt layers. This will return the pavement to its original state and restore the structural capacity.

Option 4. Recycle/bitumen stabilise. Structural Number determination				
Layer	Layer coefficient C_L (per inch)	Drainage coefficient C_D	Layer thickness t (mm/inch)	Layer contribution C_L x C_D x t
New UTFC	0.44	1	30/1.2	0.53
New asphalt binder	0.42	1	35/1.3	0.54
Recycled BSM base	0.26	1	250/10	2.60
Remaining GCS	0.14	1	50/2	0.28
Gravel subbase	0.1	0.9	200/8	0.72
			SN_{ACT}	**4.67**

5.2.4 Maintenance requirements

The figure below summarises the various maintenance and rehabilitation measures explained above, that are applicable in this example.



Maintenance intervals and rehabilitation required after 20 years

5.2.5 Construction & maintenance costs

The table below is an example of the exercise that needs to be undertaken in order to obtain the cost of the various construction activities for each rehabilitation option and for the different maintenance interventions. This example uses average unit rates prevailing in the South African roads industry in 2011.

Cost estimate for each option (Quatities for 1 km of road 10 m wide) using average unit rates									
Schedule			Rehab option # 1		Rehab option # 2		Rehab option # 3		
Item	Unit	Rate (US$)	Patch/overlay		Mill & replace		CTB/GCS/HMA		
			Quantity	Amount	Quantity	Amount	Quantity	Amount	
Milling HMA patch	m³ ton	30,0 70,0	281 703	8,438 49,219	1,500	45,000			
Reprocess GCS Prime	m³ m³ m²	13,2 50,0 0,5			1,250 10,000	16,500 5,000	 3,150 10,000	 157,500 5,000	
Tack 30 UTFC HMA	m² ton ton	0,4 80,0 70,0	21,500 750 1,500	8,600 60,000 105,000	20,000 750 3,000	8,000 60,000 210,000	20,000 750 875	8,000 60,000 61,250	
20 km haul	m³ km	0,4	5,625	2,250	30,000	12,000	incl	0	
Recycle 300 CTB 250 BSM	m³ m³	5,5 6,6					3,000	16,500	
Cement Bitumen	ton ton	150,0 400,0					158	23,625	
				233,506		**356,500**		**331,875**	

	Rehab option # 4		Intervention # 1		Intervention # 2	
	BSM / HMA		UTFC only		35 mm HMA + UTFC	
	Quantity	Amount	Quantity	Amount	Quantity	Amount
			300	9,000	650	19,500
	30,000 750 875	12,000 60,000 61,250	10,000 750	4,000 60,000	20,000 750 875	8,000 60,000 61,250
			6,000	2,400	13,000	5,200
	2,500	16,500				
	53 116	7,875 46,200				
		191,825		**75,400**		**153,950**

These costs can then be converted to ratios, as shown in the following table.

Cost per kilometre (zero discount rate)					
Rehabilitation option	Initial rehabilitation	7 year intervention	14 year intervention	Rehabilitation after 20 years	Total Cost
Option 1	2.3X	1.5X	1.5X	3.6X	9.0X
Option 2	3.6X	0.75X	0.75X	2.3X	7.4X
Option 3	3.3X	0.75X	1.5X	1.9X	7.5X
Option 4	1.9X	0.75X	0.75X	1.5X	5.0X

The effect of different discount rates can then be evaluated using the Present Worth of Costs (PWoC) model, as shown in the figure below. This figure highlights the importance of considering the time value of money. Not only are the rankings of the different options (Options 1, 2 and 3) affected, but this exercise emphasizes the true benefit of adopting a technology that provides improved performance over the full service life.

Present Worth of Costs for Alternative Rehabilitation Options

5.2.6 Energy consumption

An increasing awareness of climate change is making society focus more on energy consumption. The construction industry is not exempt and several studies have been undertaken to estimate the amount of energy being consumed, particularly in the construction of roads where large machinery is employed and the quantities of material either consumed or moved is high.

Studies have been carried out to evaluate how much energy is consumed in the production of construction materials (e.g. bitumen, cement, aggregates, etc.), as well as various construction activities (e.g. excavating, transporting, asphalt paving, etc.)

Several authors have published papers highlighting the savings that can be anticipated from adopting different construction techniques (e.g. recycling material from an existing pavement compared to conventional construction processes). Applying these approaches systematically allows the total energy consumed by all construction activities to be determined for each option, as shown in the table on the next page.

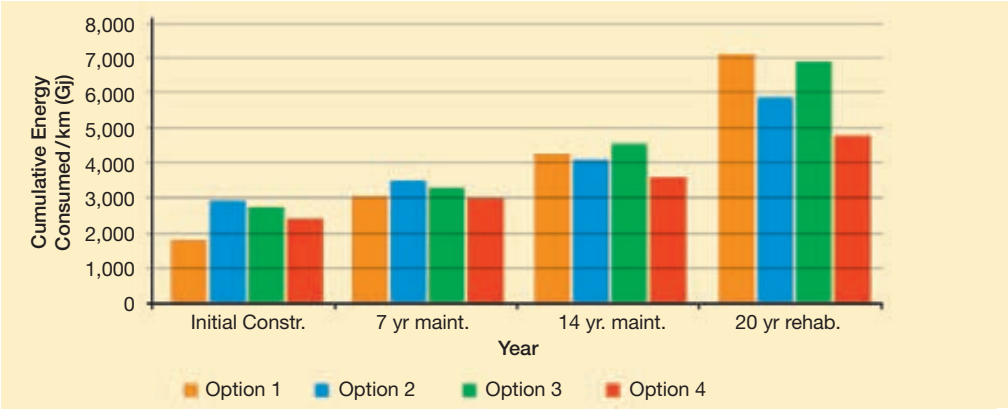
Note. There are several different sources of energy consumption data for the construction industry, some dating back to the 1970s when energy concerns were first popularised. For illustrative purposes, this example uses one source that was recently published in New Zealand.

Estimate of energy consumption for each option (Quatities for 1 km of road									
Schedule			Rehab Option # 1		Rehab Option # 2		Rehab Option # 3		
Item	Unit	Rate (MJ)	Patch/overlay		Mill & replace		CTB/GCS/HMA		
			Quantity	Energy	Quantity	Energy	Quantity	Energy	
Milling	ton	5	281	1,406	3,750	18,750			
Ingredients	ton	348	281	97,788					
Mix/Pave	ton	320	281	89,920					
Crush GCS and import									
GCS	ton	50					3,150	157,500	
Recycle existing pavement									
125 GCS	ton	11			2,625	28,875			
250 BSM	ton	11							
300 CTB	ton	11					6,300	69,300	
Stabilising agents									
Cement	ton	7,000					158	1,102,500	
Bitumen	ton	6,000							
Process & finish off layers									
125 GCS	m^2	10			10,000	100,000			
150 GCS	m^2	10					10,000	100,000	
300 CTB	m^2	10					10,000	100,000	
250 BSM	m^2	10							
30 UTFC									
Ingredients	ton	407	750	305,250	750	305,250	750	305,250	
Mix/pave	ton	320	750	240,000	750	240,000	750	240,000	
HMA									
Ingredients	ton	348	1,500	522,000	3,000	1,044,000	875	304,500	
Mix/pave	ton	320	1,500	480,000	3,000	960,000	875	280,000	
20 km haul	ton-km	20	2,812	56,245	7,500	150,000	1,625	32,500	
				1,792,609		**2,846,875**		**2,691,550**	

10 m wide) Unit rates (MJ) derived from Patrick et al						
	Rehab Option # 4		Intervention # 1		Intervention # 2	
	BSM / HMA		UTFC only		35 mm HMA + UTFC	
	Quantity	Energy	Quantity	Energy	Quantity	Energy
			750	3,750	1,625	8,125
	5,250	57,750				
	53 116	367,500 693,000				
	10,000	100,000				
	750 750	305,250 240,000	750 750	305,250 240,000	750 750	305,250 240,000
	875 875	304,500 280,000			875 875	304,500 280,000
	1,625	32,500	1,500	30,000	3,250	65,000
		2,380,500		**579,000**		**1,202,875**

The cumulative energy that is consumed by the various construction activities (including maintenance interventions and rehabilitation at the end of the service life) for all four options can be compared. For comparison purposes, these data are summarised in the table below and shown graphically in the following figure.

Cumulative energy consumed per kilometre (in GJ)				
Rehabilitation option	Initial Constr	7 yr maint.	14 yr maint.	20 yr rehab.
Option 1	1,793	2,996	4,199	7,046
Option 2	2,847	3,426	4,005	5,798
Option 3	2,692	3,271	4,474	6,855
Option 4	2,381	2,960	3,539	4,742



Cumulative Energy Consumption for Different Rehabilitation Options

Similar to the trend shown in the costing exercise, the above figure illustrates the true benefit of adopting a technology that provides improved performance over the full service life with the picture changing from that painted by the energy consumed during the initial construction.

5.2.7 Relevant comments

Environmental considerations in pavement engineering are no longer esoteric. Increasing emphasis on the environmental impact of road construction and rehabilitation, has led to sufficient data becoming available for use in analysis and decision making. Energy consumption figures have been used in combination with whole-of-life costs for four realistic rehabilitation options currently used globally in road pavements.

This provides insight into project selection leading to the following conclusions:

- Initial construction costs alone are inadequate for selection of rehabilitation alternatives. They can provide skewed and unrealistic rehabilitation selection, which will lead to unnecessary wastage of resources.
- Whole-of-life analysis using PWoC provides more realistic financing requirements for pavement upkeep over the entire analysis period.
- Energy consumption and its impact on the environment can be predicted. The inclusion of an energy consideration may influence the rankings of different rehabilitation options and thus decision making.

6 Recycling 100% reclaimed asphalt pavement (RAP) material

Asphalt is manufactured using the best available aggregates and, since these do not deteriorate over the life time of a pavement, it is not surprising that RAP is (and has been for decades) one of the most recycled materials in the world.

The advent of modern milling machines that granulate the asphalt in situ has significantly increased the salvage value of the resulting RAP by making it a more useful material at no extra cost. This chapter focuses on reusing RAP as a cold recycled material.

There are two ways that RAP material can be treated as a cold recycled material:

- In situ treatment. The existing asphalt can be reclaimed and simultaneously treated using “cold in-place recycling” technology (also referred to as “CIR” or “partial depth” recycling). This process uses a large track-mounted recycler to recover the upper portion of asphalt in the existing pavement (normally between 100 mm and 150 mm cut depth) and simultaneously mix the RAP with additives. The recyclers used for this process can normally treat a full lane width in a single pass (e.g. the Wirtgen WR 4200 or the 2200 CR fitted with a 3.8 m wide cutting drum/paving screed).
- In plant treatment. The milled asphalt is stockpiled locally and treated in a Wirtgen KMA 220 mixing plant. The treated RAP material is then returned to site and paved back in the areas that were milled. Known as “cold in plant recycling”, this technology introduces flexibility into the recycling process by separating the recovery and reuse operations. In addition, where required, this process provides the opportunity to crush and/or screen the RAP material, as well as blend it with fresh aggregate (e.g. graded crushed stone) if required, before it is recycled.

6.1 RAP material

The term “RAP” is given to any (100%) asphalt material (also referred to as “asphalt concrete”) that is recovered from an existing pavement. The type of asphalt that was originally used together with variations in the asphalt mix will therefore be reflected in the RAP material. Where multiple asphalt layers of different mixes are recovered by milling (or crushed from slabs), the resulting RAP will be a blend of all component mixes. The two key features of a RAP material are the bitumen binder (the state, amount and consistency of bitumen in the material) and the grading. Also important is the tendency for the larger RAP particles to break down further when subjected to mixing and compaction forces.

6.1.1 Bitumen binder

From a cold recycling perspective, it is most important to know whether the bitumen in the RAP material is “active or inactive”. In other words, is the RAP a “black aggregate” (inactive) with properties similar to those of graded crushed stone or is it “sticky material” (active) with inherent cohesiveness due to the bitumen in the RAP material? This is important since the state of the old binder will have a significant influence on how the recycled material will behave when it is reused.

The following observations will indicate that the RAP material may be regarded as inactive:

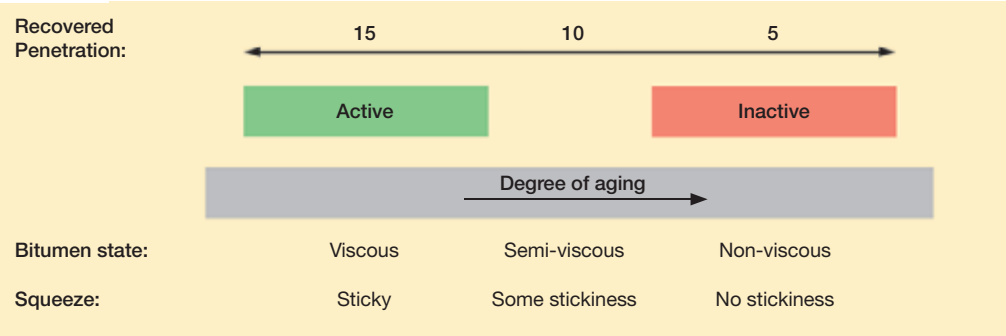
- Visual appearance: the RAP is a dull grey colour with no black shining surfaces.
- Brittleness: a chunk of RAP breaks cleanly into pieces.
- Adhesion: pieces of RAP (at ambient temperature) do not stick the hand when a sample is firmly squeezed.

Where there is doubt as to whether or not the RAP can be classified as active or inactive, a representative sample may be tested in a laboratory to determine:

- the amount of bitumen in the RAP material (percentage by mass), and
- the rheological properties of the recovered bitumen (penetration (Pen), softening point and viscosity).

The results of these tests should be used only as indicators since the tests themselves do not provide definitive answers; the results are influenced by operator competence. The bitumen extraction procedure does not necessarily extract all the bitumen (especially that amount absorbed by the aggregate) and the amount of bitumen determined may include some material smaller than 0.075 mm (filler). Recovered Pen values are notoriously variable, largely due to the influence of the solvent used to extract the bitumen. This is a “delicate” test that demands diligence on the part of the operator if repeat tests are to yield a similar result.

In spite of these concerns, the two parameters thus determined provide a meaningful guideline in classifying a RAP material. The Pen value will indicate the viscosity of the recovered bitumen, allowing the RAP to be classified as active or inactive:



The bitumen content of RAP is of little significance where the material is classified as inactive. However, depending on the intended use, the amount of bitumen in an active RAP material can be an important consideration. The various uses for recycled RAP material are discussed below in Section 6.2.

(A quick and easy way of estimating whether a RAP sample is active or inactive is to heat the sample to 70° C and manufacture 100 mm Ø specimens. Soak the specimens for 24 hours before carrying out ITS tests. If the soaked ITS value is > 100 kPa the RAP should be regarded as active.)

6.1.2 Grading of the RAP material

It is important to recognise that the grading of milled RAP material will always be influenced by the purpose of the milling operation. Where a contractor is milling for the sole purpose of removing asphalt from the road, his focus will be on production at the lowest possible cost, not on the grading of the RAP material. However, where 100% asphalt material is recycled in situ, the attention will be entirely different as grading of the milled RAP is of primary importance.

The grading of milled RAP material is influenced by the condition of the in situ asphalt and the milling operation. The main factors influencing the grading of the RAP are:

- the composition and uniformity of the existing asphalt material
- the condition of the existing asphalt material
- the temperature of the asphalt in the milled horizon
- the depth of milling
- the speed of advance of the milling machine
- the rotation speed of the milling drum
- the type of milling drum and condition of the milling tools
- the direction of cut (up-cutting or down-cutting)

6.2 Uses for cold recycled RAP material

Untreated RAP material may be used as a substitute for conventional material to construct a new pavement layer. However, it should always be borne in mind that RAP material is derived from good quality aggregates and is therefore a valuable material that should not be squandered by using it where such quality is not warranted. It should rather be retained and used as a substitute (treated where necessary) where conventional pavement materials are expensive and/or scarce. Furthermore, when treated with a suitable additive (e.g. bitumen), RAP can be used as a premium material to construct the upper layers of pavements that carry heavy loads.

The following sections discuss the options available for using RAP as a cold recycled material.

6.2.1 Untreated RAP material

For more heavily trafficked roads, only inactive RAP should be used as a substitute for graded crushed stone in a base layer. Such a layer of RAP material is constructed using the same procedures and meeting the same density requirements as those applicable to conventional crushed stone bases.

Due to the viscosity of the residual bitumen, RAP material classified as active has higher levels of cohesion than inactive RAP or natural material. Such cohesion will resist compaction effort and limit the density achieved during construction. However, under the dynamic action of applied traffic loads, the material will slowly consolidate. To counter this tendency, RAP material that classifies as active should always be blended with a nominal 30% (by volume) of graded crushed stone to achieve an inactive status.

6.2.2 RAP material treated with cement

One of the alternatives for treating RAP material is stabilisation with cement. This is normally undertaken for the purpose of constructing a semi-rigid layer in the pavement that is then covered with additional pavement layers (usually asphalt).

To determine the amount of cement addition that is necessary to achieve the required properties, a conventional "cement stabilisation mix design" should always be undertaken (as described in Section 4.2).

However, RAP is invariably a high quality material that can be reused more effectively when treated with bitumen (e.g. bitumen emulsion or foam bitumen) to take advantage of:

- the adhesive properties of the existing bitumen in the RAP which offers improved cohesion; and
- the viscosity and flexibility of the bitumen in the RAP.

6.2.3 RAP material treated with bitumen emulsion

When treating RAP material with bitumen emulsion, the extended volume of the emulsified bitumen will tend to coat the individual RAP particles. This can lead to continuous bonding, making the mix asphaltic (i.e. a cold mix asphalt material). The expected behaviour of the treated RAP material will be influenced by the following:

- Where the Pen value of the bitumen recovered from the RAP classifies the material as active (Pen value > 10), the treated material may be regarded as asphaltic.
- Where the bitumen content of the RAP material exceeds 5%, the treated material will have more asphaltic tendencies.
- Where the residual bitumen content of the emulsion added to the RAP material exceeds 2%, then asphaltic behaviour can be expected.

Where any two of the above three conditions are met, treating with bitumen emulsion will create an asphaltic material and an asphalt-type mix design should be undertaken (normally a Marshall Design or volumetric analysis of the mix in order to ascertain the design binder content). In addition, an asphalt-type product can be achieved by adding an emulsion containing a rejuvenating agent, especially when the Pen value of the bitumen recovered from the RAP material > 15.

Where the RAP material does not fit this category, treating with bitumen emulsion is more likely to produce a non-continuously bound stabilised material (BSM-emulsion) than a cold mix asphalt. A mix design should always be undertaken following the standard procedure outlined in Chapter 4 and described in Appendix 1. The table below summarises the selection of mix design method.

Guideline for selecting a mix design method when treating RAP with emulsion

Influencing factors	Material behaviour	
	Stabilised Type	Asphalt Type
1.RAP: Recovered Pen	< 10	> 10
2.RAP: Recovered bitumen (%)	< 5%	> 5%
3. ITS_{WET} specimens manufactured at 70° C (kPa)	< 100	> 100
4.Emulsion: Residual bitumen(%)	< 2%	> 2%
5.Rejuvenating agent	No	Yes
Mix design method	**Appendix 1**	**Marshall**

Bitumen emulsions are usually specially formulated for treating 100% RAP material. These can include base bitumen that deviates from the standard 80/100 Pen grade that are normally used to manufacture emulsions and can incorporate either harder or softer bitumen. The selection of the correct type of bitumen emulsion for each application is essential, as outlined below.

Worldwide, medium to slow set (stable grade) cationic bitumen emulsions are almost exclusively used for treating RAP material. The main factors influencing the formulation of a specific bitumen emulsion are:

- Break time. The time taken for the bitumen to break out of suspension can be controlled through chemical interaction between the emulsion and the aggregate.
- Cement addition. A small percentage of cement is sometimes added to “destabilise” the emulsion and trigger the break.
- Coating of the aggregate particles. This is normally assessed visually and provides a primary indication that the formulation is correct.
- Cohesion of the mix. This is influenced by the interaction between the emulsion and the aged bitumen on the aggregate that sometimes necessitates the inclusion of a rejuvenating agent.

A cold mix asphalt product is normally appropriate where distress is confined to the top of the pavement and the thickness of asphalt is sufficient to allow a thin (< 150 mm) layer of asphalt to be recycled in situ. Similar to HMA, the performance of an asphalt-type RAP material treated with bitumen emulsion is sensitive to changes in grading. For this reason it is imperative to carry out a representative mix design and to identify uniform sections of asphalt that will be recycled in situ. Visual observations and results from tests on extracted cores are often used to identify such sections. However, more reliable information can be obtained by evaluating the grading and condition of milled RAP that is generated from trial milling at various locations along the road.

Where a consistent grading cannot be guaranteed, the asphalt should rather be milled off, stockpiled and screened before being treated in a stationary plant. Where necessary, oversized RAP particles should be crushed and screened.

3800 CR: In situ full lane recycling

WR 4200: In situ full lane recycling

KMA 220: Ensuring consistency in the RAP material by pre-screening

The following special considerations should be noted for mix designs of RAP material treated with bitumen emulsion:

- Marshall Asphalt Mix Design. Various countries have their own guidelines for Stability and Flow limits of cold recycled mixes that differ from HMA limits. (It is not uncommon to obtain high Stability values for such cold recycled mixes.) The Basic Asphalt Recycling Manual published in the USA by the Asphalt Recycling and Reclamation Association (ARRA) provides some useful guidance as to appropriate values for these materials.
- Stabilisation Mix Design. As shown in the table on the previous page, where the grading of the RAP material is relatively fine and the Pen value of the recovered bitumen exceeds 10, high ITS_{DRY} values can be generated, sometimes in excess of 500 kPa. It is for this reason that a minimum ITS_{WET} value of 100 kPa has been adopted rather than a TSR percentage value which would be unnecessarily onerous. However, the combination of high ITS_{DRY}/low TSR values can also be used as an indication that the treatment process is not purely one of stabilisation but is tending towards the more continuously bound (asphaltic material). This, in turn, is a warning that the RAP material should be blended with a nominal 15% (by volume) crusher dust if a non-continuously bound stabilised material is required.

6.2.4 RAP material treated with foamed bitumen

Foamed bitumen is a stabilising agent. Bitumen added to a RAP material in a foamed state disperses as tiny bitumen “splinters” that do not coat the RAP particles with a film of fresh bitumen. The reason for adding foamed bitumen is not to rejuvenate the aged bitumen in the RAP material (to produce an asphaltic product), but to achieve a bitumen stabilised material (BSM) with its own characteristics and advantages.

From a BSM-foam perspective, treating a RAP material is different from treating all other materials due to the presence of the aged bitumen. Whereas non-RAP material demands the presence of a minimum amount of dust (>4% by mass < 0.075 mm) for the bitumen to effectively disperse, RAP material with as little as 1% passing the 0.075 mm sieve can be successfully treated with foamed bitumen. It is understood that this is due to individual bitumen splinters having sufficient heat energy to warm and adhere (as spots) to the aged bitumen in the RAP.

Provided the RAP material can be classed as inactive, treating with foamed bitumen will produce a non-continuously bound bitumen stabilised material. If the RAP material falls into the active class, then some adherence between the old bitumen coating individual RAP particles is likely, leading to a more continuously bound material that is neither asphaltic nor stabilised. For this reason, RAP material classed as active should be blended with a nominal 30% (by volume) graded crushed stone (normally with a maximum size of 20 mm) to prevent adherence between particles as a consequence of the old bitumen remaining “sticky”.

However, regardless of the apparent nature of the RAP material and whether or not it appears to be similar to that previously treated, a mix design should always be carried out on representative samples to determine the characteristics of the treated product. In particular, the results of ITS tests (100 mm diameter specimens described in Appendix 1) should be carefully interrogated since these provide the best indicators of material behaviour. As discussed under Section 4.3.11, ITS_{DRY} values in excess of 500 kPa are an indication of asphaltic behaviour, suggesting that blending with graded crushed stone (or crusher dust) is required to ensure that a non-continuously bound stabilised product is achieved.

As with all BSM-foam materials (described in Chapter 4), the bitumen demand for effective stabilisation is a function of the grading of the parent material. RAP materials tend to be coarse and therefore demand a relatively low application of foamed bitumen for effective stabilisation, normally in the range of 1.6% to 2.2% (by mass).

Hot climates. Where the base course of a heavy duty pavement in a hot climate is constructed from RAP material treated with foamed bitumen, the following aspects need attention:

- the shear properties of the mix should be determined from triaxial tests carried out at representative temperatures;
- the axle mass of heavy vehicles needs to be controlled. Overloading will generate disproportionately high stresses in the base that may result in accelerated deformation; and
- as an insurance against loss of cohesion and/or a tendency towards continuous binding, all mixes should be blended with virgin material (nominal 15% crusher dust for inactive RAP or 30% graded crushed stone for active RAP material (both by volume)).

As a final word, foamed bitumen stabilised RAP materials have proved to be remarkably successful on numerous projects worldwide and, provided they are properly designed and constructed, can be used with confidence as a substitute for asphalt bases.

Bibliography

– AASHTO. Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials Washington D.C., 1993.

– American Concrete Pavement Association. Pavement Analysis Software, base on the 1993 AASHTO Guide for the Design of Pavement Structures. Washington D.C. 1993.

– A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries. 1993. 4th Edition. Crowthorne, Berkshire: Transport Research Laboratory (TRL). (Overseas road note 31).

– Basic Asphalt Recycling Manual. Asphalt Recycling and Reclaiming Association (ARRA). Annapolis, Maryland, USA. 2001

– Bonfim V. Cold Milling of Asphalt Pavements. Published by Suiang G Oliveira, English version by Priscila Podboi Adachi & Patricia Pick, Excerçáo, Sao Paulo, Brazil, 2008

– Bredenhann SJ and Jenkins KJ. Determination of Stress-Dependent Material Properties with the FWD, for use in the Structural Analysis of Pavements using Finite Element Analysis Techniques. Conference on Asphalt Pavements for Southern Africa CAPSA '04, Sun City, South Africa, September 2004

– Claessen, A.I.M, and Ditmarch, R. Pavement evaluation and overlay design. The Shell Method. Proceedings of the Fourth International Conference on the Structural Design of Asphalt Pavements. Vol.1, Ann Arbor, 1977.

– Collings D.C., and Jenkins K.J., The long term behaviour of bitumen stabilised materials. 10th Conference on Asphalt Pavements in Southern Africa CAPSA 2011, Drakensberg, South Africa

– Collings D.C. and Jenkins K.J., Whole-of-Life Analysis of Different Pavements: The Real Picture. First International Conference on Pavement Preservation, Newport Beach, California, for April 2010

– Committee of State Road Authorities (CSRA), Guidelines for road construction materials.. Pretoria Committee of State Road Authorities Department of Transport (DoT). (DoT technical recommendations for highways; draft TRH14). 1985

– De Beer, M. 1990. Aspects of the design and behaviour of road structures incorporating lightly cementitious layers. Ph.D dissertation, University of Pretoria, Pretoria.

– De Beer, M. 1991. Use of the Dynamic Cone Penetrometer (DCP) in the design of road structures. In: Proceedings of the Regional conference for Africa on soil mechanics and foundation engineering, 10th and the International conference on tropical and residual soils, 3rd, Maseru, September 1991.

