SOUTH AFRICAN

PAVEMENT ENGINEERING MANUAL

Chapter 10

Pavement Design

THE SOUTH AFRICAN NATIONAL ROADS AGENCY SOC LTD

Reg. No.1998/009584/06

AN INITIATIVE OF THE SOUTH AFRICAN NATIONAL ROADS AGENCY LTD

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Revision 1.0

South African Pavement Engineering Manual
Chapter 10: Pavement Design

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BACKGROUND

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SCOPE

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

SAPEM consists of 14 chapters covering all aspects of pavement engineering. A brief description of each chapter is given below to provide the context for this chapter, Chapter 10.

Chapter 1: Introduction discusses the application of this SAPEM manual, and the institutional responsibilities, statutory requirements, and, planning and time scheduling for pavement engineering projects. A glossary of terms and abbreviations used in all the SAPEM chapters is included in Appendix A.

Chapter 2: Pavement Composition and Behaviour includes discussion on the history and basic principles of roads. Typical pavement structures, material characteristics and pavement types are given. The development of pavement distress and the functional performance of pavements are explained. As an introduction, and background for reference with other chapters, the basic principles of mechanics of materials and material science are outlined.

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

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

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

Chapter 6: Road Prism and Pavement Investigation discusses all aspects of the road prism and pavement investigations, including legal and environmental requirements, materials testing, and the reporting of the investigations. Chapters 6 and 7 are complementary.

Chapter 7: Geotechnical Investigations and Design Considerations covers the investigations into potential problem subgrades, fills, cuts, structures and tunnels. Guidelines for the reporting of the investigations are provided.

Chapter 8: Material Sources provides information for sourcing materials from project quarries and borrow pits, commercial materials sources and alternative sources.

Chapter 9: Materials Utilisation and Design discusses materials in the roadbed, earthworks (including cuts and fills) and all the pavement layers, including soils and gravels, crushed stones, cementitious materials, primes, stone precoating fluids and tack coats, bituminous binders, bitumen stabilised materials, asphalt, spray seals and micro surfacings, concrete, proprietary and certified products and block paving. The mix designs of all materials are discussed.

Chapter 10: Pavement Design covers many aspects of pavement design, including design considerations, estimating design traffic, pavement investigation and design processes, structural capacity estimation, and software available for pavement design. The principles of pavement design for flexible, rigid and block pavements are described. The design traffic estimation section describes various methods of obtaining traffic data, and the procedures for estimating the cumulative design traffic for the structural design period. The pavement investigation and design process sections cover both new and rehabilitation design, and look at material availability, constructability, performance and maintainability. Economic assessments necessary for evaluating alternative pavement designs are also briefly described. The structural capacity estimation methods for flexible pavements are described, including the South African Mechanistic-empirical Design Method and the Pavement Number method, as well as other methods used in South Africa. For concrete and block pavements, the Mechanistic-empirical method is described, with other older methods. The suitability of all the structural capacity estimation methods for particular applications is provided.

Chapter 11: Documentation and Tendering covers the different forms of contracts typical for road pavement projects; the design, contract and tender documentation; and, the tender process.

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

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

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

FEEDBACK

SAPEM is a "living document". The first edition was made available in electronic format in January 2013. It is envisaged that SAPEM will be updated after one year. Feedback from all interested parties in industry is appreciated, as this will keep SAPEM appropriate.

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

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1. INTRODUCTION

Pavement design is an engineering discipline, somewhere in between science and art. In contrast to scientists, engineers do not require perfect models, but models that are "good enough" to produce fit for purpose designs. Where the scientific knowledge is insufficient, it is supplemented with the experience and sometimes intuition (art) of the designer. However, it remains the professional responsibility, and liability, of the design engineer to ensure that the design is based on the accepted design practice of the time.

The loading on pavements consists of millions of relatively small magnitude loads causing the gradual or incremental deterioration of the pavement, until the level of service becomes unacceptable. The stress imposed by the external load is normally well below the strength of the material, resulting in gradual deterioration, rather than a catastrophic failure caused by one load. In some cases, climatic effects are more detrimental than the effects of loading, especially in low volume roads.

The basic objective of pavement design is to combine materials of sufficient strength in a layered system to provide the desired functional and structural service levels over the design period, subject to the applicable traffic demand and environment in which the pavement operates. The economic viability of a design is determined by:

- Functional and structural **service levels**
- **Deterioration rates** of these service levels
- **Costs** associated with the provision and maintenance of these service levels
- **Savings by the road users** resulting from improved service levels

Although the final design decision is dictated by the most economically viable design, the process of ensuring a pavement with adequate strength is provided is critical.

The ability of a pavement to provide acceptable functional and structural levels of service defines the pavement "supply". The "demand" on a pavement depends on the environments in which the pavement operates, which are:

- **Traffic environment**, which is the primary "demand" imposed on the pavement by the vehicle traffic operating on the road. The following factors are important:
  - Axle load magnitude
  - Contact stress
  - Traffic volume
- **Natural environment,** may be subdivided into the geological environment and climatic environment, which together determine:
  - Available natural material sources
  - In situ subgrade conditions
  - Moisture regime in the pavement
  - Temperature and parameters contributing to aging, specifically of asphalt layers

  The natural environment plays an increasingly important role in low volume roads, as illustrated in Figure 1.
- **Population environment**, which is of particular importance in urban areas

Flexible & Rigid Pavements

Flexible pavements or bituminous pavements typically have asphalt or seal surfacings. These structures are traditionally characterised by higher deflections or bending.

Rigid pavements or concrete pavements act in a rigid manner relative to flexible pavements, producing much lower deflections and bending.

Gravel Roads

The design of gravel roads is not discussed in this manual. However, most of the design principles discussed apply to gravel roads. A good reference for the design of gravel roads is: **TRH20: Unsealed Roads: Design, Construction and Maintenance.** 2009.

Specifications for gravel roads are included in the COLTO Standard Specifications (COLTO, 1998).