– Ebels LJ, and Jenkins KJ. Characterisation of Bitumen Stabilised Granular Pavement Material Properties using Triaxial Testing. International Conference on Advanced Characterisation of Pavement and Soil Engineering Materials ICACPSEM, Athens, Greece, June 2007

– Flexible pavement rehabilitation investigation and design. Pretoria: Committee of State Road Authorities (CSRA), Department of Transport (DoT). (DoT technical recommendations for highways; draft TRH12). South Africa 1996

– Horak, E. Aspects of deflection basin parameters used in mechanistic rehabilitation design procedures for flexible pavement in South Africa. PhD Thesis, University of Pretoria, Pretoria, 1988.

– Interim Report – Construction completion for cold-in-place recycling, Placer 80 PM 14.3/33.3,

California Department of Transportation (Caltrans), North Region Materials, Marysville, Ca, August 2006

– Jenkins, K.J. Analysis of a Pavement Layer which has been treated by Single Pass In Situ Stabilisation, Masters Degree Thesis. University of Natal, South Africa, 1994

– Jenkins, K.J. Mix Design Considerations for Cold and Half-warm Bituminous Mixes with emphasis on Foamed Bitumen. PhD Dissertation, University of Stellenbosch, South Africa, 2000 .

– Jenkins K.J. and van de Ven M.F.C. Comparisons between In Situ Recycling with Cement and Foamed Bitumen or Emulsion on Vanguard Drive in South Africa. First International Symposium on Subgrade Stabilization and In Situ Pavement Recycling using Cement ,Salamanca, Spain. 2001.

– Jenkins KJ, van de Ven MFC, Molenaar AAA and de Groot JLA, Performance Prediction of Cold Foamed Bitumen Mixes. Ninth International Conference on Asphalt Pavements, Copenhagen, Denmark, 2002.

– Jenkins K.J. Collings D.C. and Jooste F.J. TG2: The Design and Use of Foamed Bitumen Treated Materials. Shortcomings and Imminent Revisions. Recycling and Stabilisation Conference, NZIHT, Auckland, New Zealand, June 2008.

– Jooste, J.P. The measurement of deflection and curvature of road surfaces. CSIR Manual K16, National Institute for Road Research, CSIR, Pretoria, 1970.

– Jordaan, G.J. Towards improved procedures for the mechanistic analysis of cement-treated layers in pavements. Proceedings of the 7th International Conference on Asphalt Pavements, Nottingham, England, 1992.

– Jordaan, G.J. Pavement rehabilitation design based on pavement layer component tests (CBR and DCP). Research Report 91/241, Department of Transport, Pretoria, 1994.

– Kleyn, E.G. and Savage, P.F. The application of the pavement DCP to determine the bearing properties and performance of road pavements. Proceedings of the International Symposium on Bearing Capacity of Roads and Airfields. Trondheim, Norway, 1982.

– Loizos, A. and Papavasiliou, V. In situ Characterization of Pavement Materials Stabilised with Foamed Asphalt and Cement. International Conference on Advanced Characterisation of Pavement and Soil Engineering Materials ICACPSEM 2007, Athens, Greece

– Long, F,M. and Jooste, F.J. Summary of LTPP Emulsion and Foamed Bitumen Treated Sections. Technical memorandum compiled on behalf of SABITA and GDPTRW. Modelling and Analysis Systems, Cullinan, South Africa. (Gautrans report: CSIR/BE/ER/2007/0006/B)

– Long, F,M. and Jooste, F.J. A Materials Classification and Knowledge Based Structural Design Method for Pavements with Bituminous Stabilised Materials. Conference for Asphalt Pavements in Southern Africa CAPSA 2007, Gaberone, Botswana 2007

– Long F.M. Validation of Material Classification System and Pavement Number Method. Technical Memorandum. Modelling and Analysis Systems, Cullinan. Sabita/Gauteng Department of Public Transport, Road and Works, GDPTRW report no. CSIR/BE/IE/ER/2009/0028/C. Pretoria 2009

– Mathaniya ET, Jenkins KJ, and Ebels LJ. Characterisation of Fatigue Performance of Selected

Cold Bituminous Mixes. International Conference on Asphalt Pavements ICAP, Quebec, Canada, August 2006,

– Mulusa W.K., and Jenkins K.J., Characterization of Cold Recycling Mixes using a Simple Triaxial Test. 6th International Conference of Maintenance and Rehabilitation of Pavements and Technological Control. Turin, Italy, July 2009.

– Paige-Green, P., Netterberg F. And Sampson L. The carbonation of chemically stabilised road construction materials: Guide to its avoidance. Project Report PR 89/146/1. Council for Scientific and Industrial Research CSIR Transportek, South Africa, 1990.

– Paige-Green P. and Ventura D. Durability of Foamed Bitumen Treated Basalt Base Courses. Council for Scientific and Industrial Research, Transportek Division. Contract Report: CR-2004/08, Pretoria, South Africa. 2004

– Patrick, J. and Moorthy, H. Quantifying the Benefits of Waste Minimisation in Road Construction. Recycling and Stabilisation Conference, New Zealand Institute of Highway Technology, June 2008, Takapuna Beach, Auckland, New Zealand

– Technical Guidelines TG2, Second Edition, Bitumen Stabilised Materials. A Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials. ISBN 978-0-7988-5582-2, Asphalt Academy, Pretoria, South Africa. May 2009

Note: This publication is available as a free download on the Asphalt Academy website: www.asphaltacademy.co.za

– Twagira E.M., and Jenkins K.J., Age Hardening Behaviour of Bituminous Stabilised Materials. 7th International RILEM Symposium ATCBM09. Rhodes Greece, May 2009.

– Twagira E.M., and Jenkins K.J., Moisture Damage of Bituminous Stabilised Materials using a MIST Device. 7th International RILEM Symposium ATCBM09. Rhodes Greece, May 2009.

– Theyse, H.L., De Beer, M. & Rust, F.C. 1996. Overview of the South African mechanistic design analysis method. In: 75th Transportation Research Board (TRB) meeting. January 7-11, 1996.

– van Niekerk, A.A. and Hurman, M. Establishing Complex Behaviour of Unbound Road Building Materials from Simple Material Testing, Report, Delft University of Technology, Netherlands, 1995.

– van Niekerk, A.A., van Scheers, J., and Galjaard, P.J. Resilient Deformation Behaviour of Coarse Grained Mix Granulate Base Course Materials from Testing Scaled Gradings at Smaller Specimen Sizes. UNBAR 5 Conference, University of Nottingham, 2000.

– van Niekerk, A.A., Mechanical Behaviour and Performance of Granular Bases and Sub-bases in Pavements. Doctoral Dissertation. Delft University of Technology, The Netherlands. 2001.

Appendix 1 – Laboratory procedures for stabilised materials

This appendix includes detailed procedures for carrying out stabilisation mix designs in a laboratory. Three mix design procedures are described:

Section A1.1:	Cement stabilisation (applicable also to stabilisation with hydrated lime)
Section A1.2:	Bitumen stabilisation using foamed bitumen
Section A1.3:	Bitumen stabilisation using bitumen emulsion

Each section contains all the necessary steps and descriptions for carrying out the relevant mix design, including sample preparation, specimen manufacture, curing and testing procedures. In addition, all formulae required for each mix design are included in the relevant section, thereby eliminating any need to look any further for information. Furthermore, to make these procedures user-friendly and a truly "one-stop" source of information, an explanation is provided at the end of each section for interpreting the various test results.

Two sections on quality control testing for BSMs are included:

Section A1.4:	Testing field samples of bitumen stabilised material (BSMs)
Section A1.5:	Determining the strength of BSM core specimens

Finally, Section A1.6 includes comprehensive laboratory equipment lists for:

Section A1.6.1	Soils testing (basic laboratory equipment for all types of stabilisation)
Section A1.6.2	Additional items for cement (or lime) stabilisation
Section A1.6.3	Additional items for bitumen stabilisation

A1.1 Mix design procedure for cement stabilised materials

A1.1.1 Sampling and preparation

A1.1.1.1 Field sampling

Bulk samples are obtained from test pits excavated as part of the field investigations (or from borrow pits and quarries where fresh materials are to be imported and stabilised). Each layer in the upper pavement (± 300 mm) must be sampled separately and at least 100 kg of material recovered from each layer that is likely to be included in any recycling operation and will therefore require a mix design

Note:

- Samples taken from layers of bound material (asphalt and previously stabilised materials) should be pulverised in situ using a small milling machine (or a recycler) to simulate the grading that will be achieved when the pavement is recycled.

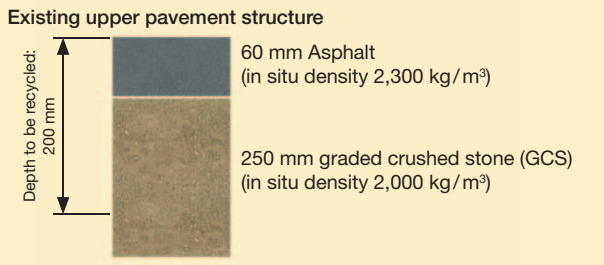
A1.1.1.2 Standard soil tests

Carry out the following standard tests on the material sampled from each individual layer or source:

- Sieve analysis to determine the grading (fines washing procedure, ASTM D 422);
- Atterberg limits to determine the plasticity index (ASTM D 4318); and
- Moisture/density relationship (AASHTO T-180).

A1.1.1.3 Sample blending

Where necessary, blend the materials sampled from the different layers (and/or new material) to obtain a combined sample representing the material from the full recycling depth. The in-situ density of the various component materials must be considered when blending materials, as illustrated in the example shown below.



Blend the materials in proportion to layer thickness and in situ density as follows:

Material	Mass/m² (kg)	Proportion by mass (%)	Per 10 kg sample (g)
Asphalt (60 mm at 2,300 kg/m³)	0.06 x 2,300 = 138	138/418 = 0.33	0.33 x 10,000 = 3,300
GCS (140 mm at 2,000 kg/m³)	0.14 x 2,000 = 280	280/418 = 0.67	0.67 x 10,000 = 6,700
Total	418	1.00	10,000

Note:

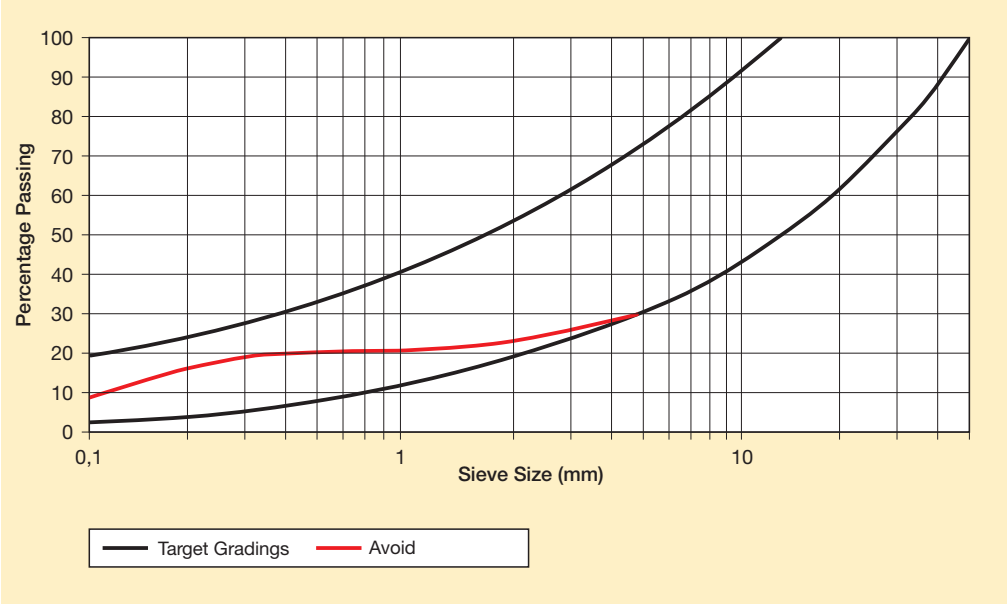
- Repeat the standard soil tests listed above to determine the grading, plasticity index and the moisture/density relationship of the blended sample.

A1.1.1.4 Gradings (sieve analyses)

Plot the grading curve for the sample that will be used in the mix designs. Include on the graph the "Recommended gradings" envelop from the table below. This plot will indicate whether additional blending with freshly imported material may be required. However, if the plot includes a "bulge" in the fractions between the 0.075 mm and 2.0 mm sieves (as shown by the red line entitled "Avoid" in the graph below), consideration should be given to blending the sample with sufficient suitable fine material (e.g. 10% by volume of minus 5 mm crusher dust) to reduce the magnitude of the bulge.

Note:

- This exercise is advisable as it allows a preliminary indication to be made of the strength that can be expected after the material has been treated with cement. (A poorly graded material is difficult to compact and the consequent low density achieved will significantly affect the strength of the stabilised material.)



Recommended grading curves

Sieve size (mm)	Recommended gradings	
	Percent passing (%)	
	Coarse	Fine
50	100	100
37.5	85	100
26.5	72	100
19	60	100
13.2	50	100
9.5	42	90
6.7	35	80
4.75	30	72
2.36	21	56
1.18	14	44
0.6	9	35
0.425	7	31
0.3	5	27
0.15	3	21
0.075	2	18

A1.1.1.5 Representative proportioning

Separate the material in the prepared bulk sample into the following four fractions:

i. Retained on the 19.0 mm sieve;
ii. Passing the 19.0 mm sieve, but retained the 13.2 mm sieve;
iii. Passing the 13.2 mm sieve, but retained on the 4.75 mm sieve; and
iv. Passing the 4.75 mm sieve.

Reconstitute representative samples in accordance with the grading determined above (for the bulk sample) for the portion passing the 19.0 mm sieve. Substitute the portion retained on 19.0 mm sieve with material that passes the 19.0 mm sieve and retained on the 13.2 mm sieve. The example in the table below explains this procedure:

Sieve analysis		Quantity of material to be included in a 10 kg sample		
Sieve size (mm)	Percentage passing (from sieve analysis on bulk sample)	Passing 4.75 mm	Passing 13.2 mm and retained on 4.75 mm	Passing 19.0 mm and retained on 13.2 mm
19.0	90.5	(53.6 / 100 x 10,000) = 5,360 g	((72.3-53.6) / 100 x 10,000) = 1,870 g	((100-72.3) / 100 x 10,000) = 2,770 g
13.2	72.3			
4.75	53.6			

If there is insufficient material passing the 19.0 mm sieve but retained on the 13.2 mm sieve for substituting that retained on the 19 mm sieve, then lightly crush the material retained on the 19.0 mm sieve to provide more of this fraction.

A1.1.1.6 Hygroscopic moisture content

Two representative air-dried samples, each approximately 1 kg, are used to determine the hygroscopic (air-dried) moisture content of the material. (Note: Larger sample size should be used for more coarsely-graded materials.) Weigh the air-dried samples, accurate to the nearest 0.1 g, and then place them in an oven at a temperature of between 105°C and 110°C until they achieve constant mass. The hygroscopic moisture content ($W_{air\text{-}dry}$) is the loss of mass expressed as a percentage of the dry mass of the sample. Determine the hygroscopic moisture using Equation A1.1.1.

$$W_{air\text{-}dry} = \frac{(M_{air\text{-}dry} - M_{dry})}{M_{dry}} \times 100 \qquad \text{[equation A1.1.1]}$$

where:

$W_{air\text{-}dry}$ = hygroscopic moisture content [% by mass]
$M_{air\text{-}dry}$ = mass of air-dried material [g]
M_{dry} = mass of oven-dried material [g]

A1.1.1.7 Sample quantities

The guidelines shown in the following table should be used to estimate the quantity of material required for the respective tests:

Test	Mass of sample required (kg)
Moisture/density relationship (modified AASHTO T180)	40
Optimum cement addition determination (150 mm Ø specimens)	120
Standard soil tests (gradings, Atterberg Limits, moisture content, etc.)	20

A1.1.2 Determination of the moisture/density relationship of treated material

This test is carried out using standard compaction effort to determine the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of the cement stabilised material.

Step 1. Weigh out the required mass of stabilising agent for each of five 7 kg samples prepared as described in Section A1.1.1. The amount of stabilising agent required (expressed as a percentage by mass of the dry sample) should be close to the anticipated optimum for the material being treated. In the absence of previous tests, the following can be used as a guideline:

Subbase layers:	2% for coarse material	(> 50% retained on 4.75 mm sieve)
	3% for fine material	(< 50% retained on 4.75 mm sieve)
Base layers:	3% for coarse material	(> 50% retained on 4.75 mm sieve)
	4% for fine material	(< 50% retained on 4.75 mm sieve)

Step 2. Add the stabilising agent to the raw material and mix immediately prior to the addition of water. In order to simulate conditions on the road, compaction of the stabilised material is delayed for one hour after mixing the untreated material with stabilising agent and water. The mixed material is placed in an air-tight container to prevent loss of moisture and is thoroughly mixed every fifteen minutes.

Step 3. Determine the OMC and MDD for the stabilised material in accordance with the modified moisture-density relationship test procedure (AASHTO T-180).

A1.1.3 Manufacture of specimens for testing

The procedure described below is for the manufacture specimens that are of 150 mm in diameter and 127 mm in height. These specimens will be used to determine the Unconfined Compressive Strength (UCS) and Indirect Tensile Strength (ITS) of the material.

Step 1. Place 20 kg of sample, prepared as described in Section A1.1.1, into a suitable mixing container.

Step 2. Determine the dry mass of the sample using equation A1.1.2.

$$M_{sample} = \frac{(M_{air\text{-}dry})}{\left(1 + \left(\frac{W_{air\text{-}dry}}{100}\right)\right)}$$

[equation A1.1.2]

where:

M_{sample}	= dry mass of the sample	[g]
$M_{air\text{-}dry}$	= air-dried mass of the sample	[g]
$W_{air\text{-}dry}$	= moisture content of air-dried sample	[% by mass]

Step 3. Determine the required amount of stabilising agent using equation A1.1.3.

$$M_{cement} = \frac{C_{add}}{100} \times M_{sample}$$

[equation A1.1.3]

where:

M_{cement}	= mass of lime or cement to be added	[g]
C_{add}	= percentage of lime or cement required	[% by mass]
M_{sample}	= dry mass of the sample	[g]

Step 4. Determine the percentage water to be added for optimum mixing purposes using equation A1.1.4.

$$W_{add} = W_{OMC} - W_{air\text{-}dry}$$ [equation A1.1.4]

where:

W_{add}	= water to be added to sample	[% by mass]
W_{OMC}	= optimum moisture content	[% by mass]
$W_{air\text{-}dry}$	= water in air-dried sample	[% by mass]

The amount (mass) of water to be added to the sample is determined using Equation A1.1.5.

$$M_{water} = \frac{W_{add}}{100} \times (M_{sample} + M_{cement})$$ [equation A1.1.5]

where:

M_{water}	= mass of water to be added	[g]
W_{add}	= water to be added to sample (from equation A1.1.4)	[% by mass]
M_{sample}	= dry mass of the sample	[g]
M_{cement}	= mass of lime or cement to be added	[g]

Step 5. Mix the material, cement and water until uniform. Allow the mixed material to stand for one hour with occasional mixing (as described in Section A1.1.2). Manufacture three specimens, each 150 mm diameter and 127 mm in height, using modified AASHTO (T-180) compaction effort.

Step 6. Samples are taken during the compaction process and dried to a constant mass to determine the moulding moisture content (Wmould) using equation A1.1.6.

$$W_{mould} = \frac{(M_{moist} - M_{dry})}{M_{dry}} \times 100$$ [equation A1.1.6]

where:

W_{mould}	= moulding moisture content	[% by mass]
M_{moist}	= mass of moist material	[g]
M_{dry}	= mass of dry material	[g]

Steps 7 to 9. Repeat the above steps for at least three different stabiliser contents

Step 10. Remove the specimens from the moulds either by dismantling the split moulds or, if ordinary moulds are used, extruding the specimens carefully with an extrusion jack, avoiding distortion to the compacted specimens.

Step 11. Record the mass and volume of each specimen and determine the dry density using equation A1.1.7.

$$DD = \frac{(M_{spec})}{Vol} \times \frac{100}{W_{mould} + 100} \times 1.000$$

[equation A1.1.7]

where:

DD	= dry density	[kg/m^3]
M_{spec}	= mass of specimen	[g]
Vol	= volume of specimen	[cm^3]
W_{mould}	= moulding moisture content	[%]

Note:

- With certain materials lacking cohesion, it may be necessary to leave the specimens in the moulds for 24 hours to develop strength before extracting. When this is necessary, the specimens in the moulds should be kept in a curing room or covered with damp cloth (hessian).

A1.1.4 Curing the specimens

A1.1.4.1 Standard curing

Cure the specimens for seven days at 95% to 100% relative humidity and at a temperature of 20°C to 25°C in a suitable curing room.

A1.1.4.2 Accelerated curing

Place each specimen in sealed plastic bags and cure in an oven at 70°C to 75°C for 24 hours. (Note. If hydrated lime is substituted for cement as the stabilising agent, the curing regime must be changed to 60°C to 62°C for 45 hours.)

After the curing period, remove the specimens from the curing room (or plastic bags) and allow to cool to ambient temperature, if necessary. Specimens for unconfined compressive strength (UCS) tests should be submerged in water at 22°C to 25°C for four hours prior to testing.

A1.1.5 Strength tests

After curing, two of the specimens are tested to determine the Unconfined Compressive Strength (UCS) and the remaining third specimen is tested for Indirect Tensile Strength (ITS). The test procedures are described below.

A1.1.5.1 Unconfined compressive strength (UCS) test

The Unconfined Compressive Strength is determined by measuring the ultimate load to failure of a specimen subjected to a constant loading rate of 140 kPa/s (153 kN/min). The procedure is as follows:

UCS specimen in press after reaching peak load

Step 1. Place the specimen on its flat side between the plates of the compression testing machine. Position the specimen such that it is centred on the loading plates.

Step 2. Apply the load to the specimen, without shock, at a rate of advance of 140 kPa/s until the maximum load is reached. Record the maximum load P (in kN), accurate to 0.1 kN.

Step 3. Calculate the UCS for each specimen to the nearest 1 kPa using equation A1.1.8.

$$UCS = \frac{(4 \times P)}{(\pi \times d^2)} \times 1{,}000{,}000$$

[equation A1.1.8]

where:

UCS	= unconfined compressive strength	[kPa]
P	= maximum load to failure	[kN]
d	= diameter of the specimen	[mm]

Step 4. Plot a graph of the UCS strengths achieved against the percentage stabilising agent added using the average UCS for the two specimens tested for each different stabiliser content. Ignore any obvious incorrect result that may have been caused by damage to the specimen before testing.

A1.1.5.2 Indirect tensile strength (ITS) test

The ITS of a specimen is determined by measuring the ultimate load to failure applied to the diametrical axis at a constant deformation rate of 50.8 mm/minute. Cured specimens are tested (unsoaked) at a temperature of 25°C (± 2°C) using the following procedure:

ITS specimen placed between loading strips

Step 1. Place the specimen onto the ITS jig. (Ensure the correct loading strips are appropriate for the diameter of the specimen.) Position the sample such that the loading strips are parallel and centred on the vertical diametrical plane.

Step 2. Place the transfer plate on the top bearing strip and position the jig assembly centrally under the loading ram of the compression testing device.

Step 3. Apply the load to the specimen, without shock, at a rate of advance of 50.8 mm per minute until the maximum load is reached.

Step 4. Record the maximum load P (in kN), accurate to 0.1 kN.

Step 5. Calculate the ITS value for each specimen to the nearest 1 kPa using Equation A1.2.13.

$$ITS = \frac{2 \times P}{\pi \times h \times d} \times 1{,}000{,}000$$ [equation A1.1.9]

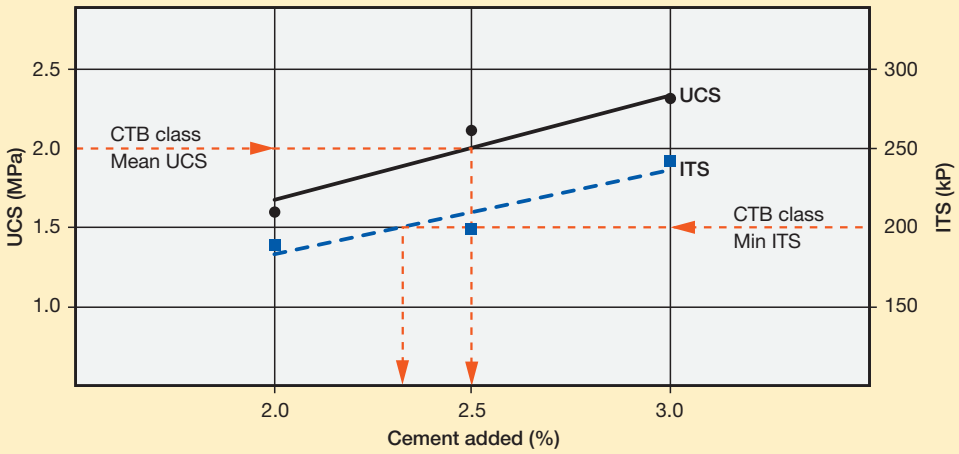
where:

ITS	= Indirect Tensile Strength	[kPa]
P	= maximum applied load	[kN]
h	= average height of the specimen	[mm]
d	= diameter of the specimen	[mm]

Step 6. Plot a graph of the ITS strength achieved against the percentage stabilising agent added.

A1.1.6 Determine the required application of stabilising agent

The required application rate of stabilising agent is that percentage at which the minimum required criteria are met.



The example in the above graph indicates that a cement addition of 2.5% will satisfy the requirements for the specific class shown for the cement stabilised material.

A1.2 Mix design procedure for foamed bitumen stabilisation

The mix design procedures for foamed bitumen stabilisation described below are undertaken on representative samples of material for the following primary objectives:

- To determine whether the material is suitable for stabilising with foamed bitumen;
- To determine whether an active filler needs to be added in conjunction with foamed bitumen;
- To determine the amounts of foamed bitumen and active filler that need to be applied for effective stabilisation; and
- To obtain an indication of the behaviour (engineering properties) of the stabilised material.

The various tests that are carried out on both untreated and treated samples are essentially “routine tests” that can be undertaken by most laboratories equipped for normal routine soils and asphalt testing.

A1.2.1 Sampling and preparation

A1.2.1.1 Field sampling

Bulk samples are obtained from test pits excavated as part of the field investigations (or from borrow pits and quarries where fresh materials are to be imported and stabilised). Each layer in the upper pavement (± 300 mm) must be sampled separately and at least 200 kg of material recovered from each layer that is likely to be included in any recycling operation and will therefore require a mix design.

Note:

- Samples taken from layers of bound material (asphalt and previously stabilised materials) should be pulverised in situ using a small milling machine (or a recycler) to simulate the grading that will be achieved when the pavement is recycled.

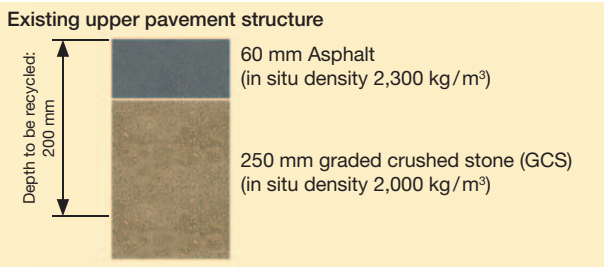
A1.2.1.2 Standard soil tests

Carry out the following standard tests on the material sampled from each individual layer or source:

- Sieve analysis to determine the grading (fines washing procedure, ASTM D 422);
- Atterberg limits to determine the plasticity index (ASTM D 4318); and
- Moisture/density relationship (AASHTO T-180).

A1.2.1.3 Sample blending

Where necessary, blend the materials sampled from the different layers (and/or new material) to obtain a combined sample representing the material from the full recycling depth. The in-situ density of the various component materials must be considered when blending materials, as illustrated in the example shown below.



Blend the materials in proportion to layer thickness and in situ density as follows:

Material	Mass/m² (kg)	Proportion by mass (%)	Per 10 kg sample (g)
Asphalt (60 mm at 2,300 kg/m³)	0.06 x 2,300 = 138	138/418 = 0.33	0.33 x 10,000 = 3,300
GCS (140 mm at 2,000 kg/m³)	0.14 x 2,000 = 280	280/418 = 0.67	0.67 x 10,000 = 6,700
Total	418	1.00	10,000

Note:

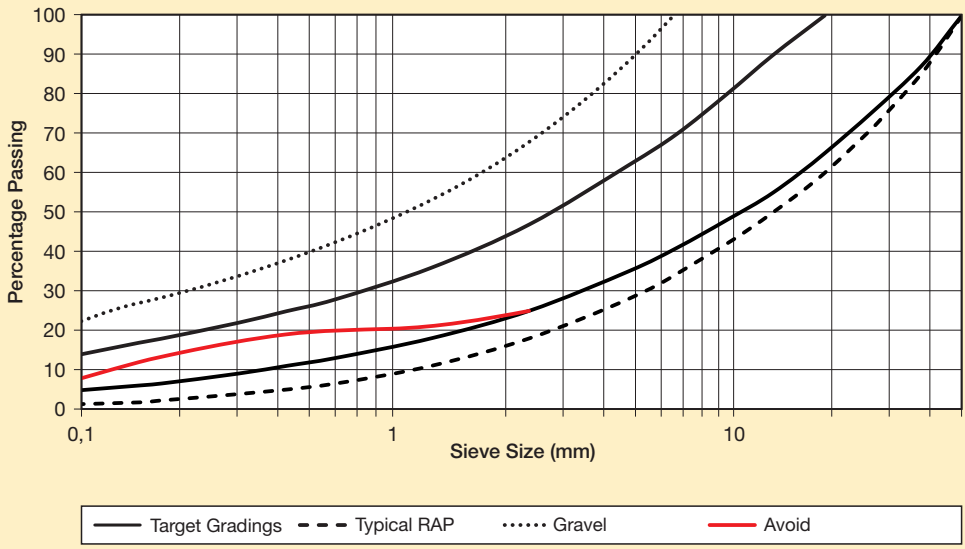
- Repeat the standard soil tests listed in Section A1.2.1.2 above to determine the grading, plasticity index and the moisture/density relationship of the blended sample.

A1.2.1.4 Gradings (sieve analyses)

Plot the grading curve for the sample that will be used in the mix designs. Include on the graph the "Target gradings" envelop from the table below. If the material is predominantly RAP or Natural Gravel, include the relevant curve (also from the table below). This plot will indicate whether additional blending with freshly imported material may be required. However, if the plot includes a "bulge" in the fractions between the 0.075 mm and 2.0 mm sieves (as shown by the red line entitled "Avoid" in the graph below), the sample should be blended with a sufficient suitable fine material (e.g. 10% by volume of minus 5 mm crusher dust) to reduce the magnitude of the bulge.

Note:

- This exercise is advisable as it allows a preliminary indication to be made of the strength that can be expected after the material has been treated with foamed bitumen. (A poorly graded material is difficult to compact and the consequent low density achieved will significantly affect the strength, especially under saturated conditions.)



Recommended grading curves

Sieve size (mm)	Percentage passing each sieve size (%)			
	Target gradings		Typical RAP material	Natural gravel
	Coarse	Fine		
50	100	100	100	100
37.5	87	100	85	100
26.5	76	100	72	100
19	65	100	60	100
13.2	55	90	50	100
9.5	48	80	42	100
6.7	41	70	35	100
4.75	35	62	28	88
2.36	25	47	18	68
1.18	18	36	11	53
0.6	13	28	7	42
0.425	11	25	5	38
0.3	9	22	4	34
0.15	6	17	2	27
0.075	4	12	1	20

A1.2.1.5 Representative proportioning

Separate the material in the prepared bulk sample into the following four fractions:

i. Retained on the 19.0 mm sieve;
ii. Passing the 19.0 mm sieve, but retained the 13.2 mm sieve;
iii. Passing the 13.2 mm sieve, but retained on the 4.75 mm sieve; and
iv. Passing the 4.75 mm sieve.

Reconstitute representative samples in accordance with the grading determined above (for the bulk sample) for the portion passing the 19.0 mm sieve. Substitute the portion retained on 19.0 mm sieve with material that passes the 19.0 mm sieve and retained on the 13.2 mm sieve. The example in the table below explains this procedure:

Sieve analysis		Quantity of material to be included for every 10 kg of sample		
Sieve size (mm)	Percentage passing (from sieve analysis on bulk sample)	Passing 4.75 mm	Passing 13.2 mm and retained on 4.75 mm	Passing 19.0 mm and retained on 13.2 mm
19.0	90.5	(53.6/100 x 10,000) = 5,360 g	((72.3-53.6)/100 x 10,000) = 1,870 g	((100-72.3)/100 x 10,000) = 2,770 g
13.2	72.3			
4.75	53.6			

If there is insufficient material passing the 19.0 mm sieve but retained on the 13.2 mm sieve for substituting that retained on the 19 mm sieve, then lightly crush the material retained on the 19.0 mm sieve to provide more of this fraction.

A1.2.1.6 Hygroscopic moisture content

Two representative air-dried samples, each approximately 1 kg, are used to determine the hygroscopic (air-dried) moisture content of the material. (Note: Larger sample size should be used for more coarsely-graded materials.) Weigh the air-dried samples, accurate to the nearest 0.1 g, and then place them in an oven at a temperature of between 105°C and 110°C until they achieve constant mass. The hygroscopic moisture content (Wair-dry) is the loss of mass expressed as a percentage of the dry mass of the sample. Determine the hygroscopic moisture using Equation A1.2.1.

$$W_{air\text{-}dry} = \frac{(M_{air\text{-}dry} - M_{dry})}{100} \times 100$$

[equation A1.2.1]

where:

$W_{air\text{-}dry}$ = hygroscopic moisture content [% by mass]
$M_{air\text{-}dry}$ = mass of air-dried material [g]
M_{dry} = mass of oven-dried material [g]

A1.2.1.7 Sample quantities

The guidelines shown in the following table should be used to estimate the quantity of material required for the respective tests:

Test	Mass of sample required (kg)
Moisture/density relationship (modified AASHTO T180)	40
Determination of active filler requirement (100 mm Ø specimens)	60
Optimum bitumen addition indication (100 mm Ø specimens)	80
Optimum bitumen addition determination (150 mm Ø specimens)	100
Standard soil tests (gradings, Atterberg Limits, moisture content, etc.)	20

A1.2.2 Active filler requirements

A1.2.2.1 Effect of plasticity

Foamed bitumen stabilisation is normally carried out in combination with a small amount (1% by mass) of active filler (cement or hydrated lime) to enhance the dispersion of the bitumen and reduce moisture susceptibility. The Plasticity Index (PI) of the material is normally used as a guideline for the use of hydrated lime or cement in the mix:

Plasticity Index: < 10	Plasticity Index: > 10
Carry out ITS tests on 100 mm Ø specimens to determine the need to add either cement or hydrated lime, as described in Section A1.2.2.2 below..	Pre-treat the material with hydrated lime (ICL value) (The initial consumption of lime (ICL value) must first be determined using the appropriate pH test.)

Pre-treatment of material with a PI > 10 requires that the lime and water be added at least 2 hours prior to the addition of the foamed bitumen. The pre-treated material is placed in an air-tight container to retain moisture. The moisture content is then checked and, if necessary, adjusted prior to adding the bitumen stabilising agent (as described in Section A1.2.4).

Note:

- Where the material is pre-treated with lime, the following tests for the “Determination of Active Filler Requirements” described under Section A1.2.2.2 below are not necessary.

A1.2.2.2 Determination of active filler requirements

Where the PI < 10, the need for an active filler and the type of active filler (cement or hydrated lime) that is appropriate for the material must first be determined by carrying out ITS tests on 100 mm diameter specimens for three different mixes made from the same sample. The amount of foamed bitumen added to each of the three mixes is constant, using the fractions passing the 4.75 mm and 0.075 mm sieves as a guideline, as shown in the following table:

Guidelines for estimating optimum foamed bitumen addition			
Fraction passing 0.075 mm sieve (%)	**Foamed bitumen addition (% by mass of dry aggregate)**		**Typical type of material**
	Fraction passing 4.75 mm sieve		
	< 50%	> 50%	
< 4	**2.0**	**2.0**	Recycled asphalt (RA/RAP)
4 – 7	**2.2**	**2.4**	RA/Graded crushed stone/ Natural gravel/blends
7 – 10	**2.4**	**2.8**	
> 10	**2.6**	**3.2**	Gravels/sands

The first of the three mixes contains no active filler, 1% cement is added to the second mix and 1% hydrated lime is added to the third mix, all three mixes being treated with the same amount of foamed bitumen. Material from each of the three mixes is used to manufacture 100 mm diameter specimens that are cured and tested to determine the relevant ITS_{DRY} and ITS_{WET} values (described in Sections A1.2.4 to A1.2.8 below). The Tensile Strength Retained (TSR) value is then used as the primary indicator for whether an active filler is required.

Where the TSR value for the mix with no active filler added is in excess of 60%, the mix design should be undertaken with no active filler. (This situation is usually confined to materials consisting of good quality crushed stone, often including a significant proportion of reclaimed asphalt (RAP) material.)

Where the TSR value of the mix with no active filler added is less than 60%, the mix with the type of active filler that produces a significantly higher TSR value (> 5%) indicates a preference for either cement or hydrated lime and should be used in the following mix designs. If the TSR values for both active fillers are of the same order (difference < 5%) then either type of active filler is suitable.

Note:

- To determine the sensitivity of the active filler, additional tests on 100 mm diameter specimens may be undertaken using the preferred active filler at a lower application rate (e.g. 0.75%). However, to avoid compromising the flexibility of the mix, **the maximum allowable application rate for active filler is 1.0%** and should only be exceeded when hydrated lime is applied as a pre-treatment to eliminate plasticity.

A1.2.3 Determination of the foaming properties of bitumen

The foaming properties of bitumen are characterised by:

- Expansion Ratio. A measure of the viscosity of the foamed bitumen, calculated as the ratio of the maximum volume of the foam relative to the original volume of bitumen; and
- Half Life. A measure of the stability of the foamed bitumen, calculated as the time taken in seconds for the foam to collapse to half of its maximum volume.

The objective of carrying out the following procedure is to determine the bitumen temperature and percentage of water addition that is required to produce the best foam properties (maximum expansion ratio and half-life) for a particular source of bitumen. These properties are measured at three different bitumen temperatures in the range of 160° C to 190° C using the following procedure:

Step 1. Heat the bitumen in the kettle of the Wirtgen WLB 10 S laboratory unit with the pump circulating the bitumen through the system until the required temperature is achieved (normally starting with 160° C). Maintain the required temperature for at least 5 minutes prior to commencing with testing.

Laboratory plant WLB 10 S

Step 2. Following standard procedures described in the User's Manual for the Wirtgen WLB 10 S, calibrate the discharge rate of bitumen ($Q_{bitumen}$) and set the timer on the unit to discharge 500 g of bitumen.

Step 3. Set the water flow-meter to achieve the required water injection rate.

Step 4. Discharge foamed bitumen into a preheated (± 75° C) steel drum for a calculated spray time for 500 g of bitumen. Immediately after the foam discharge stops, start a stopwatch.

Step 5. Using the dipstick supplied with the Wirtgen WLB 10 S (which is calibrated for a steel drum of 275 mm in diameter and 500 g of bitumen) measure the maximum height the foamed bitumen achieves in the drum. This is recorded as the maximum volume.

Step 6. Use a stopwatch to measure the time in seconds that the foam takes to dissipate to half of its maximum volume. This is recorded as the foamed bitumen's half-life.

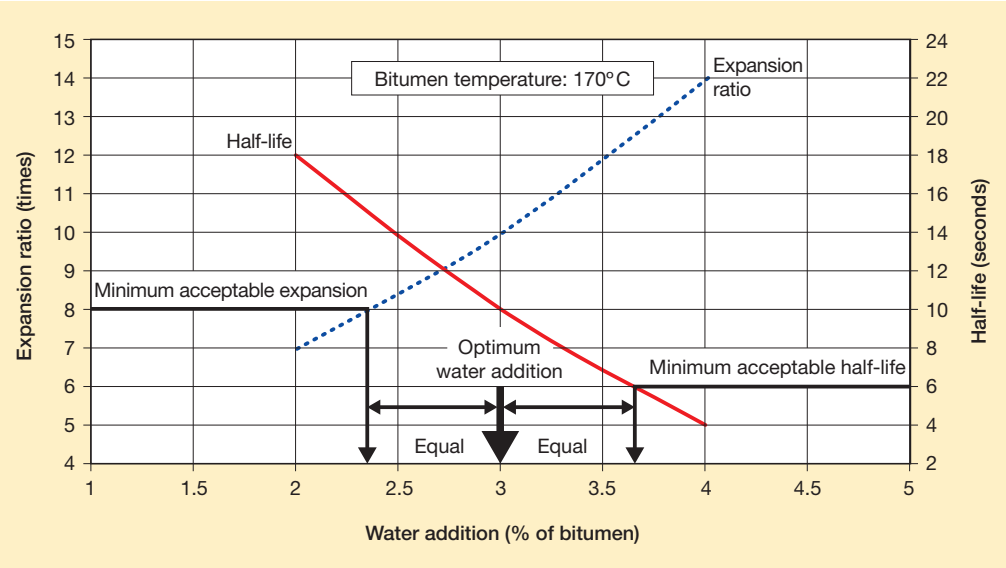
Step 7. Repeat the above procedure three times or until similar readings are achieved.

Step 8. Repeat steps 3 to 7 for a range of at least three water injection rates. (Typically, values of 2%, 3% and 4% by mass of bitumen are used.)

Step 9. Plot a graph of the expansion ratio versus half-life at the different water injection rates on the same set of axes (see the example in the graph below). The optimum water addition is chosen as an average of the two water contents required to meet these minimum criteria.

Repeat Step 1 to 9 for two other bitumen temperatures (normally 170° C and 180° C).

The temperature and optimum water addition that produces the best foam is then used in the mix design procedure described below.



Determination of optimum water addition for foaming (example)

Note:

- The absolute minimum foaming properties that are acceptable for effective stabilisation (material temperature > 15° C) are:

  Expansion ratio: 8 times
  Half-life: 6 seconds

  If these minimum requirements cannot be met, the bitumen should be rejected as unsuitable for use.

A1.2.4 Treating the sample with foamed bitumen

Laboratory plant WLB 10 S coupled to WLM 30 pugmill mixer

Step 1. Place the required mass of sample (between 20 kg and 30 kg, prepared as described in Section A1.2.1 above) into the Wirtgen WLM 30 pug mill mixer.

Step 2. Determine the dry mass of the sample using Equation A1.2.2.

$$M_{sample} = \frac{(M_{air\text{-}dry})}{\left(1 + \left(\frac{W_{air\text{-}dry}}{100}\right)\right)}$$

[equation A1.2.2]

where:

M_{sample}	= dry mass of the sample	[g]
$M_{air\text{-}dry}$	= air-dried mass of the sample	[g]
$W_{air\text{-}dry}$	= moisture content of air-dried sample	[% by mass]

Step 3. Determine the required mass of active filler (lime or cement) to be added using Equation A1.2.3.

$$M_{cement} = \frac{C_{add}}{100} \times M_{sample}$$

[equation A1.2.3]

where:

M_{cement}	= mass of lime or cement to be added	[g]
C_{add}	= percentage of lime or cement required	[% by mass]
M_{sample}	= dry mass of the sample	[g]

Step 4. Determine the percentage water to be added to achieve the ideal mixing moisture content (75% of the OMC of the material), calculated using Equation A1.2.4.

$$W_{add} = 0.75\ W_{OMC} - W_{air\text{-}dry}$$ [equation A1.2.4]

where:

W_{add}	= water to be added to sample	[% by mass]
W_{OMC}	= optimum moisture content	[% by mass]
$W_{air\text{-}dry}$	= water in air-dried sample	[% by mass]

The amount (mass) of water to be added to the sample is determined using Equation A1.2.5.

$$M_{water} = \frac{W_{add}}{100} \times (M_{sample} + M_{cement})$$ [equation A1.2.5]

where:

M_{water}	= mass of water to be added	[g]
W_{add}	= water to be added to sample (from equation A1.2.4)	[% by mass]
M_{sample}	= dry mass of the sample	[g]
M_{cement}	= mass of lime or cement to be added	[g]

Step 5. Mix the material, active filler and water in the mixer until uniform. After mixing, inspect the sample to ensure that the material is in a “fluffed” state.

- If any dust is observed, add small amounts of water (nominally 0.25% each time) and remix until a “fluffed” state is achieved with no dust visible.
- If the material is “sticky” with a tendency to pack against the side of the mixer, then the moisture content is too high for mixing with foamed bitumen. Reject the sample. Start again with a fresh sample using a lower moisture content.

Step 6. Determine the amount of foamed bitumen to be added using Equation A1.2.6.

$$M_{bitumen} = \frac{B_{add}}{100} \times (M_{sample} + M_{cement})$$ [equation A1.2.6]

where:

$M_{bitumen}$	= mass of foamed bitumen to be added	[g]
B_{add}	= foamed bitumen content	[% by mass]
M_{sample}	= dry mass of the sample	[g]
M_{cement}	= mass of lime or cement added	[g]

Step 7. Determine the timer setting on the Wirtgen WLB 10 S using Equation A1.2.7.

$$T = \frac{M_{bitumen}}{Q_{bitumen}}$$ [equation A1.2.7]

where:

T	= time to be set on WLB 10 S timer	[s]
$M_{bitumen}$	= mass of foamed bitumen to be added	[g]
$Q_{bitumen}$	= bitumen flow rate for the WLB10 S	[g/s]

Step 8. Couple the Wirtgen WLB 10 S to the WLM 30 mixer so that the foamed bitumen can be discharged directly into the mixing chamber.

Step 9. Start the mixer and allow it to mix for at least 10 seconds before discharging the required mass of foamed bitumen into the mixer. After the foamed bitumen has discharged, continue mixing for a further 30 seconds or until uniformly mixed.

Step 10. Determine the mass of water required to bring the sample to the OMC using Equation A1.2.8.

$$M_{plus} = \frac{M_{water}}{3} \qquad \text{[equation A1.2.8]}$$

where:

M_{plus} = mass of water to be added [g]

M_{water} = mass of water previously added (equation A1.2.5) [g]

Step 11. Add the additional water and mix until uniform.

Step 12. Transfer the foamed bitumen treated material into an air-tight container and immediately seal. To minimise moisture loss, manufacture the test specimens as soon as possible by following the relevant procedure for either 100 mm or 150 mm diameter specimens, as described in sections A1.2.5.1 and A1.2.5.2 respectively.

Repeat the above steps for at least four mixes with different foamed bitumen contents at 0.2% intervals.
The "Guidelines for estimating optimum foamed bitumen addition" (Section A1.2.2.2 above) should be used in determining the mid-point of the range of foamed bitumen to be added to the four samples.

An example. If the material consists of a blend of RAP and crushed stone with 39% and 8% passing the 4.75 mm and 0.075 mm sieves respectively, the guidelines in Section A1.2.2.2 indicate an optimum bitumen addition of 2.4%. The amount of foamed bitumen to be added to each sample (all with the same amount of active filler and at the same moisture content) is:

Sample 1: 2.1%
Sample 2: 2.3%
Sample 3: 2.5%
Sample 4: 2.7%

A1.2.5 Manufacture of specimens for testing

The procedures described below are for the manufacture of two different sizes of specimen using different compaction procedures:

Specimen size and compaction effort applied in the manufacturing process		
Specimen diameter	**Specimen height**	**Compaction effort**
100 mm	63.5 mm	Modified Marshall*
150 mm	95.0 mm	Modified AASHTO

* 75 blows per face

The following two questions are often raised:

1. **Which specimen size should be manufactured?** As described in Section A1.2.8 below, ITS_{DRY} and ITS_{WET} values are normally determined from 100 mm diameter specimens. 150 mm diameter specimens may be substituted for 100 mm diameter specimens to obtain the same values. However, when dealing with coarse material, (i.e. where the grading curve tends towards the coarse side of the recommended grading envelop) it is strongly recommended that 150 mm diameter specimens are manufactured and tested in place of the smaller 100 mm diameter specimens.
   Note. Only 150 mm diameter specimens are used to determine ITS_{EQUIL} and ITS_{SOAK} values.

2. **Can other compaction methods be used?** The compaction procedures described below are well known standard procedures that can be carried out in most laboratories, worldwide. Other procedures may be used (e.g. gyratory compaction, vibrating hammer, vibrating table, etc.) provided they achieve the same density target of 100% Marshall compaction for the 100 mm diameter specimens or 100% of the mod AASHTO T-180 density for 150 mm diameter specimens.

A1.2.5.1 Manufacture of 100 mm diameter specimens

A minimum of six (6) 100 mm diameter specimens, 63.5 mm in height, are manufactured from each sample of treated material by applying modified Marshall compaction effort, as described in the following steps:

Step 1 Prepare the Marshall mould and hammer by cleaning the mould, collar, base-plate and face of the compaction hammer. Note: the compaction equipment must not be heated but kept at ambient temperature.

Step 2. Weigh sufficient material to achieve a compacted height of 63.5 mm ± 1.5 mm (Approximately 1,100 g for most materials). Poke the mixture with a spatula 15 times around the perimeter and 10 times on the surface, leaving the surface slightly rounded.

Step 3. Compact the mixture by applying 75 blows with the compaction hammer. Care must be taken to ensure the continuous free fall of the hammer.

Step 4. Remove the mould and collar from the pedestal, invert the specimen (turn over). Replace it and press down firmly to ensure that it is secure on the base plate. Compact the other face of the specimen with a further 75 blows.

Step 5. After compaction, remove the mould from the base-plate and extrude the specimen by means of an extrusion jack. Measure the height of the specimen and adjust the amount of material if the height is not within the 1.5 mm limits.

Note: Coarse materials are often damaged during the extrusion process. It is therefore recommended that the specimens are left in their moulds for 24 hours allowing sufficient strength to develop before extruding.

Repeat steps 1 to 5 for the manufacture of at least six (6) specimens.

Step 6. Take ±1 kg representative samples after compaction of the second and fifth specimen and dry to a constant mass. Determine the moulding moisture using Equation A1.2.9.

$$W_{mould} = \frac{(M_{moist} - M_{dry})}{M_{dry}} \times 100$$

[equation A1.2.9]

where:

W_{mould} = moulding moisture content [% by mass]
M_{moist} = mass of moist material [g]
M_{dry} = mass of dry material [g]

A1.2.5.2 Manufacture of 150 mm diameter specimens

A minimum of six (6) 150 mm diameter specimens, 95 mm in height, are manufactured from each sample of treated material by applying modified AASHTO (T-180) compaction effort, as described in the following steps:

Step 1. Prepare the equipment by cleaning the mould, collar, base-plate and face of the compaction rammer. (Either split-moulds or standard "Proctor" moulds may be used, each fitted with a 32 mm spacer placed on the base plate to achieve specimens that are 95 mm (±1.5 mm) in height.)

Note: the modified AASHTO compaction rammer has the following specifications:

Rammer diameter:	50 mm
Mass:	4.536 kg
Drop distance:	457 mm

Step 2. Compact each specimen applying modified AASHTO (T-180) compaction effort (4 layers approximately 25 mm thick, each receiving 55 blows from the drop rammer.)

Step 3. Carefully trim excess material from specimens, as specified in the AASHTO T-180 test method.

Step 4. After compaction, remove the mould from the base-plate and extrude the specimen by means of an extrusion jack. Where split-moulds are used, separate the segments and remove the specimen.

Note: Coarse materials are often damaged during the extrusion process. It is therefore recommended that the specimens are left in their moulds for 24 hours allowing sufficient strength to develop before extruding.

Where split moulds are used, it is advisable to leave the specimen in the mould for 4 hours before splitting the mould and extracting the specimen.

Repeat steps 1 to 4 for the manufacture of at least six (6) specimens.

Step 5. Take ±1 kg representative samples after compaction of the second and fifth specimen and dry to a constant mass. Determine the moulding moisture content using Equation A1.2.9 (above).

A1.2.6 Curing the specimens

Two curing regimes are described below. The first is a standard procedure to dry the specimens to constant mass. The second procedure aims to simulate field conditions where the “equilibrium moisture content” is approximately 50% of OMC. Both 100 mm and 150 mm diameter specimens may be cured dry (to constant mass) whereas only 150 mm diameter specimens are cured at equilibrium moisture content.

A1.2.6.1 Curing dry

Place the specimens (either 100 mm or 150 mm diameter) in a forced-draft oven at 40° C and cure to constant mass (normally 72 hours).

To determine if constant mass has been achieved, weigh the specimens and place back in the oven. Remove after 4 hours and reweigh. If the mass is constant, then continue with testing. If the mass is not constant, place back in the oven and reweigh after repeated 4 hour intervals until constant mass is achieved.

When constant mass is achieved, remove the specimens from the oven and allow to cool to 25° C (± 2.0° C).

A1.2.6.2 Curing to simulate field conditions

Place the 150 mm diameter specimens in a forced-draft oven at 30° C for 20 hours (or until the moisture content has reduced to approximately 50% of OMC).

Take the specimens out of the oven, place each in a sealed plastic bag (at least twice the volume of the specimen) and place back in the oven at 40° C for a further 48 hours.

Remove specimens from the oven after 48 hours and take out of their respective plastic bags, ensuring that any moisture in the bags does not come into contact with the specimen. Allow to cool down to 25° C (± 2.0° C).

A1.2.7 Preparing the specimens for testing

After cooling, determine the bulk density of each specimen using the following procedure:

Step 1. Determine the mass of the specimen.

Step 2. Measure the height of the specimen at four evenly-spaced locations around the circumference and calculate the average height of the specimen.

Step 3. Measure the diameter of the specimen.

Step 4. Calculate the bulk density of the each specimen using Equation A1.2.10.

$$BD_{spec} = \frac{4 \times M_{spec}}{\pi \times d^2 \times h} \times 1{,}000{,}000 \qquad \text{[equation A1.2.10]}$$

where:

BD_{spec}	= bulk density of specimen	[kg/m^3]
M_{spec}	= mass of specimen	[g]
h	= average height of specimen	[mm]
d	= diameter of specimen	[mm]

Exclude from further testing any specimen whose bulk density differs from the mean bulk density of all six (6) specimens by more than 2.5%.

Step 5. Place half of the specimens (normally 3) under water in a soaking bath for 24 hours at 25° C (± 2° C). After 24 hours, remove the specimens from the water, surface dry and test immediately.

A1.2.8 Determination of the indirect tensile strength (ITS) of specimens

Specimen mounted in press for the ITS test

The ITS of a specimen is determined by measuring the ultimate load to failure applied to the diametrical axis at a constant deformation rate of 50.8 mm/minute. Ensure that the temperature of the specimens is 25° C (± 2° C) and follow the procedure described below:

Step 1. Place the specimen onto the ITS jig. (Ensure the correct loading strips are appropriate for the diameter of the specimen.) Position the sample such that the loading strips are parallel and centred on the vertical diametrical plane.

Step 2. Place the transfer plate on the top bearing strip and position the jig assembly centrally under the loading ram of the compression testing device.

Step 3. Apply the load to the specimen, without shock, at a rate of advance of 50.8 mm per minute until the maximum load is reached.

Step 4. Record the maximum load P (in kN), accurate to 0.1 kN.

Step 5. Record the displacement at break to the nearest 0.1 mm.

Step 6. Break the specimen in half and record the temperature of the specimen at its centre.

Step 7. Break up one of the unsoaked specimens and dry to a constant mass. Determine the cured moisture content using Equation A1.2.11.

$$W_{spec} = \frac{(M_{moist} - M_{dry})}{M_{dry}} \times 100$$ [Equation A1.2.11]

where:
W_{spec} = moisture content of specimen [% by mass]
M_{moist} = mass of moist material [g]
M_{dry} = mass of dry material [g]

Determine the dry density of each specimen using Equation A1.2.12.

$$DD_{spec} = BD_{spec} \times \left(1 - \frac{W_{spec}}{100}\right)$$ [Equation A1.2.12]

where:
DD_{spec} = dry density of specimen [% by mass]
BD_{spec} = bulk density of specimen [g]
W_{spec} = moisture content of specimen [g]

Step 8. Break up one of the soaked specimens and dry to a constant mass. Determine the after-soaking moisture content using Equation A1.2.11.

Step 9. Calculate the ITS value for each specimen to the nearest 1 kPa using Equation A1.2.13.

$$ITS = \frac{2 \times P}{\pi \times h \times d} \times 1{,}000{,}000$$ [Equation A1.2.13]

where:

ITS	= Indirect Tensile Strength	[kPa]
P	= maximum applied load	[kN]
h	= average height of the specimen	[mm]
d	= diameter of the specimen	[mm]

Step 10. Use the Worksheet in Annexure A1.2.1 to record the data and the form in Annexure A1.2.2 to report the results.

Step 11. For dry-cured specimens only, calculate the Tensile Strength Retained (TSR) value for the sample using Equation A1.2.14.

$$TSR = \frac{Ave\ ITS_{WET}}{Ave\ ITS_{DRY}} \times 100$$ [Equation A1.2.14]

where:

TSR	= Tensile Strength Retained	[%]
Ave ITS_{WET}	= average ITS_{WET} value	[kPa]
Ave ITS_{DRY}	= average ITS_{DRY} value	[kPa]

Note. To differentiate between the results obtained from the different curing regimes, the terminology shown in the table below should be adopted to avoid any confusion.

Term	Specimen diameter	Curing regime	Moisture content
ITS_{DRY}	100 mm or 150 mm	72 hrs unsealed	<1%
ITS_{WET}		24 hr soak	Saturated
ITS_{EQUIL}	150 mm only	20 hr unsealed, 48 hrs in sealed bag	± 50% of OMC
ITS_{SOAK}		24 hr soak	Semi-saturated

A1.2.9 Interpretation of the indirect tensile strength (ITS) test results

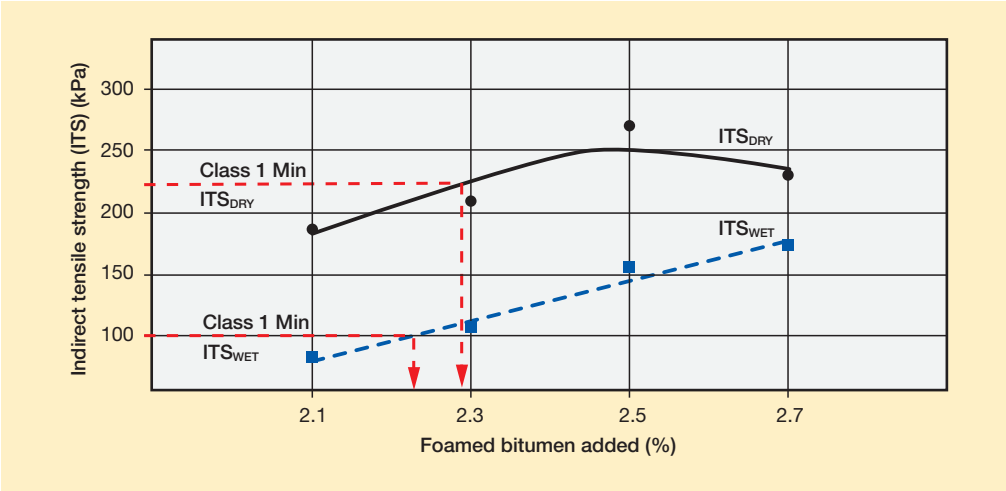
Plot the respective soaked and unsoaked ITS test results against the relevant addition of foamed bitumen, as shown in the example below. If results for two different curing regimes were obtained, each must be plotted on a separate graph.

The added foamed bitumen that meets the minimum ITS value for the required material classification is selected as the primary indicator for the minimum amount of foamed bitumen to be added. Engineering judgement is then used to determine the amount of foamed bitumen that needs to be added in order to achieve sufficient confidence, based on the variability of the test results (the “goodness of fit” of the regression curve or line through the plotted test results).

The example below explains the process.

The table and plot of ITS test results shown below are typical values achieved from a mix design using 100 mm diameter specimens for a natural granular material treated with foamed bitumen. The curve through the four ITS_{DRY} points approximates the relationship between ITS_{DRY} and added foamed bitumen. The line through the four ITS_{WET} points approximates the relationship between ITS_{WET} and added foamed bitumen. The fine dotted lines indicate that the addition of between 2.2% and 2.3% foamed bitumen will meet the requirements for a Class 1 foamed bitumen stabilised material (ITS_{DRY} > 225 kPa and ITS_{WET} > 100 kPa).

Added bitumen (%)	ITS_{DRY} (kPa)	ITS_{SOAK} (kPa)	TSR (%)
2.1	181	86	47.5
2.3	206	109	52.9
2.5	262	152	58.0
2.7	244	169	69.3



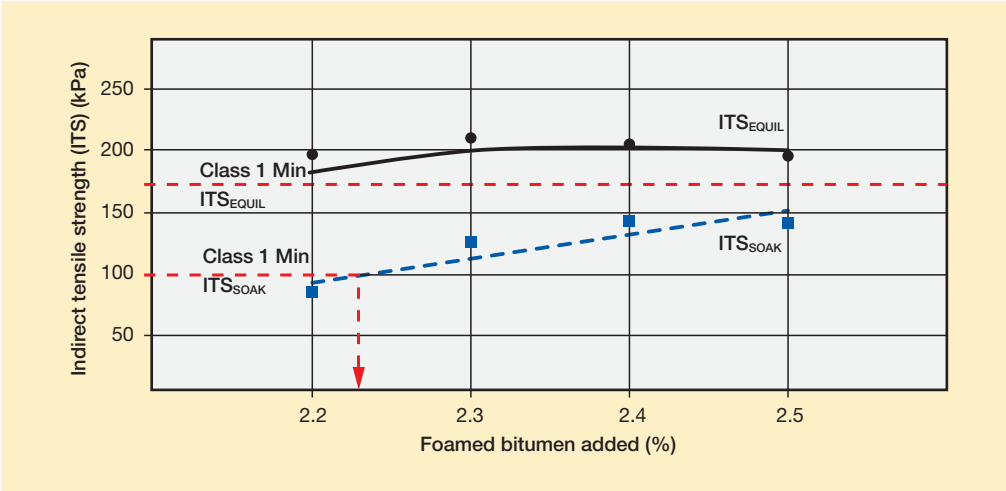
100 mm Ø specimens

The TSR values indicate that the material remains moisture susceptible after foamed bitumen treatment, especially when the application rate is below 2.3% (i.e. TSR value < 50%). It is therefore important to ensure that sufficient bitumen is applied to achieve the minimum ITS_{WET} value required to meet the requirements for Class 1 classification.

Engineering judgement is then applied, based on the understanding that ITS values are not absolute, together with an appreciation that variability must be expected when recycling material from an existing pavement. The results obtained from the mix with 2.3% added foamed bitumen fail to meet the Class 1 requirement for ITS_{DRY} whilst both ITS_{DRY} and ITS_{WET} values for the mix with 2.5% added foamed bitumen are far in excess of the minimum Class 1 requirements. This suggests that a minimum application rate of 2.4% foamed bitumen is required.

To improve on the level of confidence, additional 150 mm diameter specimens can be manufactured at 0.1% intervals of foamed bitumen addition, ranging from 2.2% to 2.5% added foamed bitumen and cured at equilibrium moisture content. (Testing specimens at equilibrium moisture content eliminates the suction forces that elevate ITS_{DRY} values with a consequent reduction in the TSR values.) The table and graph below show typical results that would be achieved, superimposed with the relevant minimum limits for ITS_{EQUIL} and ITS_{SOAK} required for a Class 1 foamed bitumen stabilised material (ITS_{EQUIL} > 175 kPa and ITS_{SOAK} > 100 kPa).

Added bitumen (%)	ITS_{EQUIL} (kPa)	ITS_{SOAK} (kPa)
2.2	198	91
2.3	211	132
2.4	205	148
2.5	195	145



150 mm Ø specimens

This example highlights the improved level of confidence that is achieved in taking the decision to add 2.3% foamed bitumen to ensure that the minimum requirements for a Class 1 bitumen stabilised material are satisfied.

Annexure A1.2.1

FOAMED BITUMEN MIX DESIGN - WORKSHEET

Project _____ Date _____

_____

Sample / Mix No.: _____ Location _____

Material description : _____

Maximum dry density _____ Optimum moisture content

Percentage < 0.075mm _____ Grading: Coarse | Medium | Fine

Plasticity Index _____

Bitumen Source _____ Bitumen type _____

Active Filler Type _____ Filler Source _____

MOISTURE DETERMINATION

		Hygroscopic	Specimen manufacture: Sample 1	Specimen manufacture: Sample 2	After Curing: Dry	After Curing: Soaked
Pan No.						
Mass wet sample + pan	m1					
Mass dry sample + pan	m2					
Mass pan	mp					
Mass moisture	m1-m2 = Mm					
Mass dry sample	m2-mp= Md					
Moisture content	Mm/Mdx100=Mh					

Mass of air-dried sample placed in the mixer (kg) _____

Percentage of water added to sample for mixing: _____ Amount of water added : _____

Percentage water added to sample for compaction _____ Amount of water added : _____

Total percentage water added: _____ Total water added: _____

Foamed bitumen addition (%): _____ Active filler addition (%): _____

Foam water injection rate (%) _____

Temperatures (˚C) Material: _____ Bitumen: _____ Water: _____

SPECIMEN DETAILS

Specimen ID						
Date Moulded						
Date removed from oven						
Date tested						
Diameter (mm)						
Individual height measurements (mm)						
Average height (mm)						
Mass after curing (g)						
Bulk density (kg/m^3)						
Average bulk density						
Dry density (kg/m^3)						

ITS TEST

Specimen condition	Unsoaked (ITS_{DRY} / ITS_{EQUIL})			Soaked (ITS_{WET} / ITS_{SOAK})		
Maximum load (kN)						
Internal temperature (˚C)						
Deformation (mm)						
ITS (kPa)						
Average ITS (kPa)						
TSR (%)						

Annexure A1.2.2

FOAMED BITUMEN MIX DESIGN REPORT (Dry curing)

Project _____ Date _____

Sample number: _____ Location _____

Material description : _____

Maximum dry density _____ Optimum moisture content

Percentage < 0.075mm _____ Grading: Coarse | Medium | Fine

Plasticity Index _____

Bitumen Source _____ Bitumen type _____

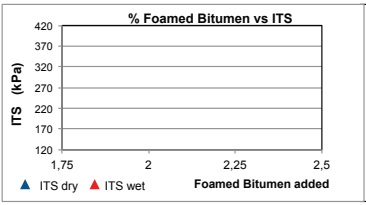
Active Filler Type _____ Filler Source _____

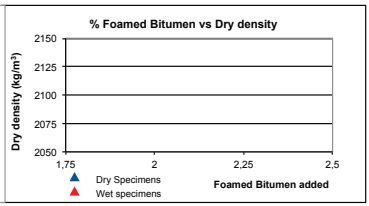
FOAMED BITUMEN STABILISED MATERIAL SPECIMENS

Compactive effort				**mm**	specimen diameter
Date moulded					
Date tested					
Foamed Bitumen added	(%)				
Active filler added	(%)				
Moulding moisture content	(%)				

TEST RESULTS

ITS_{DRY}	(kPa)				
Moisture content at break	(%)				
Dry Density	(kg/m$^{3)}$				
Average deformation	(mm)				
Temperature at break	(˚C)				
ITS_{WET}	(kPa)				
Moisture content at break	(%)				
Dry Density	(kg/m$^{3)}$				
Average deformation	(mm)				
Temperature at break	(˚C)				
Tensile Strength Retained	(%)				
Material classification					





Comments _____

A1.3 Mix design procedure for bitumen emulsion stabilisation

The mix design procedures for bitumen emulsion stabilisation described below are undertaken on representative samples of material for the following primary objectives:

- To determine whether the material is suitable for stabilising with bitumen emulsion;
- To determine whether an active filler needs to be added in conjunction with bitumen emulsion;
- To determine the amounts of bitumen emulsion and active filler that need to be applied for effective stabilisation; and
- To obtain an indication of the behaviour (engineering properties) of the stabilised material.

The various tests that are carried out on both untreated and treated samples are essentially "routine tests" that can be undertaken by most laboratories equipped for normal routine soils and asphalt testing.

A1.3.1 Sampling and preparation

A1.3.1.1 Field sampling

Bulk samples are obtained from test pits excavated as part of the field investigations (or from borrow pits and quarries where fresh materials are to be imported and stabilised). Each layer in the upper pavement (± 300 mm) must be sampled separately and at least 200 kg of material recovered from each layer that is likely to be included in any recycling operation and will therefore require a mix design.

Note:

- Samples taken from layers of bound material (asphalt and previously stabilised materials) should be pulverised in situ using a small milling machine (or a recycler) to simulate the grading that will be achieved when the pavement is recycled.

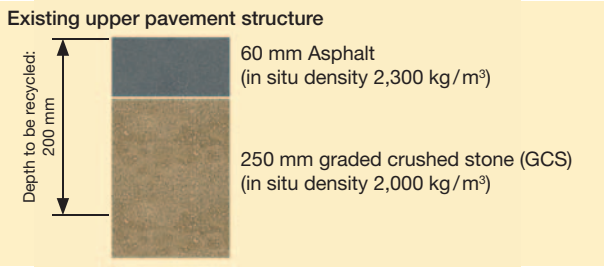
A1.3.1.2 Standard soil tests

Carry out the following standard tests on the material sampled from each individual layer or source:

- Sieve analysis to determine the grading (fines washing procedure, ASTM D 422);
- Atterberg limits to determine the plasticity index (ASTM D 4318); and
- Moisture/density relationship (AASHTO T-180).

A1.3.1.3 Sample blending

Where necessary, blend the materials sampled from the different layers (and/or new material) to obtain a combined sample representing the material from the full recycling depth. The in-situ density of the various component materials must be considered when blending materials, as illustrated in the example shown below.



Blend the materials in proportion to layer thickness and in situ density as follows:

Material	Mass/m² (kg)	Proportion by mass (%)	Per 10 kg sample (g)
Asphalt (60 mm at 2,300 kg/m³)	0.06 x 2,300 = 138	138/418 = 0.33	0.33 x 10,000 = 3,300
GCS (140 mm at 2,000 kg/m³)	0.14 x 2,000 = 280	280/418 = 0.67	0.67 x 10,000 = 6,700
Total	418	1.00	10,000

Note:

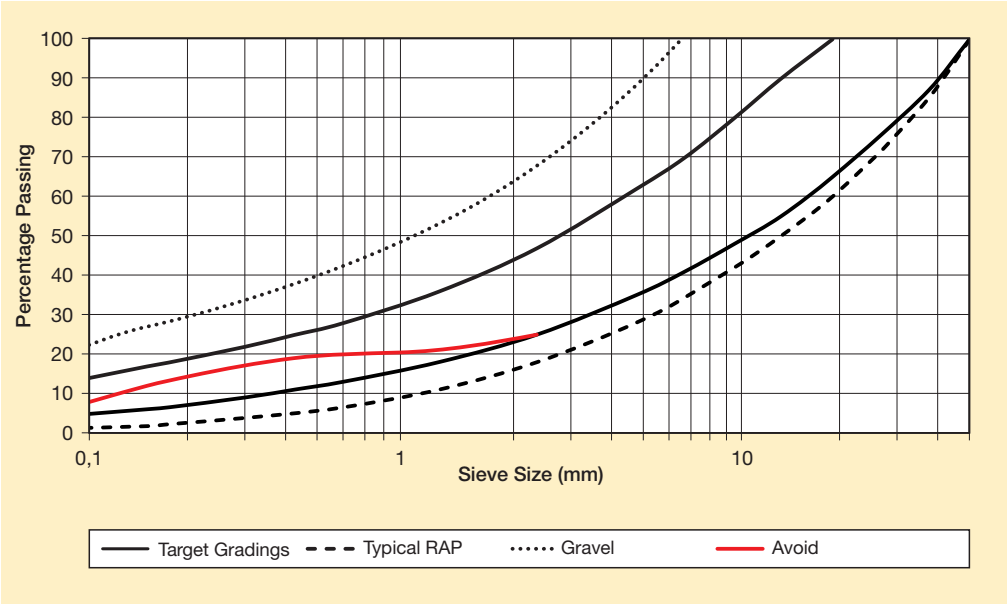
- Repeat the standard soil tests listed in Section A1.3.1.2 above to determine the grading, plasticity index and the moisture/density relationship of the blended sample.

A1.3.1.4 Gradings (sieve analyses)

Plot the grading curve for the sample that will be used in the mix designs. Include on the graph the "Target gradings" envelop from the table below. If the material is predominantly RAP or Natural Gravel, include the relevant curve (also from the table below). This plot will indicate whether additional blending with freshly imported material may be required. However, if the plot includes a "bulge" in the fractions between the 0.075 mm and 2.0 mm sieves (as shown by the red line entitled "Avoid" in the graph below), the sample should be blended with a sufficient suitable fine material (e.g. 10% by volume of minus 5 mm crusher dust) to reduce the magnitude of the bulge.

Note:

- This exercise is advisable as it allows a preliminary indication to be made of the strength that can be expected after the material has been treated with bitumen emulsion. (A poorly graded material is difficult to compact and the consequent low density achieved will significantly affect the strength, especially under saturated conditions.)



Recommended grading curves

Sieve size (mm)	Percentage passing each sieve size (%)			
	Recommended gradings		Natural gravel	Typical RAP
	Coarse	Fine		
50	100	100	100	100
37.5	87	100	100	85
26.5	76	100	100	72
19	65	100	100	60
13.2	55	90	100	50
9.5	48	80	100	42
6.7	41	70	100	35
4.75	35	62	88	28
2.36	25	47	68	18
1.18	18	36	53	10
0.6	12	27	42	6
0.425	10	24	38	4
0.3	8	21	34	3
0.15	3	16	27	1
0.075	2	10	20	0

A1.3.1.5 Representative proportioning

Separate the material in the prepared bulk sample into the following four fractions:

i. Retained on the 19.0 mm sieve;
ii. Passing the 19.0 mm sieve, but retained the 13.2 mm sieve;
iii. Passing the 13.2 mm sieve, but retained on the 4.75 mm sieve; and
iv. Passing the 4.75 mm sieve.

Reconstitute representative samples in accordance with the grading determined above (for the bulk sample) for the portion passing the 19.0 mm sieve. Substitute the portion retained on 19.0 mm sieve with material that passes the 19.0 mm sieve and retained on the 13.2 mm sieve. The example in the table below explains this procedure:

Sieve analysis		Quantity of material to be included for every 10 kg of sample		
Sieve size (mm)	Percentage passing (from sieve analysis on bulk sample)	Passing 4.75 mm	Passing 13.2 mm and retained on 4.75 mm	Passing 19.0 mm and retained on 13.2 mm
19.0	90.5	(53.6/100 x 10,000) = 5,360 g	((72.3-53.6)/100 x 10,000) = 1,870 g	((100-72.3)/100 x 10,000) = 2,770 g
13.2	72.3			
4.75	53.6			

If there is insufficient material passing the 19.0 mm sieve but retained on the 13.2 mm sieve for substituting that retained on the 19 mm sieve, then lightly crush the material retained on the 19.0 mm sieve to provide more of this fraction.

A1.3.1.6 Hygroscopic moisture content

Two representative air-dried samples, each approximately 1 kg, are used to determine the hygroscopic (air-dried) moisture content of the material. (Note: Larger sample size should be used for more coarsely-graded materials). Weigh the air-dried samples, accurate to the nearest 0.1 g, and then place them in an oven at a temperature of between 105°C and 110°C until they achieve constant mass. The hygroscopic moisture content (Wair-dry) is the loss of mass expressed as a percentage of the dry mass of the sample. Determine the hygroscopic moisture using Equation A1.3.1.

$$W_{air\text{-}dry} = \frac{(M_{air\text{-}dry} - M_{dry})}{100} \times 100$$

[equation A1.3.1]

where:

$W_{air\text{-}dry}$ = hygroscopic moisture content [% by mass]
$M_{air\text{-}dry}$ = mass of air-dried material [g]
M_{dry} = mass of oven-dried material [g]

A1.3.1.7 Sample quantities

The guidelines shown in the following table should be used to estimate the quantity of material required for the respective tests:

Test	Mass of sample required (kg)
Moisture/density relationship (modified AASHTO T180)	40
Determination of active filler requirement (100 mm Ø specimens)	60
Optimum bitumen addition indication (100 mm Ø specimens)	80
Optimum bitumen addition determination (150 mm Ø specimens)	100
Standard soil tests (gradings, Atterberg Limits, moisture content, etc.)	20

A1.3.2 Active filler requirements

A1.3.2.1 Effect of plasticity

Bitumen emulsion stabilisation is normally carried out in combination with a small amount (1% by mass) of active filler (cement or hydrated lime) to enhance bitumen adhesion and reduce moisture susceptibility. The Plasticity Index (PI) of the material is normally used as a guideline for the use of hydrated lime or cement in the mix:

Plasticity Index: < 10	Plasticity Index: > 10
Carry out ITS tests on 100 mm Ø specimens to determine the need to add either cement or hydrated lime, as described in Section A1.3.2.2 below..	Pre-treat the material with hydrated lime (ICL value) (The initial consumption of lime (ICL value) must first be determined using the appropriate pH test.)

Pre-treatment of material with a PI > 10 requires that the lime and water be added at least 2 hours prior to the addition of the bitumen emulsion. The pre-treated material is placed in an air-tight container to retain moisture. The moisture content is then checked and, if necessary, adjusted prior to adding the bitumen stabilising agent (as described in Section A1.3.4).

Note:

- Where the material is pre-treated with lime, the following tests for the “Determination of Active Filler Requirements” described under Section A1.3.2.2 below are not necessary..

A1.3.2.2 Determination of active filler requirements

Where the PI < 10, the need for an active filler and the type of active filler (cement or hydrated lime) that is appropriate for the material must first be determined by carrying out ITS tests on 100 mm diameter specimens for three different mixes made from the same sample. The amount of bitumen emulsion added to each of the three mixes is constant for each, using the fractions passing the 4.75 mm and 0.075 mm sieves as a guideline, as shown in the following table:

Guidelines for estimating optimum bitumen emulsion addition (60% residual bitumen)			
Fraction passing 0.075 mm sieve (%)	**Bitumen emulsion (Residual bitumen) addition (% by mass of dry aggregate)**		**Typical type of material**
	Fraction passing 4.75 mm sieve		
	< 50%	> 50%	
< 4	**3.3 (2.0)**	**3.3 (2.0)**	Recycled asphalt (RA/RAP)
4 – 7	**3.7 (2.2)**	**4.0 (2.4)**	RA/Graded crushed stone/ Natural gravel/blends
7 – 10	**4.0 (2.4)**	**4.7 (2.8)**	
> 10	**4.3 (2.6)**	**5.3 (3.2)**	Gravels/sands

The first of the three mixes contains no active filler, 1% cement is added to the second mix and 1% hydrated lime is added to the third mix, all three mixes being treated with the same amount of bitumen emulsion. Material from each of the three mixes is used to manufacture 100 mm diameter specimens that are cured and tested to determine the relevant ITS_{DRY} and ITS_{WET} values (described in Sections A1.3.4 to A1.3.8 below). The Tensile Strength Retained (TSR) value is then used as the primary indicator for whether an active filler is required.

Where the TSR value for the mix with no active filler added is in excess of 60%, the mix design should be undertaken with no active filler. (This situation is usually confined to materials consisting of good quality crushed stone, often including a significant proportion of reclaimed asphalt (RAP) material.)

Where the TSR value of the mix with no active filler added is less than 60%, the mix with the type of active filler that produces a significantly higher TSR value (> 5%) indicates a preference for either cement or hydrated lime and should be used in the following mix designs. If the TSR values for both active fillers are of the same order (difference < 5%) then either type of active filler is suitable.

Note:

- To determine the sensitivity of the active filler, additional tests on 100 mm diameter specimens may be undertaken using the preferred active filler at a lower application rate (e.g. 0.75%). However, to avoid compromising the flexibility of the mix, **the maximum allowable application rate for active filler is 1.0%** and should only be exceeded when hydrated lime is applied as a pre-treatment to eliminate plasticity.

A1.3.3 Determine the fluid / density relationship

The Optimum Fluid Content (OFC) and the Maximum Dry Density (MDD) of the material stabilised with bitumen emulsion is determined using standard compaction effort.

The OFC for bitumen emulsion stabilised material is the percentage by mass of bitumen emulsion plus additional moisture required to achieve the maximum dry density in the treated material. As described below, the OFC is determined by adding a constant percentage of bitumen emulsion whilst varying the amount of water added.

Step 1. Measure out the bitumen emulsion as a percentage by mass of the air-dried material for each of five prepared samples (following the procedure described in Section A1.3.1.2). The percentage of bitumen emulsion added is normally between 2 and 3% residual bitumen (e.g. for 3% residual bitumen, add 5% of a 60% bitumen emulsion).

Step 2. The bitumen emulsion and water is added to the material and mixed until uniform immediately prior to compaction.

Step 3. Determine the OFC and MDD for the stabilised material in accordance with the modified moisture-density relationship test procedure (AASHTO T-180).

A1.3.4 Treating the sample with bitumen emulsion

Prepare the sample and treat with bitumen emulsion using the following procedure:

Step 1. Place the required mass of sample (between 20 kg and 30 kg, prepared as described in Section A1.3.1 above) into the Wirtgen WLM 30 pugmill mixer.

Step 2. Determine the dry mass of the sample using Equation A1.3.2.

$$M_{sample} = \frac{(M_{air\text{-}dry})}{\left(1 + \left(\frac{W_{air\text{-}dry}}{100}\right)\right)}$$ [equation A1.3.2]

where:

M_{sample}	= dry mass of the sample	[g]
$M_{air\text{-}dry}$	= air-dried mass of the sample	[g]
$W_{air\text{-}dry}$	= moisture content of air-dried sample	[% by mass]

Step 3. Determine the required mass of active filler (lime or cement) to be added using Equation A1.3.3.

$$M_{cement} = \frac{C_{add}}{100} \times M_{sample}$$ [equation A1.3.3]

where:

M_{cement}	= mass of lime or cement to be added	[g]
C_{add}	= percentage of lime or cement required	[% by mass]
M_{sample}	= dry mass of the sample	[g]

Step 4. Determine the amount of bitumen emulsion to be added using Equation A1.3.4.

$$M_{emul} = \frac{RB_{reqd}}{PRB} \times M_{sample}$$ [equation A1.3.4]

where:

M_{emul}	= mass of bitumen emulsion to be added	[g]
RB_{reqd}	= percentage of residual bitumen required	[%]
PRB	= percentage residual bitumen in emulsion	[% by mass]
M_{sample}	= dry mass of the sample	[g]

Step 5. Determine the amount of water to be added to achieve the OFC of the material using Equation A1.3.5

$$M_{water} = \left(\frac{(W_{OFC} - W_{air-dry})}{100} \right) \times M_{sample} - M_{emul}$$

[equation A1.3.5]

where:

M_{water}	= mass of water to be added	[g]
W_{OFC}	= optimum fluid content	[% by mass]
$W_{air-dry}$	= moisture content of air-dried sample	[% by mass]
M_{sample}	= dry mass of the sample	[g]
M_{emul}	= mass of the bitumen emulsion added	[g]

Step 6. Mix the material, active filler, bitumen emulsion and water in the mixer until uniform.

Step 7. Transfer the bitumen emulsion treated material into an air-tight container and immediately seal. To minimise moisture loss, manufacture the test specimens as soon as possible by following the relevant procedure for either 100 mm or 150 mm diameter specimens, as described in sections A1.3.5.1 and A1.3.5.2 respectively.

Repeat the above steps for at least four mixes with different residual bitumen contents at 0.2% intervals.
The "Guidelines for estimating optimum bitumen emulsion addition" (Section A1.3.2.2 above) should be used in determining the mid-point of the range of bitumen emulsion to be added to the four samples.

An example. If the material consists of a blend of RAP and crushed stone with 39% and 8% passing the 4.75 mm and 0.075 mm sieves respectively, the guidelines indicate an optimum bitumen emulsion (residual bitumen) addition of 4.0 (2.4)%. The amount of bitumen emulsion (residual bitumen) to be added to each sample (all with the same amount of active filler and at the optimum fluid content) is:

Sample 1: 3.5 (2.1)%
Sample 2: 3.8 (2.3)%
Sample 3: 4.2 (2.5)%
Sample 4: 4.5 (2.7)%

A1.3.5 Manufacture of specimens for testing

The procedures described below are for the manufacture of two different sizes of specimen using different compaction procedures:

Specimen size and compaction effort applied in the manufacturing process		
Specimen diameter	**Specimen height**	**Compaction effort**
100 mm	63.5 mm	Modified Marshall *
150 mm	95.0 mm	Modified AASHTO

* 75 blows per face

The following two questions are often raised:

1. **Which specimen size should be manufactured?** As described in Section A1.3.8 below, ITS_{DRY} and ITS_{WET} values are normally determined from 100 mm diameter specimens. 150 mm diameter specimens may be substituted for 100 mm diameter specimens to obtain the same values. However, when dealing with coarse material, (i.e. where the grading curve tends towards the coarse side of the recommended grading envelop) it is strongly recommended that 150 mm diameter specimens are manufactured and tested in place of the smaller 100 mm diameter specimens.
   Note. Only 150 mm diameter specimens are used to determine ITS_{EQUIL} and ITS_{SOAK} values.

2. **Can other compaction methods be used?** The compaction procedures described below are well known standard procedures that can be carried out in most laboratories, worldwide. Other procedures may be used (e.g. gyratory compaction, vibrating hammer, vibrating table, etc.) provided they achieve the same density target of 100% Marshall compaction for the 100 mm diameter specimens or 100% of the mod AASHTO T-180 density for 150 mm diameter specimens.

A1.3.5.1 Manufacture of 100 mm diameter specimens

A minimum of six (6) 100 mm diameter specimens, 63.5 mm in height, are manufactured from each sample of treated material by applying modified Marshall compaction effort, as described in the following steps:

Step 1 Prepare the Marshall mould and hammer by cleaning the mould, collar, base-plate and face of the compaction hammer. Note: the compaction equipment must not be heated but kept at ambient temperature.

Step 2. Weigh sufficient material to achieve a compacted height of 63.5 mm ± 1.5 mm (Approximately 1,100 g for most materials). Poke the mixture with a spatula 15 times around the perimeter and 10 times on the surface, leaving the surface slightly rounded.

Step 3. Compact the mixture by applying 75 blows with the compaction hammer. Care must be taken to ensure the continuous free fall of the hammer.

Step 4. Remove the mould and collar from the pedestal, invert the specimen (turn over). Replace it and press down firmly to ensure that it is secure on the base plate. Compact the other face of the specimen with a further 75 blows.

Step 5. After compaction, remove the mould from the base-plate and extrude the specimen by means of an extrusion jack. Measure the height of the specimen and adjust the amount of material if the height is not within the 1.5 mm limits.

Note: Coarse materials are often damaged during the extrusion process. It is therefore recommended that the specimens are left in their moulds for 24 hours allowing sufficient strength to develop before extruding.

Repeat steps 1 to 5 for the manufacture of at least six (6) specimens.

Step 6. Take ±1 kg representative samples after compaction of the second and fifth specimen and dry to a constant mass. Determine the moulding moisture using Equation A1.3.6.

$$W_{mould} = \frac{(M_{moist} - M_{dry})}{M_{dry}} \times 100 \qquad \text{[equation A1.3.6]}$$

where:

W_{mould}	= moulding moisture content	[% by mass]
M_{moist}	= mass of moist material	[g]
M_{dry}	= mass of dry material	[g]

A1.3.5.2 Manufacture of 150 mm diameter specimens

A minimum of six (6) 150 mm diameter specimens, 95 mm in height, are manufactured from each sample of treated material by applying modified AASHTO (T-180) compaction effort, as described in the following steps:

Step 1. Prepare the equipment by cleaning the mould, collar, base-plate and face of the compaction rammer. (Either split-moulds or standard "Proctor" moulds may be used, each fitted with a 32 mm spacer placed on the base plate to achieve specimens that are 95 mm (±1.5 mm) in height.)

Note: the modified AASHTO compaction rammer has the following specifications:

Rammer diameter:	50 mm
Mass:	4.536 kg
Drop distance:	457 mm

Step 2. Compact each specimen applying modified AASHTO (T-180) compaction effort (4 layers approximately 25 mm thick, each receiving 55 blows from the drop rammer.)

Step 3. Carefully trim excess material from specimens, as specified in the AASHTO T-180 test method.

Step 4. After compaction, remove the mould from the base-plate and extrude the specimen by means of an extrusion jack. Where split-moulds are used, separate the segments and remove the specimen.

Note: Coarse materials are often damaged during the extrusion process. It is therefore recommended that the specimens are left in their moulds for 24 hours allowing sufficient strength to develop before extruding.

Where split moulds are used, it is advisable to leave the specimen in the mould for 4 hours before splitting the mould and extracting the specimen.

Repeat steps 1 to 4 for the manufacture of at least six (6) specimens.

Step 5. Take ±1 kg representative samples after compaction of the second and fifth specimen and dry to a constant mass. Determine the moulding moisture content using Equation A1.3.6 (above).

A1.3.6 Curing the specimens

Two curing regimes are described below. The first is a standard procedure to dry the specimens to constant mass. The second procedure aims to simulate field conditions where the “equilibrium moisture content” is approximately 50% of OMC. Both 100 mm and 150 mm diameter specimens may be cured dry (to constant mass) whereas only 150 mm diameter specimens are cured at equilibrium moisture content.

A1.3.6.1 Curing dry

Place the specimens (either 100 mm or 150 mm diameter) in a forced-draft oven at 40° C and cure to constant mass (normally 72 hours).

To determine if constant mass has been achieved, weigh the specimens and place back in the oven. Remove after 4 hours and reweigh. If the mass is constant, then continue with testing. If the mass is not constant, place back in the oven and reweigh after repeated 4 hour intervals until constant mass is achieved.

When constant mass is achieved, remove the specimens from the oven and allow to cool to 25° C (± 2.0° C).

A1.3.6.2 Curing to simulate field conditions

Place the 150 mm diameter specimens in a forced-draft oven at 30° C for 24 hours (or until the moisture content has reduced to approximately 50% of OMC).

Take the specimens out of the oven, place each in a sealed plastic bag (at least twice the volume of the specimen) and place back in the oven at 40° C for a further 48 hours.

Remove specimens from the oven after 48 hours and take out of their respective plastic bags, ensuring that any moisture in the bags does not come into contact with the specimen. Allow to cool down to 25° C (± 2.0° C).

A1.3.7 Preparing the specimens for testing

After cooling, determine the bulk density of each specimen using the following procedure:

Step 1. Determine the mass of the specimen.

Step 2. Measure the height of the specimen at four evenly-spaced locations around the circumference and calculate the average height of the specimen.

Step 3. Measure the diameter of the specimen.

Step 4. Calculate the bulk density of the each specimen using Equation A1.3.7.

$$BD_{spec} = \frac{4 \times M_{spec}}{\pi \times d^2 \times h} \times 1{,}000{,}000 \qquad \text{[equation A1.3.7]}$$

where:

BD_{spec}	= bulk density of specimen	[kg/m^3]
M_{spec}	= mass of specimen	[g]
h	= average height of specimen	[mm]
d	= diameter of specimen	[mm]

Exclude from further testing any specimen whose bulk density differs from the mean bulk density of all six (6) specimens by more than 2.5%.

Step 5. Place half of the specimens (normally 3) under water in a soaking bath for 24 hours at 25° C (± 2° C). After 24 hours, remove the specimens from the water, surface dry and test immediately.

A1.3.8 Determination of the indirect tensile strength (ITS) of specimens

The ITS of a specimen is determined by measuring the ultimate load to failure applied to the diametrical axis at a constant deformation rate of 50.8 mm/minute. Ensure that the temperature of the specimens is 25° C (± 2° C) and follow the procedure described below:

Step 1. Place the specimen onto the ITS jig. (Ensure the correct loading strips are appropriate for the diameter of the specimen.) Position the sample such that the loading strips are parallel and centred on the vertical diametrical plane.

Step 2. Place the transfer plate on the top bearing strip and position the jig assembly centrally under the loading ram of the compression testing device.

Step 3. Apply the load to the specimen, without shock, at a rate of advance of 50.8 mm per minute until the maximum load is reached.

Step 4. Record the maximum load P (in kN), accurate to 0.1 kN.

Step 5. Record the displacement at break to the nearest 0.1 mm.

Step 6. Break the specimen in half and record the temperature of the specimen at its centre.

Step 7. Break up one of the unsoaked specimens and dry to a constant mass. Determine the cured moisture content using Equation A1.3.8.

$$W_{spec} = \frac{(M_{moist} - M_{dry})}{M_{dry}} \times 100$$ [Equation A1.3.8]

where:
W_{spec} = moisture content of specimen [% by mass]
M_{moist} = mass of moist material [g]
M_{dry} = mass of dry material [g]

Determine the dry density of each specimen using Equation A1.3.9

$$DD_{spec} = BD_{spec} \times \left(1 - \frac{W_{spec}}{100}\right)$$ [Equation A1.3.9]

where:
DD_{spec} = dry density of specimen [% by mass]
BD_{spec} = bulk density of specimen [g]
W_{spec} = moisture content of specimen [g]

Step 8. Break up one of the soaked specimens and dry to a constant mass (at 105° C to 110° C). Determine the after-soaking moisture content using Equation A1.3.8.

Step 9. Calculate the ITS value for each specimen to the nearest 1 kPa using Equation A1.3.10.

$$ITS = \frac{2 \times P}{\pi \times h \times d} \times 1{,}000{,}000$$ [Equation A1.3.10]

where:

ITS	= Indirect Tensile Strength	[kPa]
P	= maximum applied load	[kN]
h	= average height of the specimen	[mm]
d	= diameter of the specimen	[mm]

Step 10. Use the Worksheet in Annexure A1.3.1 to record the data and the form in Annexure A1.3.2 to report the results.

Step 11. For dry-cured specimens only, calculate the Tensile Strength Retained (TSR) value for the sample using Equation A1.3.11.

$$TSR = \frac{Ave\ ITS_{WET}}{Ave\ ITS_{DRY}} \times 100$$ [Equation A1.3.11]

where:

TSR	= Tensile Strength Retained	[%]
Ave ITS_{WET}	= average ITS_{WET} value	[kPa]
Ave ITS_{DRY}	= average ITS_{DRY} value	[kPa]

Note. To differentiate between the results obtained from the different curing regimes, the terminology shown in the table below should be adopted to avoid any confusion.

Term	Specimen diameter	Curing regime	Moisture content
ITS_{DRY}	100 mm or 150 mm	72 hrs unsealed	<1%
ITS_{WET}		24 hrs soak	Saturated
ITS_{EQUIL}	150 mm only	24 hrs unsealed, 48 hrs in sealed bag	± 50% of OMC
ITS_{SOAK}		24 hr soak	Semi-saturated

A1.3.9 Interpretation of the indirect tensile strength (ITS) test results

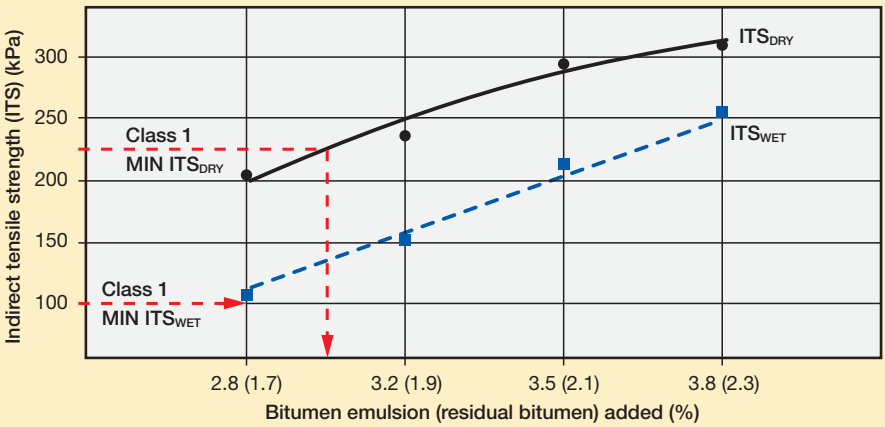
Plot the respective soaked and unsoaked ITS test results against the relevant addition of bitumen emulsion, as shown in the example below. If results for two different curing regimes were obtained, each must be plotted on a separate graph.

The added bitumen emulsion that meets the minimum ITS value for the required material classification is selected as the primary indicator for the minimum amount of bitumen emulsion to be added. Engineering judgement is then used to determine the amount of bitumen emulsion that needs to be added in order to achieve sufficient confidence, based on the variability of the test results (the “goodness of fit” of the regression curve or line through the plotted test results).

The example below explains the process.

The table and plot of ITS test results are typical ITS values achieved from a mix design using 100 mm diameter specimens for a RAP/crushed stone blended material stabilised with a 60% residual bitumen emulsion.

Added bitumen emulsion (residual bitumen) (%)	ITS_{DRY} (kPa)	ITS_{WET} (kPa)	TSR (%)
2.8 (1.7)	204	109	53.4
3.2 (1.9)	234	151	64.5
3.5 (2.1)	282	214	75.9
3.8 (2.3)	327	253	77.4



100 mm Ø specimens

The curve through the four ITS_{DRY} points approximates the relationship between ITS_{DRY} and added bitumen emulsion (residual bitumen). The line through the four ITS_{WET} points approximates the relationship between ITS_{WET} and added bitumen. The fine dotted lines indicate that the addition of at least 3.0 (1.8)% of bitumen emulsion (residual bitumen) is required to meet the requirements for a Class 1 bitumen stabilised material (ITS_{DRY} > 225 kPa and ITS_{WET} > 100 kPa). Based on these ITS test results and the corresponding TSR values, the level of confidence may be increased by selecting a bitumen emulsion addition that meets the requirements. For example, selecting 3.5 (2.1)% bitumen emulsion (residual bitumen) addition will significantly increase the probability of the stabilised material achieving the minimum ITS_{DRY} and ITS_{WET} values under field conditions.

Annexure A1.3.1

BITUMEN EMULSION MIX DESIGN - WORKSHEET

Project _____ Date _____

Sample / Mix No.: _____ Location _____

Material description : _____

Maximum dry density _____ Optimum fluid content

Percentage < 0.075mm _____ Grading: Coarse | Medium | Fine

Plasticity Index _____

Emulsion Source _____ Emulsion type _____

Residual bitumen (%) _____

Active Filler Type _____ Filler Source _____

MOISTURE DETERMINATION

			Specimen manufacture		After Curing	
		Hygroscopic	Sample 1	Sample 2	Dry	Soaked
Pan No.						
Mass wet sample + pan	m1					
Mass dry sample + pan	m2					
Mass pan	mp					
Mass moisture	m1-m2 = Mm					
Mass dry sample	m2-mp= Md					
Moisture content	Mm/Mdx100=Mh					

Mass of air-dried sample placed in the mixer (kg) _____

Percentage of water added to sample for mixing: _____ Amount of water added : _____

Total percentage water added: _____ Total water added: _____

Bitumen emulsion addition (%): _____ Active filler addition (%): _____

Residual bitumen addition (%) _____

Temperatures (˚C) Material: _____ Emulsion: _____ Water: _____

SPECIMEN DETAILS

Specimen ID						
Date Moulded						
Date removed from oven						
Date tested						
Diameter (mm)						
Individual height measurements (mm)						
Average height (mm)						
Mass after curing (g)						
Bulk density (kg/m^3)						
Average bulk density						
Dry density (kg/m^3)						

ITS TEST

Specimen condition	**Unsoaked (ITS_{DRY} / ITS_{EQUIL})**			**Soaked (ITS_{WET} / ITS_{SOAK})**		
Maximum load (kN)						
Internal temperature (˚C)						
Deformation (mm)						
ITS (kPa)						
Average ITS (kPa)						
TSR (%)						

Annexure A1.3.2

BITUMEN EMULSION MIX DESIGN REPORT (Dry curing)

Project _____ Date _____

Sample number: _____ Location _____

Material description : _____

Maximum dry density _____ Optimum fluid content

Percentage < 0.075mm _____ Grading: Coarse | Medium | Fine

Plasticity Index _____

Bitumen Emulsion Type _____

Emulsion Supplier _____ Residual Bitumen (%) _____

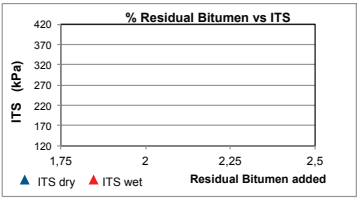
Active Filler Type _____ Filler Source _____

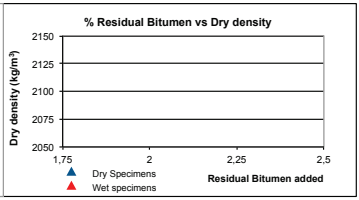
BITUMEN EMULSION STABILISED MATERIAL SPECIMENS

Compactive effort				mm	specimen diameter
Date moulded					
Date tested					
Bitumen emulsion added	(%)				
Residual bitumen added	(%)				
Active filler added	(%)				
Moulding moisture content	(%)				

TEST RESULTS

ITS_{DRY}	(kPa)				
Moisture content at break	(%)				
Dry Density	(kg/m$^{3)}$				
Average deformation	(mm)				
Temperature at break	(°C)				
ITS_{WET}	(kPa)				
Moisture content at break	(%)				
Dry Density	(kg/m$^{3)}$				
Average deformation	(mm)				
Temperature at break	(°C)				
Tensile Strength Retained	(%)				
Material classification					





Comments _____

A1.4 Testing field samples of bitumen stabilised materials (BSMs)

A1.4.1 Field sampling

Samples (± 100 kg) of treated material are to be obtained on site:

- Where the material is mixed in situ, the sample is taken immediately behind the recycler from the full thickness of treated material (before it is compacted).
- Where the material is plant mixed and placed by paver or grader, the samples may be taken either at the mixing plant or from site following standard sampling procedures.

Place each sample in an air-tight container and seal to prevent moisture loss. The container or bag used for sampling must be sufficiently large to ensure that the material remains loose and does not compact inside the container.

Field samples are to be transported to the laboratory within two hours of sampling and the test specimens manufactured within four hours of being sampled.

A1.4.2 Sample preparation

Prepare the sample by passing through a 19 mm sieve. Discard the fraction retained on the 19 mm sieve. Place the sample in an air-tight container and check that the temperature is at a temperature of between 22° C and 25° C. If the temperature of the material is not within this temperature range, place the entire sample in an air cabinet (or similar) until the material is within this range.

A1.4.3 Adjust the moisture content

Since field samples normally lie in the range of 60% to 80% of Optimum Moisture Content (OMC), sufficient water must be added to the sample to bring it to OMC before the test specimens are manufactured. (Due to the variability of recycled material, it is seldom that the OMC will be known with certainty. The amount of water required to bring the sample to OMC must usually be determined (for each and every sample) using the procedure described in Section A1.4.3.2 below.)

A1.4.3.1 Where the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of the material is known with certainty

- Determine the moisture content of the field sample;
- Adjust the moisture content of 10 kg of the sample to achieve OMC, mix thoroughly and place in an air-tight container; then
- Proceed to Section 4 and manufacture 100 mm diameter specimens for testing.

A1.4.3.2 Where the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of the material is not known

Adjust the moisture content of the sample to bring it to the OMC.

Step 1. Prepare the Marshall mould and hammer by cleaning the mould, collar, base-plate and hammer face. (Note: the compaction equipment must not be heated but kept at ambient temperature.)

Step 2. Weigh out 1,100 g of material and place in the mould. Poke the mixture with a spatula 15 times around the perimeter and 10 times on the surface, leaving the surface slightly rounded. Ensure that the material does not lose moisture or segregate during placing in the mould and seal the remaining sample in the air-tight container.

Step 3. Compact the material by applying 75 blows with the compaction hammer. Care must be taken to ensure the continuous free-fall of the hammer. Remove the mould and collar from the pedestal, invert the specimen (turn over). Replace it and press down firmly to ensure that it is secure on the base plate. Compact the other face of the specimen with a further 75 blows.

Step 4. After compaction, remove the mould from the base-plate and extrude the specimen by means of an extrusion jack.

Step 5. Determine the mass of the specimen.

Step 6. Measure the height of the specimen at four evenly-spaced locations around the circumference and calculate the average height of the specimen.

Step 7. Measure the diameter of the specimen.

Step 8. Calculate the "interim" bulk density of the specimen using equation A1.4.1:

$$BD_{INT} = \frac{4 \times M_{SPEC}}{\pi \times d^2 \times h} \times 100\,000$$ [Equation A1.4.1]

where:

BD_{INT}	= interim bulk density	[kg/m³]
M_{SPEC}	= mass of specimen	[g]
h	= average height of specimen	[mm]
d	= diameter of specimen	[mm]

Then, using Equation A1.4.2, calculate the "true" bulk density by excluding the total amount of moisture that was added:

$$BD = \frac{BD_{INT}}{\left(100 + \frac{W_{ADD}}{100}\right)} \times 100$$ [Equation A1.4.2]

where:

BD	= bulk density	[kg/m³]
BD_{INT}	= interim bulk density (from Equation 1)	[kg/m³]
W_{ADD}	= total moisture added to sample	[%]

Step 9. Plot the bulk density against water addition. (The first point on the graph is zero water addition.)

Step 10. Weigh out another 1,100 g of material. Add 5.5 ml of water (0.5% by mass) and mix thoroughly before placing in the mould, following the procedure described in Step 2.

Step 11. Repeat Steps 3 to 9 and compare the bulk density with that achieved from the previous specimen. Continue to follow Steps 3 to 10 making specimens with additional water added until the bulk density reduces from the previous specimen.

Step 12. The amount of water added to the specimen that returns the highest bulk density (in percent) is then added to 10 kg of the remaining sample. Thoroughly mix the material and return to the air-tight container.

Note. If moisture is observed seeping out of the mould before a turning point in the curve is achieved, then the amount of water addition required to achieve maximum density is the amount of water added to the material at which seepage occurred, less 0.5%.

A1.4.4 Manufacture of 100 mm diameter specimens for testing

Using specimen height measurements from Step 6 (Section A1.4.3 above) as a guideline, determine the mass of material required to achieve a specimen height of 63.5 mm (±1.5 mm). Then, follow the procedure described in Steps 2 to 4, using the revised mass of material to manufacture six (6) specimens, each with a height of 63.5 mm (±1.5 mm).

Note. Where the OMC of the sample is known and the moisture adjustment procedure described above in Section 3 was not carried out, follow steps 2 and 3 (Section A1.4.3) using 1,100 g of sample prepared at OMC. Measure the height of the specimen (Step 6) and, if necessary, adjust the mass to achieve a specimen height of 63.5 mm (±1.5 mm). Then, follow the procedure described in Steps 2 to 4, using the revised mass of material to manufacture six (6) specimens, each with a height of 63.5 mm (±1.5 mm).

After all specimens have been manufactured, use the remaining sample material to determine the moulding moisture content following standard oven drying procedures.

A1.4.5 Curing the specimens

Place the specimens in a forced-draft oven at 40° C and cure to constant mass (normally 72 hours).

To determine if constant mass has been achieved, weigh the specimens and place back in the oven. Remove after 4 hours and reweigh. If the mass is constant, then continue with testing. If the mass is not constant, place back into the oven and reweigh at 4 hour intervals until constant mass is achieved.

After constant mass has been achieved, remove the specimens from the oven and allow to cool to 25° C (± 2.0° C).

After cooling, determine the bulk density of each specimen following Steps 5 to 8 described in Section A1.4.3.

Exclude from further testing any specimen whose bulk density differs from the mean bulk density of all six (6) specimens by more than 2.5%.

Half of the specimens (3) are then placed under water in a soaking bath for 24 hours at 25° C (± 2° C). After 24 hours, remove the specimens from the water, surface dry and test immediately.

A1.4.6 Determination of Indirect Tensile Strength (ITS)

The ITS is determined by measuring the ultimate load to failure of a specimen that is subjected to a constant deformation rate of 50.8 mm/minute on its diametrical axis. Ensure that the temperature of the specimens is 25° C (± 2° C) and follow the procedure described below:

Step 1. Place the specimen onto the ITS jig. Position the sample such that the loading strips are parallel and centred on the vertical diametrical plane.

Step 2. Place the transfer plate on the top bearing strip and position the jig assembly centrally under the loading ram of the compression testing device.

Step 3. Apply the load to the specimen, without shock, at a rate of advance of 50.8 mm per minute until the maximum load is reached.

Step 4. Record the maximum load P (in kN), accurate to 0.1 kN.

Step 5. Record the displacement at break to the nearest 0.1 mm.

Step 6. Break the specimen in half and record the temperature of the specimen at its centre.

Step 7. Break up one of the dry and one of the wet specimens and determine the moisture content following standard oven drying procedures.

Step 8. Calculate the ITS value for each specimen to the nearest 1 kPa using Equation A1.4.3:

$$ITS = \frac{2 \times P}{\pi \times h \times d} \times 100\,000$$ [Equation A1.4.3]

where:

ITS	= Indirect Tensile Strength	[kPa]
P	= maximum applied load	[kN]
h	= average height of the specimen	[mm]
d	= diameter of the specimen	[mm]

Then, using Equation A1.4.2, calculate the “true” bulk density by excluding the total amount of moisture that was added:

Step 9. Use the form on the following page to report the results.

ITS DETERMINATION for BSM FIELD SAMPLE

PROJECT ..... Position .....

Sample No.: ..... Date .....

Description : .....

FIELD TREATMENT

Maimum dry density ..... (If known) Optimum moisture content .....

Bitumen type / source ..... Bitumen applied (%) .....

Active filler type / source ..... Active filler applied (%) .....

MOISTURE DETERMINATION		Specimen manufacture		After Testing	
		Field	Moulding	Dry	Soaked
Pan No.					
Mass wet sample + pan	m1				
Mass dry sample + pan	m2				
Mass pan	mp				
Mass moisture	m1 - m2 = Mm				
Mass dry sample	m2 - mp = Md				
Moisture content	Mm / Md x 100 = Mh				

SPECIMEN DETAILS	**DRY**			**SOAKED**		
Date Moulded						
Date removed from oven						
Specimen ID						
Diameter (mm)						
Individual Thickness Readings (mm)						
Avgerage Thickness (mm)						
Mass after curing (g)						
Bulk density (kg/m^3)						
Avg bulk density (kg/m^3)						
Moisture content (%)						
Dry density (kg/m3)						
Avg dry density (kg/m^3)						

ITS TESTING						
Condition	**Dry (ITS_{DRY})**			**Soaked (ITS_{WET})**		
Date tested						
Maximum load (kN)						
Temperature (°C)						
Deformation (mm)						
Tensile strength (kPa)						
Avg tensile strength (kPa)						

A1.5 Determining the strength of BSM core specimens

Core samples may be extracted from the full thickness of the completed layer and tested for ITS values. Due to the relatively low strength of a BSM, 150 mm diameter cores are preferable to the 100 mm diameter cores normally extracted for HMA. Cores cannot be successfully extracted until the BSM has developed sufficient strength and the delay period is dictated by the rate of moisture loss from the material which is primarily a function of weather conditions and layer thickness. When conditions are warm and dry, cores can usually be extracted from a 150 mm thick layer of BSM-foam after 14 days. The delay period for BSM-emulsion is further influenced by the stability of the emulsion and delays of 30 days are normal.

A1.5.1 Extracting core samples

The core barrel used to extract samples of BSM must be in a good condition. The amount of water added whilst drilling should be kept to an absolute minimum and the rate of penetration kept sufficiently low to prevent erosion and damage. After extraction, core samples must be wrapped individually in a soft cloth and carefully packed for transporting to the laboratory.

A1.5.2 Cutting core specimens

Use a rotary saw fitted with a large diameter diamond-tipped blade to cut specimens from the portion of the core that suffered least damage during extraction and handling. The thickness (height) of individual specimens cut from the core is dictated by the diameter:

- 100 mm diameter cores: 63 mm
- 150 mm diameter cores: 95 mm

Where possible, more than one specimen should be cut from each core sample.

A1.5.3 Curing the core specimens

Cure the specimens to constant mass in a forced-draft oven at 40° C (normally 72 hours).
Where the ITS_{WET} value is to be determined, place the specimens in a soaking bath for 24 hours.

A1.5.4 Determination of bulk density

Follow the procedures described in Section A1.2.7 for determining the bulk density of each core specimen.

A1.5.5 Determination of the Indirect Tensile Strength (ITS)

Follow the procedures described in Section A1.2.8 for testing the core specimens to determine the ITS_{DRY} and ITS_{WET} values and the resulting TSR value. These values are then used to determine whether the material has met the minimum specified requirements.

Note. Where the ITS results for specimens manufactured from field samples are in conflict with those obtained from core specimens, the results for the core specimens should be taken as being the correct values.

A1.6 Laboratory equipment requirements

A1.6.1 Laboratory equipment for soils testing

Description	Quantity
Sample Preparation	
Riffler (25 mm openings)	1
Riffler pans	3
Sieves 450 mm diameter	
19.0 mm	1
13.2 mm	1
4.75 mm	1
20 litre air-tight containers (Plastic buckets with lids)	20
50 kg mechanical balance	1
Sieve Analysis (gradings)	
Sieves 200 mm diameter	
50.0 mm	1
37.5 mm	1
25.0 mm	1
19.0 mm	1
12.5 mm	1
9.5 mm	1
4.75 mm	1
2.36 mm	1
1.18 mm	1
0.60 mm	1
0.30 mm	1
0.15 mm	1
0.075 mm	3
Pan	1
Lid	1
Electronic balance (15 kg ± 0.1 g)	1
Forced-draft drying oven (minimum 240 litre capacity)	1
Pans (±300 mm Ø)	10
Sieve brush	1
<u>Optional Equipment</u>	
Mechanical Sieve Shaker	1
Atterberg Limits (plasticity)	
Casagrande Liquid Limit Device	1
Grooving tool	1
Mixing bowls (±100 mm Ø)	2
Spatula	1
Wash bottle (250 ml)	1
Timer	1
Glass pane (300 mm x 300 mm)	1
Glass jars (100 ml)	50
Drying oven (use oven in Sieve Analysis section)	1

Description	Quantity
Moisture / density relationship Modified AASHTO (T-180)	
150 mm Ø mould (including base plate, spacer and collar)	18
Compaction rammer (4.536 kg mass with 457 mm drop and 50 mm diameter)	1
Electronic balance (12 kg ± 0.1 g)	1
Mixing basin (± 0.5 m x0.5 m x 0.3 m)	1
Mixing trowel	1
1 litre plastic measuring cylinder	1
Steel straight edge (for trimming)	1
Containers for moisture content (half liter capacity)	50
Forced-draft drying oven (400 litre capacity)	1
<u>Optional Equipment</u>	
Mechanical compactor with rotating base plate	1
California bearing ratio	
150 mm Ø moulds (including perforated base plate and surcharge weights)	30
Compaction rammer (2.495 kg mass with 305 mm drop and 50 mm diameter)	1
Swell gauge	1
Soaking bath (2 m x 1 m x 0.4 m)	1
Compression testing machine	1

A1.6.2 Additional laboratory equipment cement (or lime) stabilisation

Note: This equipment is required **IN ADDITION** to the list shown under A1.4.1, Laboratory equipment for soils testing

Description	Quantity
Compaction	
150 mm Ø steel split moulds & collar	3
Specimen curing	
250 mm x 350 mm perforated trays	12
Forced-draft drying oven (400 litre capacity)	1
Plastic bags (± 10 litre)	500
Electronic balance (10 kg ±0.1 g)	1
Temperature controlled waterbath (Note. The CBR soaking bath may be used if the ambient temperature is constant at ± 25 deg. C)	1
Indirect Tensile Strength Testing	
ITS test jig for 150 mm Ø specimens	1
Compression testing machine * (loading rate 50.8 mm/min)	1
Unconfined Compressive Strength Testing	
150 mm Ø load transfer plate	1
Compression testing machine * (loading rate 153 kN/min)	1

Description	Quantity
Ancillary Equipment	
Teltru Thermometer (250° C) (for bath and ovens)	3
Shovel (300 mm)	1
Gloves (heat resistant)	1
500 mm paint brushes	2
Broom with handle (soft)	1
Hammer (2 kg)	1
Silicon grease (100 g)	1
Rags for cleaning	1
String	1
Marker pen (or paint) (for marking specimens)	1
Grain scoop or similar	2
Electronic thermometer	1
Hand cleaner	1

* Note. The same compression testing machine as used in the CBR test may be used if equipped with adjustable loading rate

A1.6.3 Additional laboratory equipment for bitumen stabilisation

Note: This equipment is required **IN ADDITION** to the list shown under A1.4.1, Laboratory equipment for soils testing

Description	Quantity	Description	Quantity
Foamed bitumen mix designs only		**Indirect Tensile Strength Testing**	
Wirtgen WLB10 S Laboratory unit (complete with air compressor)	1	ITS test jig for 100 mm Ø specimens	1
		ITS test jig for 150 mm Ø specimens	1
		Compression testing machine*	1
Mechanical mixing equipment		**Ancillary Equipment**	
Wirtgen WLM 30 pugmill mixer	1	Teltru Thermometer (250° C) (for bath and ovens)	3
		Shovel (300 mm)	1
		Gloves (heat resistant)	1
		500 mm paint brushes	2
100 mm Ø specimen manufacture		Broom with handle (soft)	1
Marshall compactor (Manual or automatic with wooden pedestal & hammer)	1	Hammer (2 kg)	1
		Silicon grease (100 g)	1
100 mm Ø moulds (with collar and base plate)	24	Rags for cleaning	1
Extrusion jack	1	String	1
Vernier calipers (25 mmm)	1	Marker pen(or paint) (for marking specimens)	1
		Grain scoop or similar	2
		Electronic thermometer	1
		Hand cleaner	1
Specimen curing		* Note. The same compression testing machine as used in the CBR test may be used if equipped with adjustable loading rate	
250 mm x 350 mm perforated trays	12		
Forced-draft drying oven (400 litre capacity)	1		
Plastic bags (± 10 litre)	500		
Electronic balance (10 kg ±0.1 g)	1		
Temperature controlled waterbath			
(Note. The CBR soaking bath may be used if the ambient temperature is constant at ± 25 deg. C)	1		

Appendix 2 – Determining structural capacity from traffic information

A2.1 Terminology associated with traffic

There are three key terms/abbreviations used to describe the traffic that uses a road and these need to be clearly understood to avoid confusion.

- Annual average daily traffic (AADT) measured in vehicles per day. Sometimes abbreviated to average daily traffic (ADT). This is a measure of the total daily volume of traffic using a road. It includes all traffic travelling in both directions and takes no cognisance of the different types of vehicles (cars, trucks) that make up the traffic spectrum, nor the number of traffic lanes. Although AADT is widely used to describe traffic volumes, it is not a very useful measure for structural design purposes. An understanding of the traffic spectrum and split between lanes is essential for determining the pavement structure required.

- Equivalent standard axle load (ESAL). The loading of heavy vehicles is always governed by legislation and road pavements are designed accordingly. The term “legal axle load” usually defines the maximum load permitted on a single axle. This varies from country to country, typically 80 kN to 130 kN.
  For pavement design purposes, the axle configuration of a vehicle is also important in determining the load applied in terms of “Equivalent Standard Axle Loads” (ESALs) with the “standard axle” defined (e.g. 80 kN).

Pavements are designed to carry a certain number of ESALs. This is called the Structural Capacity of a pavement and is usually expressed in millions (e.g. 5×10^6 ESALs).

- Average daily equivalent (ADE) traffic. This is the most useful information for pavement design since it defines the number of equivalent standard axle loads that are currently using the road per lane. Determining this key number is discussed below.

A2.2 Traffic loading classification

Pavements are classified by the number of ESALs that the road is designed to carry during its service life (i.e. the Structural Capacity). This introduces a time frame and requires that a “design life” be defined. Road authorities normally expect a return on their investment in a pavement and, typically, periods ranging from 5 to 30 years are used in such calculations. This return period is then used to define the design life of the pavement. Predicting the design traffic (or the number of ESALs expected during that period) is therefore most important as the cost implications of inaccurate data is obvious in terms of layer numbers, thickness and material composition.

Table A2.1 Typical pavement classification

Class	ESALs x 10^6
T0	< 0.3
T1	0.3 – 1.0
T2	1.0 – 3.0
T3	3.0 – 10.0
T4	10.0 – 30.0
T5	30.0 – 100.0

Many different classification systems exist to describe traffic loading. The often-used terms “light/medium/heavy” are too subjective and cannot be used for pavement design.
A system that classifies the traffic into loading ranges is normally adopted, such as the one illustrated in Table A2.1 that has been adopted as a regional guideline for all Southern African countries.

Translating traffic counts into useful design information requires converting data collected on the traffic spectrum (discussed below) into ESALs. Table A2.2 may be used as a rough guide to determine the number of ESALs that will be applied to the road surface by different types of heavy vehicles. It should be noted that light vehicles are not assigned an ESAL factor and are therefore of no consequence from a pavement design perspective. The ESALs per vehicle data presented in Table A2.2 were derived from survey data collected in South Africa and may not be representative for other countries with different vehicle types and traffic spectra. This information therefore needs to be obtained from the relevant road authority (where available) or from a counting exercise.

Table A2.2 Example of typical ESALs per heavy vehicle

Vehicle type	Normal Range	Average
2-axle truck	0.3 – 1.1	0.70
2-axle bus	0.4 – 1.5	0.73
3-axle truck	0.8 – 2.6	1.70
4-axle truck	0.8 – 3.0	1.80
5-axle truck	1.0 – 3.0	2.20
6-axle truck	1.6 – 5.2	3.50
7-axle truck	3.8 – 5.0	4.40
Average	2.5 – 6.0	

Ultra-heavy classes are also defined for extra heavy-duty pavements, such as those constructed for major runways and mine haul roads. Such pavements, however, are beyond the scope of this manual and should be regarded as special application pavements.

A2.3 Traffic loading estimates

Available data is used as a basis for estimating the ADE of existing traffic loading. Where only AADT figures are available, the formula given in equation A2.1 below can be used to provide an initial estimate of the ADE.

$$ADE = AADT \times f_H \times f_E \times f_L \times f_G \times f_W$$ [equation A2.1]

where:

f_H = percentage heavy vehicles in the traffic spectrum;
f_E = estimated average ESAL per heavy vehicle; and
f_L = lane distribution factor (see Table A2.3)
f_G = gradient factor (see Table A2.4)
f_W = lane width factor (see Table A2.5)

Determining the relevant "lane distribution factor" requires careful consideration, especially for multi-lane divided highways where the slow-lane invariably carries a higher number of heavy vehicles than the middle- or fast-lanes.

Table A2.3 Lane distribution factor (example) f_L

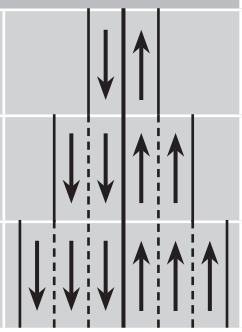
Total lanes (Both directions)		Slow lane	Middle lane	Fast lane
2		0.5		
4		0.48		0.15
6		0.35	0.3	0.13

Table A2.3 gives guidelines based on traffic patterns. These factors are likely to vary from country to country as they typical reflect driver habits, law enforcement practices, etc. The lane-split data in Table A2.3 reflects normal traffic patterns for inter-city highways. Roads that are predominantly unidirectional haul routes (e.g. farm-to-market and mine-to-railhead roads) will obviously not fit this pattern.

The "gradient factor" takes cognisance of the increase in effective wheel loading due to the reduction in speed of heavy vehicles crawling up steep slopes whilst the "lane width factor" recognises the load concentrating effect of confining traffic. Table A2.4 and A2.5 include the factors recommended by the German highway authorities in their publication "RStO 01"

Table A2.4 Gradient factor

Gradient (%)	f_G
Less than 2	1.0
2 to 4	1.02
4 to 5	1.05
5 to 6	1.09
6 to 7	1.14
7 to 8	1.20
8 to 9	1.27
9 to 10	1.35
More than 10	1.45

Table A2.5 Lane width factor

Lane width	F_W
Less than 2.5 m	2.0
2.5 m to 2.75 m	1.8
2.75 m to 3.25 m	1.4
3.25 m to 3.75 m	1.1
More than 3.75 m	1.0

(The design codes in some countries exclude the Lane Width Factor since channelised flow is assumed. F_w is therefore a constant 1.0)

Estimates based on AADT, average percentage heavy vehicles and average number of ESALs per heavy vehicle (F_E) should be treated with care. They are purely estimates and should not be used as primary input for the design of major pavements. When there is any doubt about the accuracy or relevance of the traffic data available, a detailed traffic survey should be undertaken to determine the "correct" ADE per lane. Various survey methods may be employed, including:

A2.3.1 Traffic counts

These are normally carried out at specific points (such as intersections, etc) over a 12-hour, 18-hour or 24-hour period. The actual count may then be converted to 24-hour periods by applying appropriate factors to obtain the AADT, where required. The number of heavy vehicles is normally expressed as a percentage of AADT.

Traffic counts should include the total number of vehicles per day travelling in each lane (in each direction) separated into vehicle types. The road category (e.g. major highway or rural farm access road) and available technology will normally dictate whether the count is conducted electronically. Sophisticated methods are currently available for counting the number of axles (and even weighing-in-motion, see below), but these are expensive and therefore only used on important highways. Physical counts are still the most popular and, depending on the level of competence of the people employed, reliable visual observations can simultaneously be obtained to confirm certain assumptions and ensure a more accurate traffic loading prediction. The following details should be assessed from observations:

- freight on vehicles (empty, half loaded or fully loaded) and nature of loads;
- vehicle type and number of axles per heavy vehicle;
- land use and current development trends; and
- possible "traffic attraction" once the road is completed.

These observations assist in assigning ESALs per vehicle for the various vehicle types (shown in Table A2.2) as well as improving the accuracy of traffic growth predictions.

Once the surveys are complete, the ADE is determined for each lane using the formula:

$$ADE = \Sigma(n_J \times (F_E)_J)$$ [equation A2.2]

where:
n_J = number of vehicles for each vehicle type (J) in the traffic spectrum;
$(F_E)_J$ = estimated average ESAL per vehicle for each vehicle type (J). (See Table A2.2)

The degree and extent of overloading is an important statistic since pavement design philosophy is based on the standard axle load. Overloading causes severe pavement damage. Any information on overloading should therefore be obtained, usually from the law enforcement authorities. In the absence of reliable information, it is advisable to make an assessment by conducting a load survey. A representative sample of the heavy vehicles should be physically weighed to determine the number (or percentage) of vehicles that are overloaded, and the degree of overloading. The results can then be extrapolated to the whole population of heavy vehicles.

A2.3.2 Static or dynamic weighing procedures

These are physical measurements taken in the field to determine the range of actual axle loads:

- Static weighing is the stationary weighing of vehicles and is thus limited to a sample of vehicles on a specific road. Care has to be taken that the sample chosen represents the full traffic spectrum and not only those that are laden. Weighing carried out for law enforcement purposes should therefore not be used.
- Dynamic weighing is a continuous weighing-in-motion procedure typically over a seven-day period at a particular site. This method is the most accurate and suitable for traffic estimation and gives the number of axles in each of the predefined axle mass categories. However due to relative high costs, this method is seldom justifiable for minor roads.

Once the surveys are complete, the ADE is determined for each lane using the formula:

$$ADE = \Sigma(n_M \times D_M)$$ [equation A2.3]

where:
n_M = number of axles for each predefined axle mass category (M);
D_M = calculated average ESALs per axle mass category (M).

The average ESALs per axle mass category is calculated from:

$$D_M = (P_M / SAL)^d$$ [equation A2.4]

where:
P_M = axle load in kN for each axle mass category
SAL = relevant Standard Axle Load in kN (e.g. 80 kN)
d = damage coefficient. Dependent on both pavement type and material in the various layers. A value of n = 4 is generally used as an average. Shallow pavements (relatively thin but strong upper layers) have n-values exceeding 4, whereas less sensitive (deep) pavements have n-values of less than 4.

A2.4 Determination of design traffic (structural capacity)

Once the current ADE per lane has been determined, the increase in ADE due to expected traffic growth during the design period is calculated thus:

$$ESALs_{total} = ADE \times f_J$$ [equation A2.5]

where:
$ESALs_{total}$ = structural capacity for design period

$$f_J = \text{cumulative growth factor} = \frac{365 \times (1 + 0.01i) \times [(1 + 0.01i)^y - 1]}{(0.01i)}$$ [equation A2.6]

where:
i = anticipated traffic growth rate in percent
y = number of years in the design life

The cumulative growth factor (f_J) may also be obtained from standard tables, such as the one shown below in Table A2.6.

Table A2.6 Cumulative Growth Factor f_J					
Design life y (years)	f_J for traffic growth of i per annum				
	i = 2%	i = 4%	i = 6%	i = 8%	i = 10%
5	1,937	2,056	2,181	2,313	2,451
8	3,195	3,498	3,829	4,193	4,592
10	4,077	4,558	5,100	5,711	6,399
12	4,993	5,704	6,527	7,481	8,586
15	6,438	7,601	9,005	10,703	12,757
20	9,046	11,304	14,232	18,039	22,996
25	11,925	15,809	21,227	28,818	39,486
30	15,103	21,290	30,588	44,656	66,044

Note:
The data shown in Table A2.6 does not include all values for the variables i and y. Care should be taken when estimating in-between values since direct interpolation or extrapolation may produce an inaccurate result. Equation A2.6 should rather be used to calculate f_J.

When estimating the anticipated traffic growth rate (i), growth influences other than normal economic growth factors should be identified. Upgrading roads and pavement rehabilitation often attracts traffic that would normally use alternative routes.

A2.5 Practical approach to estimating design traffic

The procedures described above should always be followed when determining the structural capacity (design traffic) at project level. However, there is often a need for a rough estimate of the structural capacity requirements for a specific road, to obtain a "ball-park" figure to guide planning at network level as well as assessing the inadequacies of existing pavements. In addition, such estimates are usually made at the start of a project to "get a feel" for the type of pavement required relative to the level of traffic that needs to be accommodated.

Where the number of heavy vehicles is known, a very rough estimate of the structural capacity requirements can be made using Table A2.7. This table relates the "Number of heavy vehicles per lane per day" to "Design traffic" (structural capacity) in terms of ESALs x 10^6, with three variables; compound traffic growth, pavement design life (years) and load factor (average number of ESALs per heavy vehicle).

For example. Where the number of heavy vehicles travelling in one direction of a two-lane road is 100 per day, the average ESALs per heavy vehicle is 2 and the compound traffic growth is 4% per annum, the structural capacity required for a 10 year design life indicated from Table A2.7 is 0.91 x 10^6 ESALs. If the average number of ESALs per heavy vehicle increases to 3.5, the structural capacity requirement increases to 1.6 x 10^6 ESALs.

Cautionary note:
Users of this table must recognise that any structural capacity estimate thus derived has severe limitations and can only be used as an indicator. The major shortcoming is the lack of definition of a "heavy vehicle", all are assumed to have the same number of 80 kN ESALs. Pavement designs should therefore not be based on such information; a proper traffic analysis and forecast following the procedures described in this section should always be undertaken.

Table A2.7. Guide for estimating Design Traffic (Structural Capacity) in millions of equivalent 80 kN standard axle loads (ESALs x 10^6)

No. of heavy vehicles per day	Compound traffic growth	Pavement design life (years)						
		5				10		
		Vehicle load factor (80 kN ESALs per heavy vehicle)						
		0.6	2	3.5	4.4	0.6	2	3.5
10	2%							
	4%							
	6%							
	8%							
20	2%							0.29
	4%							0.32
	6%							0.36
	8%							0.40
50	2%			0.34	0.43		0.41	0.71
	4%			0.36	0.45		0.46	0.80
	6%			0.38	0.48		0.51	0.89
	8%			0.40	0.51		0.57	1.00
100	2%		0.39	0.68	0.85		0.82	1.43
	4%		0.41	0.72	0.90	0.27	0.91	1.60
	6%		0.44	0.76	0.96	0.31	1.02	1.78
	8%		0.46	0.81	1.02	0.34	1.14	2.00
500	2%	0.58	1.94	3.39	4.26	1.22	4.08	7.13
	4%	0.62	2.06	3.60	4.52	1.37	4.56	7.98
	6%	0.65	2.18	3.82	4.80	1.53	5.10	8.92
	8%	0.69	2.31	4.05	5.09	1.71	5.71	9.99
1,000	2%	1.16	3.87	6.78	8.52	2.45	8.15	14.27
	4%	1.23	4.11	7.20	9.05	2.73	9.12	15.95
	6%	1.31	4.36	7.63	9.60	3.06	10.20	17.85
	8%	1.39	4.63	8.09	10.18	3.43	11.42	19.99
3,000	2%	3.49	11.62	20.34	25.57	7.34	24.46	42.80
	4%	3.70	12.34	21.59	27.14	8.20	27.35	47.85
	6%	3.93	13.09	22.90	28.79	9.18	30.60	53.55
	8%	4.16	13.88	24.28	30.53	10.28	34.26	59.96
5,000	2%	5.81	19.37	33.91	42.62	12.23	40.77	71.34
	4%	6.17	20.56	35.98	45.23	13.67	45.58	79.76
	6%	6.54	21.81	38.17	47.98	15.30	51.00	89.24
	8%	6.94	23.13	40.47	50.88	17.13	57.11	99.94

Key: < 0.256 x 10^6 ESALs

Table A2.7. Guide for estimating Design Traffic (Structural Capacity) in millions of equivalent 80 kN standard axle loads (ESALs x 10^6)

	15				20			
4.4	0.6	2	3.5	4.4	0.6	2	3.5	4.4
			0.27 0.32 0.37	0.28 0.33 0.40 0.47		0.28 0.36 0.36	0.32 0.40 0.50 0.63	0.40 0.50 0.63 0.79
0.36 0.40 0.45 0.50		0.26 0.30 0.36 0.43	0.45 0.53 0.63 0.75	0.57 0.67 0.79 0.94		0.45 0.57 0.72 0.90	0.63 0.79 1.00 1.26	0.80 0.99 1.25 1.59
0.90 1.00 1.12 1.26	0.27 0.32	0.64 0.76 0.90 1.07	1.13 1.33 1.58 1.87	1.42 1.67 1.98 2.35	0.27 0.34 0.43 0.54	1.13 1.42 1.80 1.81	1.58 1.98 2.49 3.16	1.99 2.49 3.13 3.97
1.79 2.01 2.24 2.51	0.39 0.46 0.54 0.64	1.29 1.52 1.80 2.14	2.25 2.66 3.15 3.75	2.83 3.34 3.96 4.71	0.54 0.68 0.85 1.08	2.26 2.85 3.61 9.05	3.17 3.96 4.98 6.31	3.98 4.97 6.26 7.94
8.97 10.03 11.22 12.56	1.93 2.28 2.70 3.21	6.44 7.60 9.01 10.70	11.27 13.30 15.76 18.73	14.16 16.72 19.81 23.55	2.71 3.39 4.27 5.41	11.30 14.23 18.04 18.09	15.83 19.78 24.91 31.57	19.90 24.87 31.31 39.69
17.94 20.05 22.44 25.13	3.86 4.56 5.40 6.42	12.88 15.20 18.01 21.41	22.53 26.60 31.52 37.46	28.33 33.44 39.6+2 47.09	5.43 6.78 8.54 10.82	22.61 28.46 36.08 54.28	31.66 39.56 49.81 63.14	39.80 49.74 62.62 79.37
53.81 60.16 67.32 75.38	11.59 13.68 16.21 19.27	38.63 45.61 54.03 64.22	67.60 79.81 94.56 112.39	84.99 100.33 118.87 141.28	16.28 20.35 25.62 32.47	67.82 85.39 108.24 90.46	94.98 118.69 149.44 189.41	119.41 149.21
89.68 100.27 112.19 125.63	19.32 22.80 27.02 32.11	64.38 76.01 90.05 107.03	112.67 133.02	141.64	27.14 33.91 42.70 54.12	113.04 142.32		

> 250 x 10^6 ESALs

Appendix 3 – Guidelines for compiling specifications for recycling projects

Pavement rehabilitation by recycling is a relatively young technology. As a consequence, few standard specifications have been compiled that adequately cover the requirements for construction. These guidelines have been included to assist in the compilation of relevant contract documentation. They follow the normal format and headings of a standard specification, giving guidance and examples of important features that need to be included to avoid the conflict and claim procedures that are normally encountered when specifications are ambiguous.

A3.1 Scope

Specifications for the recycling portion of the works must cover all operations in connection with the construction of a new pavement layer using predominantly the material recycled from upper layers of an existing road. These operations include:

- breaking down and recovering material in the upper layers of existing road pavements;
- changing the nature of the recovered material by the addition of imported material;
- the provision and application of stabilising agents and water; and
- mixing, placing, compacting and shaping to achieve a new pavement layer.

Pavement composition is seldom uniform over long sections of road. In addition to variations in the thickness of individual layers, the quality of the material quality used in the original construction often varies. Maintenance, upgrading and rehabilitation measures that are applied during the service life of the road also introduce further variability into the upper portion of the pavement. Where there is more than one type of recycling operation to be carried out, each needs to be fully described. For example, if the length of the project to be rehabilitated by recycling is 32.9 km and there are seven different pavement types each with a different treatment, these need to be detailed as shown in the example table below.

Table showing the different recycling operations required (example)

No.	Start	End	Length (m)	Recycling depth (mm)	Treatment required for rehabilitation
1	29+900	32+900	3,000	175	Pre-treat with 2% lime. Within 24 hours, stabilise with 3.0% foamed bitumen
2	32+900	35+000	2,100	200	Recycle with 1% cement and 2.7% foamed bitumen
3	35+000	38+600	3,600	150	Add 75 mm layer of crusher dust before recycling with 2.5% foamed bitumen
4	38+600	49+400	10,800	300	Recycle with 3% cement as a new subbase. Import 125 mm screened RAP and recycle 150 mm with 2% foamed bitumen
5	49+400	54+800	5,400	225	Pre-treat with 2% lime. Within 24 hours, stabilise with 3.0% foamed bitumen
6	54+800	61+000	6,200	150	Add 75 mm layer of crusher dust before recycling with 2.5% foamed bitumen
7	61+000	62+800	1,800	300	Recycle with 3% cement as a new subbase. Import 125 mm screened RAP and recycle 150 mm with 2% foamed bitumen

The general description of the pavement to be recycled under the contract, the depth and type of recycling (e.g. 175 mm deep treated with 1% ordinary portland cement and 2.5% bitumen emulsion), as well as the end-product required are must be clearly defined.

In addition, the design requirements for each type of recycled material treated with stabilising agents needs to be specified. This is normally included as a table showing the minimum requirements in terms of strength parameters and density of the compacted layer. The following table shows an example from a typical recycling project that includes both cement and foamed bitumen stabilisation:

Section	Material origin/depth of recycling (mm)	Application rate of stabilising agent (% by mass)		Minimum strength requirements		Minimum density
		Cement	Foamed bitumen	ITS_{DRY} (kPa)	UCS (MPa)	% of mod AASHTO T-180
km1+200 to 2+800	In-situ/250	2.5		200	1.5	98
km1+200 to 2+800	Imported/125	1	2.0	225	n/a	102
km2+800 to 8+600	In-situ/200	1	2.5	175	n/a	100
km8+600 to 12+800	In-situ/300	3.0		200	1.5	98
km8+600 to 12+800	Imported/150	1	2.0	225	n/a	102

The results of mix designs coupled with assumptions made in determining the pavement design are used as guidelines in compiling such a table. The relevant methods to be used for determining strength (test methods) also need to be specified.

A word of caution to those compiling such specifications: The values specified must be attainable in the field and it is therefore important to liaise with those responsible for the field investigations and the design procedures. For example, specifying a density requirement of 104% of the modified AASHTO density for a recycled layer overlying an unstabilised material is impractical. Likewise, specifying a strength requirement similar to the maximum achieved from laboratory mix designs is unrealistic.

In addition, the following clause is normally included to place responsibility where it belongs, namely with the contractor who is in control of the works:

It shall be incumbent on the contractor to organise and execute his operations such that these requirements are met.

A3.2 Materials

A3.2.1 In-situ pavement material

Details of all pavement investigations that were undertaken by those responsible for designing and specifying the rehabilitation requirements need to be shown, normally in an appendix. These investigations include:

- detailed description of the existing pavement structures that are to be recycled;
- the results of tests undertaken, indicating the grading, plasticity and other relevant properties of the material to be recycled from the upper pavement layers; and
- relevant in-situ moisture contents of the various materials in the existing pavement, measured at the time the investigations were undertaken.

The following disclaimer is normally included:

This information is offered in good faith but, in the circumstances pertaining to sampling and testing procedures and the type of information furnished, no guarantee can be given that all the information is correct or representative of the in-situ conditions at the time of construction. Any reliance placed by the contractor on this information shall therefore be at his own risk, and he shall undertake his own separate testing programme to determine the conditions prevailing at the time of construction.

A3.2.2 Imported natural or processed material

Where the project requirements call for imported material to be blended with that recycled from the existing pavement, the reason for such import needs to be given together with clear specifications for the type and quantity of material to be imported. Material is normally imported for one or more of the following reasons:

- to change the grading of the recycled material;
- to effect mechanical modification;
- to supplement the recycled material for shape correction purposes; and/or
- to increase the total pavement thickness.

Contractors need to understand the reasoning behind such requirements in order to formulate their most cost effective offer since this exercise embodies a host of alternatives (e.g. purchase material from a commercial source or establish their own crushing facility).

In addition, any special conditions under which the material is to be imported must be given (e.g. import before or after the existing pavement is pre-pulverised). The type of material (e.g. graded crushed stone with CBR >100% within a specified grading envelope) and amount to be imported must be clear (e.g. 30% by volume of new layer. Alternatively, spread as a nominal 75 mm layer (after compaction) on the existing road surface).

A3.2.3 Stabilising agents

The type and quality of all stabilising agents that are to be used on the project must be clearly specified together with any relevant standards governing their manufacture (e.g. Ordinary portland cement conforming to the requirements of BS 12.) and use. In addition, any special requirements (e.g. handling and storage) must be stated. For example, the following paragraph is normally included when cement is specified as a stabilising agent:

From the time of purchase to the time of use, all cement shall be kept under cover and protected from moisture, all in accordance with the manufacturer's or supplier's recommendations. All consignments of these materials shall be used in the same sequence as that of their delivery on site. Stocks stored in excess of three months shall not be used in the works without authorisation.

Where foamed bitumen stabilisation is specified, the type of bitumen to be used must be specified (e.g. 80/100 Penetration Grade) and the following requirements are normally included under this section of the specifications:

Bitumen for foamed bitumen stabilisation shall be heated, stored and applied strictly in accordance with the requirements detailed below. (List the specific requirements, e.g. maximum temperature of 195°C). All bitumen for stabilisation shall be delivered to the site in bulk tankers. Each bulk tanker shall be issued with a 'Certificate of Loading' that contains the following information:

- haulage unit's identification details;
- product identification (e.g. 150/200 Pen grade bitumen);
- name of the bitumen supplier;
- the relevant batch number and date of manufacture;
- assized weighbridge certificate indicating the net mass of product;
- the temperature at which the product was loaded into the tanker;
- the date, time and place of loading;
- comments concerning any abnormalities of the state of the tanker at the time of loading (e.g. internal cleanliness, details of the previous load carried and whether there was any residual product from the previous load; and
- details of any chemical or other substance added to the product before, during or after the loading procedure (e.g. anti-stripping agent).

A3.2.4 Water for construction

Any limitations on the quality of water to be used in the construction needs to be specified. The following paragraph is normally included:

Water shall be clean and free from detrimental concentrations of acids, alkalis, salts, sugar and other organic or chemical substances. If the water used is not obtained from a public drinking water main, tests may be required to prove its suitability.

A3.3 Plant and Equipment

To protect against poor workmanship resulting from the use of inappropriate machinery, the following paragraphs are typical of those normally included in recycling specifications:

> All plant and equipment shall be supplied and operated in such a manner as to recycle in situ pavement material to the specified depth and construct a new layer, all in accordance with the requirements of the specifications. All plant and equipment deployed on the site shall be of adequate rated capacity and in good working order. Obsolete, poorly maintained, or dilapidated plant will not be allowed on site.
> The minimum compliance requirements for plant and equipment to be used for the recycling work are given in the following sub-clauses. The contractor shall provide the Engineer with details and technical specifications of all the plant and equipment to be used for the recycling work at least two weeks prior to the first proposed usage.

A3.3.1 Recyclers

The following is normally included to ensure that the recycler used on site is adequate:

Recycling shall be effected by utilising a purpose-built recycler to recover the material in the upper layers of the existing pavement and blend together with any imported material pre-spread as a uniform layer on the existing road surface. The machine employed shall be capable of achieving the required grading and consistency of mix in a single pass. As a minimum, the recycler shall have the following features:

- It shall be factory-built by a proprietary manufacturer having a demonstrable track record and manufacturing history in the particular type of equipment;
- If older than 10 years, the machine shall be certified by the manufacturer or manufacturer's authorised agent to confirm operational fitness-for-purpose dated not more than 3 months earlier than the date on which it commences work on the project;
- The milling drum shall have a minimum cut width of 2 metres with the capability of changing the speed of rotation. The machine shall be capable of recycling to a maximum depth (specified in these documents) in a single pass;
- A level-control system that maintains the depth of milling within a tolerance of ± 10 millimetres of the required depth during continuous operation;
- The milling drum shall rotate within an enclosed chamber inside which water and stabilising agents are added to the recovered material at the rate required to achieve compliance with the specified requirements during a continuous operation.
- All spray systems fitted to the recycler shall be controlled by micro-processor to regulate the flow rate with the speed of advance of the machine. All spray systems will also have the ability to allow variable widths of application; and
- The recycler shall have sufficient power to mix the recycled material together with all additives to produce a homogenously mixed material during continuous operation.

Additional specifications are included to cover the type of stabilising agent that is applied. The following are examples for stabilising with cement, bitumen emulsion and foamed bitumen.

Additional requirements when stabilising with cement.
Where the cement stabilising agent is not applied directly on the surface of the road prior to recycling, the recycler shall be fed with cement slurry that is produced in a separate mobile mixing unit pushed ahead of the recycler. Such a mixing unit shall have the following minimum features:

- the capability of supplying the cement slurry at the required rate to comply with the specified cement application rate during continuous operation;
- capable of regulating the application rate of cement slurry in accordance with the speed of advance of the recycler and volume of material during continuous operation;
- provide uniform application of cement slurry to the recycled material to produce a homogenous mixture; and
- a micro-processor controlled method for monitoring cement usage during operation that can be validated by simple physical measurement for control purposes.

Additional requirements when stabilising with bitumen emulsion.
In addition, the recycler shall have the following capabilities:

- To supply the bitumen emulsion at the specified application rate during continuous operation;
- To regulate the application rate of bitumen emulsion in accordance with speed of advance of the recycler and volume of material being recycled;
- To provide uniform application of the bitumen emulsion to the recycled material to produce a homogenous mixture; and
- A method for monitoring bitumen emulsion application during operation that can be reconciled by simple physical measurement for control purposes.

Additional requirements when stabilising with foamed bitumen.
In addition, the recycler shall have the following features:

- a series of expansion chambers mounted equidistant on the spraybar (maximum spacing 200 mm) for creating the foamed bitumen;
- the capability of providing a constant supply of foamed bitumen at the specified application rate during continuous operation;
- capable of regulating the quality of foamed bitumen and regulating the application rate in accordance with speed of advance of the recycler and volume of material being recycled;
- provide a uniform application of the foamed bitumen across the width of application to produce a homogenous mixture;
- a method for monitoring bitumen application during operation that can be reconciled by simple physical measurement for control purposes;
- functioning temperature and pressure gauges on the bitumen supply line for monitoring purposes;

- a means of demonstrating that all expansion chambers are producing foamed bitumen at any time during the operation (no blockages); and
- a means of providing a representative sample of foamed bitumen at any stage during normal operations (test nozzle).

The following clause is normally added at the end of this section:

The mixed material shall exit from the mixing chamber in a manner that prevents particle segregation and be continuously placed back in the excavation created by the recycler as it advances. Spreading and placing to form the new layer shall be carried out by a motor grader only after the primary compaction has been achieved (unless placed by a screed mounted on the rear of the recycling machine).

A3.3.2 Equipment for compaction and finishing

To prevent the incorrect application of compaction equipment and guard against the “bridging” phenomenon, it is advisable to include the following:

Initial compaction of the recycled material shall be undertaken using a single-drum vibrating roller operated only in high-amplitude vibration mode. The static mass of the roller to be used shall be in determined by the thickness of the recycled layer, in accordance with the following table:

Thickness of compacted layer	Minimum static mass of roller (tons)	Drum type
< 150 mm	12	smooth
150 mm to 200 mm	15	smooth or padfoot
200 mm to 250 mm	18	padfoot
> 250 mm	20	padfoot

The operating speed of the primary roller shall never exceed 3 km/hr and the number of passes applied over the full width of each cut shall be sufficient to achieve at least the specified layer density in the lower two-thirds of the layer. Where a compactometer is specified for controlling density, it shall be fitted to the primary roller.

A3.3.3 Tankers for the supply of bitumen stabilising agents

The following clause should be included when stabilising with either bitumen emulsion or foamed bitumen:

Only tankers with a capacity exceeding ten thousand (10,000) litres shall be employed to supply the recycler with bitumen stabilising agents. Each tanker shall be fitted with two recessed pin-type tow hitches, one in front and the at the rear, thereby allowing the tanker to be pushed from behind by the recycler, and to push a water tanker in front. No leaking tanker will be permitted on the site. In addition, each tanker shall be equipped with:

- A functioning thermometer to show the temperature of the contents in the bottom third of the tank; and
- A rear feed valve, with a minimum internal diameter of 75 mm when fully opened, that is capable of draining the contents of the tank.

Where foamed bitumen is applied, the following additional bullets are included:

- all-round cladding to retain heat; and
- a heating system capable of raising the temperature of the contents of the tank by at least 20° C per hour.

A3.4 Construction

Typical clauses normally included in this section are reproduced below.

A3.4.1 General limitations and requirements

Weather Limitations

No work shall be undertaken during misty or wet conditions, nor shall any work commence if there is a risk that it may not be complete before such conditions set in. Similarly, work shall not be undertaken if the ambient air temperature is below 5° C. No further work, other than finishing and compaction, will be permitted if the air temperature falls below 10° C during operations.

Spreading of powdered chemical stabilising agents (lime and cement) on the road ahead of the recycler shall not be permitted when windy conditions adversely affect the operation.

Accommodation of Traffic

The contractor shall be responsible for the comfortable passage of public traffic over sections of the road on which he has occupation and shall at all times take the necessary care to protect the public and to facilitate the flow of traffic.

Time Limitations

The maximum time period between mixing the recycled material with a stabilising agent and compacting the placed material shall be determined by the type of stabilising agent that is employed:

– cement:	Three (3) hours;
– hydrated lime:	Eight (8) hours if kept moist;
– bitumen emulsion:	24 hours (or before the emulsion breaks);
– foamed bitumen:	24 hours; and
– proprietary products:	As per manufacturer's instructions.

A3.4.2 Requirements before recycling commences

Production Plan

Prior to the start of work every day, the contractor shall prepare a production plan detailing his proposals for the forthcoming day's work. As a minimum, this plan shall include:

- a sketch plan showing the overall layout of the length and width of road intended to be recycling during the day, broken into the number of parallel cuts required to achieve the specified width and the overlap dimensions at each longitudinal joint between cuts;
- the sequence and length of each cut to be recycled before starting on the adjacent or following cut;
- an estimate of the time required for milling, compacting and finishing each cut. The time required to recycle each cut should be indicated on the sketch plan; and
- the location where quality assurance tests are to be taken.

Unless stated to the contrary, longitudinal joints shall be planned to coincide with each and every change in cross-fall across the road width, regardless of the implications on overlap width.

Referencing the horizontal alignment

Prior to commencing with the recycling work, the existing horizontal alignment shall be referenced using a series of pegs (or poles) placed on either side of the road. These pegs (or poles) shall be positioned outside the working area at a constant distance from, and at right-angles to the centre-line, and shall be used to reinstate the centre-line after recycling operations are complete. The distance between successive pegs (or poles) shall not exceed 20 m on curves, or 40 m on tangents (straights).

Preparing the Surface

Before any recycling work commences, the surface of the existing road shall be prepared by:

- cleaning all vegetation, garbage and other foreign matter from the full road width, including any adjacent lanes or shoulders that are not to be recycled;
- removing any standing water;
- premilling where high-spots are to be removed (if required); and
- accurately pre-marking the proposed longitudinal cut lines on the existing road surface.

In addition, the contractor shall record the location of all road marking features (e.g. extent of barrier lines) that will be obliterated by recycling.

Surface shape and level requirements

Unless otherwise stated, design drawings will not be issued detailing the final level requirements for the surface of the rehabilitated road. Where the grade line and cross-sectional shape of the existing road are not excessively distorted, it shall be the contractor's responsibility to conduct his operations in such a manner as to ensure that the surface levels of the completed recycled layer are in sympathy with those that existed prior to recycling. Where surface defects are to be corrected and/or modifications made to the grade line, instructions will be issued detailing the new surface level requirements. These may be achieved prior to recycling by either premilling to remove in-situ material or by importing material and accurately spreading on the existing road surface.

Addition of imported material

Where the design calls for material to be imported as make-up material for the purpose of shape correction, the prescribed material shall be imported and spread on the existing road surface prior to recycling. The method of placing and spreading the imported material shall be such as to achieve the required surface levels and may therefore require the use of a paver, motor grader or other such plant. Should the thickness of imported material exceed the intended recycling depth, then the requirements for shape correction will have to be modified by regrading the road surface on either side of the low point.

Where the design calls for material to be imported for the purpose of altering the grading of the recycled material, or effecting mechanical modification, the prescribed material shall be imported and spread on the surface of the existing road as a layer of uniform thickness prior to recycling.

Pre-milling

Where required, pre-milling shall be undertaken using a milling machine (not a recycler) to:

- Remove material from the road. Isolated high spots shall be removed and/or minor modifications made to crest vertical curves by accurate milling. The material resulting from such milling operations shall be loaded onto trucks and removed from site.
- Break down (pulverise) thin layers of asphalt. Badly cracked asphalt layers (full-depth crocodile cracks at intervals < 100 mm), and/or sections where thin asphalt overlays are delaminating, shall be pre-milled immediately in advance of the recycling operation. To ensure that the milling operation achieves the required degree of pulverisation, the depth of milling shall be constantly monitored and adjusted so that the bottom of the milling drum remains within the lower half of the cracked / delaminating asphalt layer.

  The pulverised asphalt material generated from such pre-milling shall remain on the road, behind the milling machine, where it shall be spread across the width of recycling and rolled with a smooth-drum roller.

Where an acceptable degree of pulverisation cannot be achieved, the machine shall be operated in reverse (i.e. down-cutting) with the same controls applied to the depth of milling. If such reverse milling fails to produce an acceptable degree of pulverisation, the offending asphalt layer shall be milled off and removed.

Pre-pulverising existing pavement material

Pre-pulverising shall only be undertaken for the purpose of:

- breaking down excessively hard material;
- loosening the material across the road width so that it can be cross-mixed by grader;
- exposing the loosened (fluffed-up) material to the atmosphere to promote drying; or
- loosening the material in the existing pavement so that it can be loaded and removed from site.

The depth of pre-pulverising shall be carefully controlled throughout the operation to ensure that the cut horizon always remains at least 50 mm above the bottom of the subsequent recycling / stabilisation horizon.

Unless the objective of pre-pulverising is to dry the material, a water tanker shall be coupled to the recycler and sufficient water added to allow the material to be compacted to a minimum density of 95% of the mod AASHTO density. Except where the material is to be cross-mixed, it shall be compacted immediately behind the recycler before using a grader to pre-shape the material in accordance with final level requirements. Where cross-mixing is ordered, the material shall be bladed by grader across the specified width to achieve a uniform blend of material before being compacted and shaped.

A3.4.3 Addition of stabilising agents

The type of stabilising agent and the required application rate, expressed as a percentage of the mass of the material to be stabilised, will normally be determined from mix design tests carried out prior to the work commencing and issued to the Contractor as an instruction.

Chemical stabilising agents (cement and lime)

The method of applying chemical stabilising agents shall be at the contractor's discretion and may be either:

- spread as a uniform layer of dry stabilising agent on the prepared road surface prior to recycling; or
- fluidised as a slurry by premixing with water and pumped to the recycler for injection through a spray-bar into the mixing process; or
- premixed in a batch plant and spread on the road surface together with any imported material.

Dry stabilising agents shall be spread uniformly over the full width of road to be recycled during each pass of the recycler, either by means of a mechanical spreader at the prescribed rate of application as a continuous process, or by hand. Where spreading is done by hand, pockets or bags of the stabilising agent shall be spaced at equal intervals along each individual cut. The bags shall be emptied and the contents spread evenly over the entire area of cut, excluding any overlap.

Mechanically operated mixers shall be used for the manufacture of slurry from dry powdered stabilising agents and water. The mixer shall be equipped with a screen with openings not exceeding 5 mm and shall be capable of producing a slurry of uniform consistency and constant water content at the rate required for stabilisation.

Bitumen stabilising agents

The bitumen stabilising agent shall be added to the recycling process by pumping from a mobile bulk tanker that is pushed ahead of the recycler. Where foamed bitumen is applied, tankers shall be equipped with a built-in thermometer and heating facilities to ensure that the bitumen is maintained within 5° C of the specified application temperature. Any bitumen that has been heated above the maximum specified temperature shall not be used and shall be removed from the site.

A one-litre sample of bitumen stabilising agent shall be taken from each tanker load and retained in a sealed tin as a provision for later testing.

The following sentence should be included when working with foamed bitumen:

Not later than two (2) minutes after starting to recycle with each new tanker load, the foaming characteristics of the bitumen shall be checked using the test nozzle on the recycler.

Addition of fluid stabilising agents

The pumping system required to inject a fluid stabilising agent into the mixing process shall be controlled by the same micro-processor system that monitors travel speed for the control of water addition.

Controlling the moisture content of recycled material

Sufficient water shall be added during the recycling process to meet the moisture requirements specified below. Water shall be added only by means of the micro-processor control system on the recycler and particular care shall be taken to prevent any portion of the work from excessive wetting. Any portion of the work that becomes too wet will be rejected and the contractor shall be responsible for correcting the moisture content by drying out and reprocessing the material, together with fresh stabilising agent where cementitious stabilising agent is employed, all at his own expense.

At the time of compaction, the type of stabilising agent applied shall govern the moisture content of the recycled material:

i) Cementitious stabilising agents

The moisture content during compaction shall never exceed 75% of the saturation moisture content of the natural material (before stabilising), calculated at Maximum Dry Density. The moisture content at the specified degree of saturation shall be determined using the following formula:

$$W_v = S_r \times \{(X_w / X_d) - (1{,}000 / G_s)\}$$

where:

W_v = moisture content of the material at the specified degree of saturation (%)
S_r = specified degree of saturation (%)
X_w = density of water (kg/m^3)
X_d = maximum dry density of the natural material (kg/m^3)
G_s = apparent density of the material (kg/m^3)

ii) Non-cementitious stabilising agents and material recycled without stabilising agents
The moisture content during compaction shall not exceed the optimum moisture content, nor shall it be less than 70% of the optimum moisture content.

iii) Bitumen emulsion stabilising agents
The total fluid content of the material during compaction shall not exceed the optimum fluid content. The total fluid content shall be determined by summing the total amount of bitumen emulsion applied (not only the water fraction) to the in-situ moisture content before mixing, plus any other water applied independent of the water fraction of the emulsion.

A3.4.4 Recycling

The recycling machine shall be set up and operated to ensure the following key requirements are met:

- **Grading of the recycled material**
  The forward speed of the recycling machine, rate of rotation of the milling drum and the positioning of the gradation control beam shall be set so that the in-situ material is broken down to an acceptable grading. The contractor shall take all necessary steps to ensure that the grading that results from the recycling process conforms to those established during the Trial Section, as described in Clause A3.5 below.

- **Addition of water and fluid stabilising agents**
  The micro-processor control system for the addition of water and fluid stabilising agents shall be set and carefully monitored to ensure compliance with the requirements for compaction moisture and stabiliser content. Where practical, bulk bitumen tankers shall be dipped at the end of each cut to check actual usage against the calculated theoretical demand.

- **Control of cut depth**
  The actual depth of cut shall be physically measured by dipping from a stringline pulled between survey control poles at least once every 100 m along the cut length using the same references on the survey control poles that will be used to achieve final surface levels.

- **Overlap on longitudinal joints**
  To ensure complete recycling across the full width of the road, longitudinal joints between successive cuts shall overlap by a minimum of 150 mm. Cut lines pre-marked on the road surface shall be checked to ensure that only the first cut is the same width as the milling drum. All successive cut widths shall be narrower than the drum width by at least 150 mm. The recycling machine shall be steered so as to

accurately follow the pre-marked cut lines. Any deviation in excess of 100 mm shall be rectified immediately by reversing to where the deviation commenced and reprocessing along the correct line, without the addition of any further water or stabilising agent.

The overlap width shall be confirmed before starting each new cut sequence and any adjustments made to ensure that the amount of water and fluid stabilising agent to be added is reduced proportionately by the width of the overlap.

- **Continuity of stabilisation (lateral joints)**
  The contractor shall ensure that between successive cuts (along the same longitudinal cut line) no gaps of unrecycled material remain, nor are any untreated wedges created where the milling drum first enters the existing material. The exact location at which each cut terminates shall be carefully marked. This mark shall coincide with the position of the centre of the mixing drum at the point at which the supply of stabilising agent ceased. To ensure continuity of the stabilised layer, the next successive cut shall be started at least 0.5 m (500 mm) behind this mark.

- **Speed of advance**
  The speed of advance shall be checked and recorded at least once every 200 m of cut to ensure conformity with the planned production rate and ongoing compliance with the recycling process. Acceptable tolerance limits shall be dependent on the type of recycler and material being recycled, but shall not be less than 6 m/min nor greater than 12 m/min.

A3.4.5 Subgrade instability

Where subgrade instability is identified either by preliminary investigations, or during the recycling process, it shall be treated by:

- recovering the material in the pavement layers overlying the unstable material by either milling or excavating and loading into trucks for transport to temporary stockpile;
- excavating the unstable material to the prescribed depth, and removal to spoil;
- treating the exposed roadbed, as specified; and
- backfilling the excavation using both temporary stockpiled and imported material.

Backfilling shall be undertaken in layers not thicker than 200 mm after compaction, and shall continue in successive layers utilising appropriate material for each layer. When backfilling is complete, recycling shall continue.

A3.4.6 Compaction and finishing

Initial compaction

The recycled material shall be rolled initially using a heavy vibrating roller to achieve the specified compaction. Rolling shall commence immediately behind the recycler and shall follow the predetermined sequence as described in Clause A3.5 below.

Density control shall be applied to:

– the average density achieved over the full layer thickness; and
– the density of the lowest two-thirds of the layer, which shall not be less than the average density for the full layer thickness, minus 2%.

Alternatively, where refusal density is specified, the following should be included:

The recycled material shall be rolled initially using a heavy vibrating roller fitted with an integrated compactometer system. Compactive effort shall be applied to the recycled material immediately behind the recycler in high amplitude vibrating mode only. Successive sections, each no longer than 100 m, shall be compacted and rolling on each section shall continue until the integrated compactometer system indicates that the maximum density achievable has been reached before the roller moves forward to the next section.

Level and shape control

Processed material shall be spread by the recycler to fill the cut void. Such spreading may be achieved either by a screed attached to the rear of the recycler (track-mounted machines), or by applying sufficient pressure to the rear door of the recycling chamber to ensure that the material spreads across the full cut width (tyre-mounted machines). The spread material shall then be initially compacted (as described above) before cutting final levels with a motor grader using references on the survey control poles as a guide.

Final compaction, watering, finishing and curing

After final shaping, a smooth-drum vibrating roller operating in low amplitude vibration mode shall complete the compaction process. The road surface shall then be treated with a light application of water, or diluted bitumen emulsion where specified, and rolled with a pneumatic-tyred roller to achieve a close-knit texture. The surface of the completed recycled layer shall be kept continuously damp by frequent light watering.

The final completed layer shall be free from:
– surface laminations (or biscuits);
– portions exhibiting segregation of fine and coarse aggregate; and
– corrugations, or any other defects that may adversely affect the performance of the layer.

A bitumen surfacing (asphalt or seal) shall not be applied to the surface until the moisture content of the material in the upper 100 mm horizon of the recycled layer is below 50% of the optimum moisture content of the material.

- **Opening to traffic**
  Unless otherwise instructed, the full road width shall be opened to traffic outside normal daylight working hours. All temporary road signs, delineators and other traffic control facilities shall be in place before the road is opened to traffic.

A3.5 Trial sections

Trial sections should always be specified since they force the contractor to rehearse before commencing with the recycling work. The following clause is usually included:

At the start of the project the contractor shall assemble all items of plant and equipment that he proposes to use for cold in-place recycling, and shall process the first section of road to be rehabilitated in order to:

- demonstrate that the equipment and processes that he proposes to employ are capable of constructing the recycled layer in accordance with the specified requirements;
- determine the effect on the grading of the recycled material by varying the forward speed of the recycling machine and the rate of rotation of the milling drum; and
- determine the sequence and manner of rolling necessary to obtain the minimum compaction requirements.

A Trial Section shall be at least 200 m in length of full lane-width or half-road width. Should the contractor make any alterations in the methods, processes, equipment or materials used, or if he is unable to comply consistently with the specifications due to changes in the in-situ material, or for any other reason, he may be required to undertake further demonstrations before continuing with the permanent work.

A3.6 Protection and maintenance

The following standard clause is usually included:

The contractor shall protect and maintain the completed recycled layer until the next layer or surfacing is applied. In addition to frequent light watering to prevent the surface from drying out, maintenance shall include the immediate repair of any damage to or defects in the layer and shall be repeated as often as it is necessary. Repairs shall be made so as to ensure that an even and uniform surface is restored after completion of the repair work. The cost of such repairs shall be borne by the contractor except where the damage is the result of fair wear and tear from early trafficking. Damage caused by prolonged trafficking as a result of a delay in applying the next layer or surfacing shall not be deemed to be fair and tear if such a delay was due to circumstances under the control of the contractor.

A3.7 Construction tolerances

Construction tolerances specified are usually similar to those for new construction, as shown in the following example:

Only where premilling to achieve the prescribed shape prior to recycling is specified will both surface levels and layer thickness be subjected to statistical analyses. The completed recycled layer shall comply with the construction tolerances given below:

A3.7.1 Surface levels

The following is a typical specification for surface levels:

A lot size shall be at least 50 levels taken in a random pattern. The lot will comply with the requirements specified if it meets the following tolerances:

- $H_{90} \leq 20$ mm (ie. at least 90% of all surface levels measured are within 20 mm, plus or minus, of the specified levels); and
- $H_{max} \leq 25$ mm (ie. individual spot levels shall not deviate by more than 25 mm from the specified levels).

A3.7.2 Layer thickness

Since the thickness of the stabilised layer is one of the most important determinants of ultimate performance, relatively tight tolerances should be specified, as shown in the following example:

A lot size shall be at least 20 layer thickness measurements. The lot will comply with the requirements specified if it meets the following tolerances:

- $D_{90} \geq 10$ mm (ie. at least 90 % of all thickness measurements are equal to or greater than the specified thickness minus 10 mm);
- $D_{mean} \geq D_{spec} - (D_{spec}/20)$ (ie. the mean layer thickness for the lot shall not be less than the specified layer thickness, minus the specified layer thickness divided by twenty); and
- $D_{max} < 15$ mm (ie. no individual layer thickness measurement shall be less than the specified thickness minus 15 mm).

A3.7.3 Widths

Nowhere shall the width of the recycled layer be less than the width specified.

A3.7.4 Cross-section

When tested with a 3 m straightedge laid at right-angles to the road centre-line, the surface shall not deviate from the bottom of the straightedge by more than 10 mm. At any cross section, the difference in level between any two points shall not vary from their difference in level computed from the required cross-section by more than 15 mm.

A3.7.5 Surface regularity

The following is a typical specification for surface regularity:

When testing the finished recycled layer with a standard rolling straightedge, the number of surface irregularities shall not exceed:

- six (6) for the average number of irregularities per 100 m equal to or exceeding 6 mm when taken over 300 m to 600 m lengths; and
- eight (8) for the number of irregularities equal to or exceeding 6 mm when taken over a single 100 m section.

Any individual irregularity measured with the rolling straightedge, or a 3 m straightedge laid parallel to the road centre-line, shall not exceed 10 mm. However, where it can be shown that irregularities are caused by factors beyond the control of the contractor (e.g. damage caused by early trafficking), this requirement may be relaxed.

A3.8 Routine inspection and tests

The onus rests with the contractor to produce work that conforms in quality and accuracy of detail to all the requirements of the specifications and drawings. The contractor is therefore responsible for instituting a quality control system to ensure positive control of the works and to demonstrate that specified requirements have been met.

Excluding the geometric details (described above under “Construction Tolerances”), a further three key parameters of the recycled work requires careful control, namely:

- the application rate of stabilising agents;
- strength achieved in the recycled / treated material; and
- density achieved in the new recycled layer.

These are dealt with below in separate sub-sections.

A3.8.1 Application rate of stabilising agents

Due to variability in existing distressed pavements, recycled material is seldom homogeneous. Normal tests to confirm that the required amount rate of stabilising agent has been applied are therefore often inappropriate as they produce misleading results. For example, calculating the bitumen content as a percentage by mass of a material recycled with foamed bitumen cannot provide the answer to how much foamed bitumen was actually added because:

- the amount of bitumen extracted from such a mix is likely to vary considerably due to the presence of bitumen in the existing pavement (in existing asphalt, additional surfacing layers and in patches);
- expressing bitumen content as a percentage of the total mass of the mix is a concept borrowed from asphalt technology as it assumes the aggregate conforms to a uniform grading. Since recycled material has a variable grading, the inclusion or exclusion of one large piece of aggregate will significantly change the percentage of bitumen in the sample, making the result meaningless.

Alternative control measures are therefore required and these are normally based on consumption checks, or physical measurements of the actual quantity of stabilising agent applied compared with the specified application rate.

Cement stabilisation

Where cement is spread by hand on the existing road surface prior to recycling, checking the application rate is relatively straightforward provided the surface is pre-marked for individual bags.

Where a spreader is used to apply the cement on the existing road surface prior to recycling, the standard “canvas-patch” test, or similar, is normally used to check the application rate.

Where cement is applied by means of slurry injection, the actual consumption of cement (and water) can be obtained from the computer that controls the slurry mixing unit. In addition, weighbridge certificates should be obtained from bulk cement deliveries and verified against daily usage indicated by the computer.

Bitumen stabilisation

The actual consumption of both bitumen emulsion and foamed bitumen is best controlled by ensuring that all tankers delivering product to the recycler are provided with weighbridge certificates for each load. The mass of material stabilised with each tanker (estimated from the length of cut multiplied by the application width, depth of cut and assumed material density) cut then be reconciled with the amount consumed, provided always that the tanker is drained.

A3.8.2 Strength of the stabilised material

A bulk sample (±200 kg) is normally taken from behind the recycler, at least one per 2,500 m^2 of pavement recycled. This material is placed in a sealed container and taken immediately to the laboratory for testing. Normal tests include:

- moisture content;
- moisture/density relationship to determine the maximum dry density (used also for determining the percentage compaction achieved in the field;
- strength determination from manufactured briquette specimens. (Note that the moisture content of the material is always adjusted to the optimum moisture content before manufacturing the briquettes with standard compactive effort. In addition, the temperature of the material being compacted should be similar to that in the field.)

Other tests that are sometimes required include sieve analyses and plasticity determination.

Where there is reason to suspect that the required strength has not been achieved, a 150 mm diameter core can be extracted from the layer and tested. This is usually undertaken some 2 to 4 weeks after the layer was constructed.

See Appendix 1 for detailed test procedures.

A3.8.3 Density achieved

Determining the field density of a layer constructed from recycled material is seldom a straightforward exercise due to two features of recycled material:

- variability of the recycled material that affects the maximum dry density value against which field density is compared; and
- bitumen in that portion of the material recycled from existing asphalt and/or bitumen surfacing that affects the moisture content reading of nuclear gauges.

Bulk samples therefore need to be taken at each and every test location and tested in the laboratory to determine the maximum dry density of the material and the actual field moisture content. This invariably increases the workload of the site laboratory and often causes a delay in obtaining results.

In addition, where the specified density is not achieved, there may be reason to suspect that poor underlying support may be a contributing factor making it practically impossible to obtain a higher density than that already achieved. This is a potential source of conflict that requires additional tests to resolve (normally a limited DCP survey).

The integrated compactometer system fitted to the primary roller has recently been adopted on recycling projects to indicate when refusal density has been achieved. This simple system offers a solution to the problems described above and is therefore recommended for controlling field density. The following is an example of a specification for such equipment:

The density required is the “refusal density” and shall be defined as the maximum density achievable in the field, as indicated by an integrated compactometer system. Such a system shall be fitted to the single-drum vibrating roller used for initial compaction behind the recycler. Rolling shall continue until the unit indicates that no further densification is being achieved under additional passes of the roller.
This information shall be stored in the system’s computer, downloaded on a daily basis and used to generate a comprehensive compaction record indicating the level of compaction that was actually achieved every 2 m of cut, together with proof that maximum densification had been achieved.

Check tests using normal test methods are still required on the completed layer, but at a much-reduced intensity, normally 6 tests per day’s work.

A3.9 Measurement and payment

To avoid any arguments, the following should always be included:

The description of certain pay items states that quantities will be determined from 'authorised dimensions'. This shall be taken to mean the dimensions as specified or shown on any drawing or written instruction given to the contractor, without any allowance for tolerances. If the work is constructed in compliance with the authorised dimensions, plus or minus any tolerances allowed, quantities will be calculated from the authorised dimensions regardless of the actual

A3.9.1 Measurement items

Each item to be used for the measurement (and payment) of the recycling work requires a full definition. The following are examples of typical items:

Item	Unit
A3.01 Prepare the existing road surface prior to recycling	square metre (m^2)

The unit of measurement shall be the square metre of existing road surface that is to be rehabilitated by recycling, calculated from the authorised width dimension multiplied by the actual length as measured along the centre-line of the road.

The rate tendered shall include full compensation for all work necessary to clean the road of all water, vegetation, garbage, and other foreign matter and for the removal, transporting and disposal of all resulting debris, as specified.

Item	Unit
A3.02 Recycling in all in-situ pavement materials	
for the construction of new pavement layers:	
a) .....mm (specify) thick completed layer:	
i) Road width of 5.0 m or less	cubic metre (m^3)
ii) Road width greater than 5.0 m, but less than 6.0 m.	cubic metre (m^3)
iii) etc. for road width increments of 1 m.	
b) etc. for each specified layer thickness.	

The unit of measurement shall be the cubic metre of completed pavement layer constructed by recycling the in-situ pavement material, regardless of the hardness or type of such material, and with or without the inclusion of imported material. The quantity shall be calculated from the authorised dimensions for width and thickness of the completed layer, multiplied by the actual length as measured along the centre-line of

the road. The authorised width shall not be increased to include any allowance for the specified minimum overlap between adjacent cuts, nor for the number of cuts required to cover the full road width.

The rates tendered shall include full compensation for setting out the works, for recycling all types of material in the existing pavement structure to the specified depth together with any stabilising agents and/or imported material that may have been incorporated, for the supply and addition of water, for mixing, placing and compacting the material, for the reworking of all material in overlapping adjacent cuts regardless of the number of cuts or width of overlap necessary to cover the full road width, for all curing, protection and maintenance of the layer, and for conducting all process and acceptance control inspections, measurements and tests.

Item	Unit
A3.03 Extra over Item A3.02	
for layers of asphalt material within the recycled portion of the existing pavement, where the average thickness of asphalt is:	
a) More than 50 mm, but less than or equal to 75 mm	cubic metre (m^3)
b) More than 75 mm, but less than or equal to 100 mm	cubic metre (m^3)
c) Etc. for increments of 25 mm.	

The unit of measurement shall be the same as for item A3.02, the extra over measurement applying to the full layer thickness regardless of the relative proportions of asphalt and other material that constitutes the material in the total thickness of the milled layer. No additional payment shall be made where the asphalt thickness is less than or equal to 50 mm.

The rates tendered shall include full compensation for all additional direct and indirect costs incurred as a result of recycling material that includes layers of asphalt thicker than 50 mm. These additional costs shall include, but shall not be limited to, the extra wear and tear on plant and equipment, additional ground-engaging tools, additional costs incurred where the rate of advance dictates that the pavement is first premilled before stabilising, and all allowances for delays caused by the resulting slow rate of production.

Item	Unit
A3.04 Imported material for addition to the cold in-place recycling process:	
a) Crushed-stone products from commercial sources:	
i) Size and description of crushed product	ton (t)
ii) Etc. for each product size and type.	
b) Natural gravels and sands from commercial sources:	
i) Size and description of natural product	cubic metre (m^3)
ii) etc. for each type of natural material.	
c) etc. for each different imported material.	

The unit of measurement for crushed-stone products purchased from commercial sources shall be the ton of material brought onto the site and incorporated into the recycled material. Measurement shall be based on weighbridge tickets. The unit of measurement for natural material, purchased from commercial sources or obtained from borrow pits, shall be the cubic metre, measured as 70 % of the struck volume of the haul vehicle.

The tendered rate shall include full compensation for procuring, furnishing and spreading the imported material on the existing road as a level-correcting layer, or as a layer of uniform thickness, for hauling the material from the point of supply to its final position on the road, for watering and light rolling where required, and for any wastage.

Item	Unit
A3.05 Active filler:	
a) Ordinary Portland cement	ton (t)
b) Etc. for each type of chemical stabilising agent specified.	

The unit of measurement shall be the ton of active filler actually consumed in the recycling process. Measurement shall be based on weighbridge tickets where the supply is bulk, or on agreed counts where the supply is in pockets or bags.

The tendered rate shall include full compensation for procuring and providing the active filler, for its addition to the recycling process, including all transport, handling, storage under cover where required, rehandling and spreading, or fluidising into a slurry and pumping into the process, for all wastage and safety measures necessary during handling, and for the disposal of any packaging.

Item	Unit
A3.06 Bitumen stabilising agents:	
a) Bitumen emulsion:	
i) 60% residual bitumen, slow-set cationic	ton (t)
ii) Etc. for each different type of bitumen emulsion.	
b) Foamed bitumen, produced from:	
i) 80/100 penetration-grade bitumen	ton (t)
ii) etc. for each different type of bitumen.	

The unit of measurement shall be the ton of bitumen stabilising agent actually consumed in the recycling process. Measurement shall be based on physical dips of bulk tankers, undertaken before and after the bituminous stabilising agent is applied, supported by weighbridge tickets issued for each tanker-load at the point of supply.

The tendered rate shall include full compensation for procuring and providing the bitumen stabilising agent, for its addition to the recycling process, for any chemicals or other additives introduced, for the water added to achieve foaming, where required, for all transport, heating, handling, storage, and applying by pumping into the process, for all wastage and for taking all safety measures necessary during handling.

A3.9.2 Example of a typical schedule of quantities

The following schedule is an example of a schedule of quantities for a recycling project that includes the items listed above. The quantities relate to a 30 km long road, 7.3 m wide that is recycled to a depth of 175 mm (existing pavement includes an asphalt layer 80 mm thick) stabilised with 1% cement and 4% bitumen emulsion.

Item	Description	Unit	Quantity	Rate	Amount
A3.01	Prepare the existing road surface prior to recycling	m^2	220,000		
A3.02	Recycling in all in-situ pavement materials for the construction of new pavement layers: a) 175 mm thick layer: i) Road width >7.0 m but less than 7.5 m	m^3	40,000		
A3.03	Extra over Item A5.02 for layers of asphalt material within the recycled portion of the existing pavement, where the average thickness of asphalt is: a) More than 75 mm, but less than or equal to 100 mm	m^3	40,000		
A3.04	Chemical stabilising agents: a) Ordinary Portland cement	ton	1,000		
A3.05	Bituminous stabilising agents: a) Bitumen emulsion i) 60% residual bitumen, cationic slow set	ton	4,000		

Appendix 4 – The principles of economic analysis

The following is an extract from “Guidelines for Conducting the Economic Evaluation of Urban Transportation Projects: WJ Pienaar, University of Stellenbosch”.

A4.1 Introduction

Economic evaluation is the conceptual framework for the assessment of all gains (benefits) and losses (costs) of investment projects regardless of to whom they accrue within a country. A benefit is regarded as any gain in utility emanating from the operation and use of a facility and a cost is any loss of utility associated with the implementation of a project, where utility is measured in terms of opportunity costs. (The term economic evaluation does not include financial and social evaluation).

The primary purpose of the economic evaluation of urban transport projects based on economic efficiency and the implementation of subsequent recommendations is to minimise total transport cost, provided transport needs are effectively met. The total transport cost of a project comprises recurrent and non-recurrent costs. Non-recurrent costs comprise the initial cost (planning cost plus the opportunity cost of establishing a facility). Recurrent costs are incurred continuously throughout the service life of a facility and consist of the facility user costs and facility maintenance costs. An increase in non-recurrent costs generally gives rise to a decrease in recurrent costs, and vice versa. Minimisation of transport cost can therefore be achieved by determining the optimum trade-off between the cost categories.

There are three evaluation criteria on which the viability of a project may be based:

- Absolute advantage – which may be determined by the net present value technique;
- Relative advantage – which is usually determined through either the benefit/cost ratio technique or the internal rate of return technique; and
- Minimum total cost – which may be determined by the present worth of costs technique.

A4.2 Comparability of cost on a time basis

An economic evaluation of alternative transport projects requires individual cost evaluations on a common time basis, since money has a continuous time value. Thus, an amount X would be more valuable now than, say, in a year's time. This greater relevance of present power of disposal over funds, compared with eventual power of disposal over the same amount, is referred to as time preference propensity. Although the rate of inflation has an influence on the intensity of time preference propensity, the prevalence of inflation itself is not the cause that money has a time value. Even in non-inflationary periods there is still a time preference attached to money that is related to a community's average income from savings and investment. Thus, an amount kept in the purse where it cannot yield a return, is being denied the opportunity to grow in an alternative way. The average time preference of an amount may therefore be equated with its opportunity or alternative cost, as reflected in the average return on capital over a period.

Because transport facilities, such as roads, have a service life of several years, even decades, the evaluation period generally extends over twenty or more years. Facility maintenance cost and facility user cost incurred over a period therefore manifestly become less relevant as increasingly distant values are incorporated in the evaluation process. The method of determining the present worth of future funds is known as discounting. The rate used to compute the present worth of a cost in year n is referred to as the discount rate, and represents the continuous time value of money. An economic assessment is possible only after all future values have been expressed in equivalent terms, in other words after they have been reduced to their worth at a common point in time by means of a representative discount rate.

The basic formula for discounting a single future amount to its present worth becomes more comprehensible if discounting is regarded as the reverse of compound interest computation or, in other words, the conversion of present worth into future worth by making use of a specific rate of interest.

If, say, the time value of an amount = i percent per annum, its present worth (PW) in a year's time will increase to $PW(1 + i/1\ 00)$.
After two years its worth would equal: $PW(1 + i/100)(1 + i/100) = PW(1 + i/100)^2$.
After three years the PW would have increased to an amount equalling $PW(1 + i/100\}^3$, and so on, until after n years it would equal $PW(1 + i/100)^n$.

Since discounting is the reciprocal of interest computation, the calculation applied to $PW(1 + I/100)^n$ in order to discount it for a number of years at i percent would have to be inverted. In other words, one would have to multiply it by $1/(1 + i/100)^n$ to obtain its present worth.
The term $(1 + i/100)^n$ is known as the interest function, while its opposite, namely $1/(1 + i/100)^n$, is known as the discount function or the present worth factor.

The following formula is used to compute the PW of a. future amount (FA) at the end of year n at a discount rate of i percent per annum:

$$PW = FA/(1 + i)^n \qquad \text{(equation A4.1)}$$

where:
PW = present worth (value in year nil)
FA = future amount at the end of year n
i = annual discount rate as a fraction of 100
n = discount period in years

A4.3 Economic evaluation techniques

Proposed projects to be evaluated can be divided into two groups:

- Mutually exclusive proposals. Exclusive proposals are alternative methods of fulfilling the same function. The choice of anyone of the proposals will therefore exclude all the others. The cost-benefit analysis of mutually exclusive proposals involves the selection of the most efficient, i.e. most cost-effective alternative.

- Independent projects. These fulfil different functions and are therefore not alternatives for one another. Examples of independent projects are: a proposed modal transfer facility in suburb X, a proposed street-widening in suburb Y, and a proposed traffic interchange in suburb Z. More than one independent project can be selected for implementation. In fact, it is possible that all independent projects may be selected if they are all economically justified and sufficient funds are available. The economic evaluation of independent projects involves the ranking of the economically justified projects in terms of their economic merit.

Various techniques, all based on the principle of discounted cash flows, can be used for cost-benefit analysis. Four of the more commonly used techniques are explained below, namely:

- Present worth of cost (PWoC) technique;
- Benefit/cost ratio (B/C) technique;
- Internal rate of return (IRR) technique; and
- Net present value (NPV) technique.

In terms of their underlying philosophy, these techniques can be classified into two groups. For the first group, only the cost of each alternative is calculated, the argument being that the alternative with the least cost would be superior. The PWoC technique falls in this group.

In the second group of techniques, both benefits and costs of alternatives are calculated. Benefits are defined as savings in recurring costs relative to the null alternative (i.e. the existing situation or present facility whose improvement or replacement is being investigated). The underlying philosophy of techniques falling into this group, is that an alternative will be economically variable if benefits exceed costs. The cost of a project can be defined as the opportunity cost of economic resources sacrificed in implementing (providing) the project. The method of identifying the best alternative depends on the specific technique. Three techniques fall into this group: the NPV, the B/C ratio, and the IRR technique.

A4.3.1 Present worth of cost (PWoC) technique

This technique selects the lowest cost alternative among mutually exclusive projects. All economic costs (i.e. the opportunity costs) associated with the provision, maintenance and use of each possible alternative are discounted to their present worth. Given the objective of economic efficiency, the alternative that yields the lowest PWoC is regarded as the most cost-effective (beneficial) proposal. This method can be expressed as follows:

$$PWoC = C_a + PW(M + U) \quad \text{(equation A4.2)}$$

where:

PWoC	= present worth of cost
C_a	= all initial costs incurred in constructing a facility, minus the discounted residual value at the end of the analysis period
PW(M + U)	= present worth of all maintenance and road user costs during the period under review.

Note that in the case of the null alternative (i.e. the existing facility whose possible replacement or upgrading is being investigated, and against which the other mutually exclusive alternatives are measured), PWOC = PW(M + U).

A4.3.2 Benefit/Cost (B/C) ratio technique

This technique selects the most advantageous alternative by determining the ratio between project benefits (i.e. annual savings relative to the null alternative) and discounted initial project costs. The anticipated benefit during the analysis period is determined by subtracting the present worth of an alternative's forecasted road user cost, plus road maintenance costs, from the present worth of the existing facility's forecasted user cost, plus maintenance cost.

The ratio between the sum of the discounted benefits and the sum of the initial project costs is obtained by dividing the former by the latter. All proposals with a ratio value greater than one are viable, while the one with the highest ratio value is economically the most advantageous. However, when mutually exclusive alternatives are compared, incremental analysis must be used to identify the most economical alternative. The method can be expressed as follows:

$$B/C = [PW\,(M_O + U_O) - (PW\,(M_a + U_a)]/C_a \qquad \text{(equation A4.3)}$$

where:

B/C = benefit/cost ratio

o = existing facility

a = alternative under consideration

A4.3.3 Internal rate of return (IRR) technique

This technique calculates the anticipated internal rate of return of each alternative relative to the null alternative. The distinctive feature of this technique is that its application does not entail a singular discounting procedure with one prescribed rate only.

Annual savings ("returns") for the period under review are discounted to the beginning of the period. The sum of these discounted amounts are compared with the discounted initial cost. Different rates of return are selected iteratively and applied until at a certain rate the sum of the annual discounted returns equals initial costs. This is then referred to as the (anticipated) internal rate of return. The alternative with the largest internal rate of return can be regarded as the most advantageous, although the actual criterion is to compare the rate thus obtained with the prevailing real discount rate. If it exceeds the prevailing discount rate, the alternative is economically viable. However, when mutually exclusive alternatives are compared, incremental analysis must be used to identify the most economical alternative.

The method can be expressed as follows:

$$\mathbf{IRR = r \text{ when } \{PW(M_o + U_o) - PW(M_a + U_a)\} = C_a} \qquad \text{(equation A4.4)}$$

where:
r = rate at which the left-hand and right-hand sides of the equation are equal.

A4.3.4 Net present value (NPV) technique

This technique selects an alternative among the mutually exclusive projects that has the greatest net present value. The discounted initial cost of an alternative is subtracted from the sum of the discounted annual savings that the alternative will achieve in comparison with the existing facility. All alternatives with benefits reflecting a positive net present value are viable, while the alternative with the highest present value is the most advantageous.

The technique can be expressed thus:

$$NPV = PW(M_o + U_o) - PW(M_a + U_a) - C_a \quad \text{(equation A4.5)}$$

A4.4 Period of analysis and the terminal and residual value of transport facilities

The opportunity cost principle excludes the possibility of a transport facility possessing an additional value over and above the opportunity cost of its land reserve during its service life, the reason being that investment in transport facilities is regarded as sunken (i.e. it is taken that the development itself has no possible alternative application). For this reason it is desirable that the analysis (evaluation) period should stretch over the entire design life or planned lifespan of a facility. However, there are sound practical reasons why the analysis period should sometimes be shorter than the planned service life. Due to risk/future uncertainties, all projections and forecasts for periods exceeding twenty years are extremely speculative owing to, for example, the difficulty of predicting future traffic volumes, modal split, changes in technology, land use, demographic features, etc.

The periods for which forecasts and projections are made can be extended beyond 20 years on condition that they remain reliable. However, it is recommended that analysis periods should not exceed 30 years (usually a 20 year analysis period is used), even though the logical solutions to certain transport problems may be durable projects with service lives substantially longer than 30 years. To be fair and just (realistic) with regards to investment in durable projects, a departure is made from the pure but limiting opportunity cost rule by introducing a second best notional valuation rule in the form of a “residual value”. This value purports to represent an artificial opportunity cost of a project during its service life and is widely, if not universally, accepted as a legitimate convention and justified departure from the opportunity cost rule. Had this not been the case, very few durable transport projects would ever have been implemented on account of the outcome of an economic evaluation.

Should it be found that a project has an expected terminal value (i.e. the r salvage value of the facility at the end of its design life) or a residual value (i.e. the salvage value of the facility during its design life), such values should be discounted and deducted from discounted initial cost. The reason why this is done is that an IRR and a B/C are returns or yields relative to the investment cost. For example, if a project has a design or service life of 30 years and, after termination of this period, the land is again utilised for the same economic activity as had been the case before the 30 year period, the cost of the initial investment is equal to the opportunity cost of facility development (i.e. direct planning, design and construction) plus the benefit which is being forfeited by not using its land reserve in its alternative application during the service life. In other words, the investment cost in terms of which return or yield is calculated is the opportunity cost attached to developing the facility as well as the opportunity cost of engaging the land for transportation purposes up to that point when it is again released for alternative use.

Wirtgen GmbH
Reinhard-Wirtgen-Strasse 2 · 53578 Windhagen · Germany
Phone: +49 (0) 26 45/131-0 · Fax: +49 (0) 26 45/131-242
Internet: www.wirtgen.com · E-Mail: info@wirtgen.com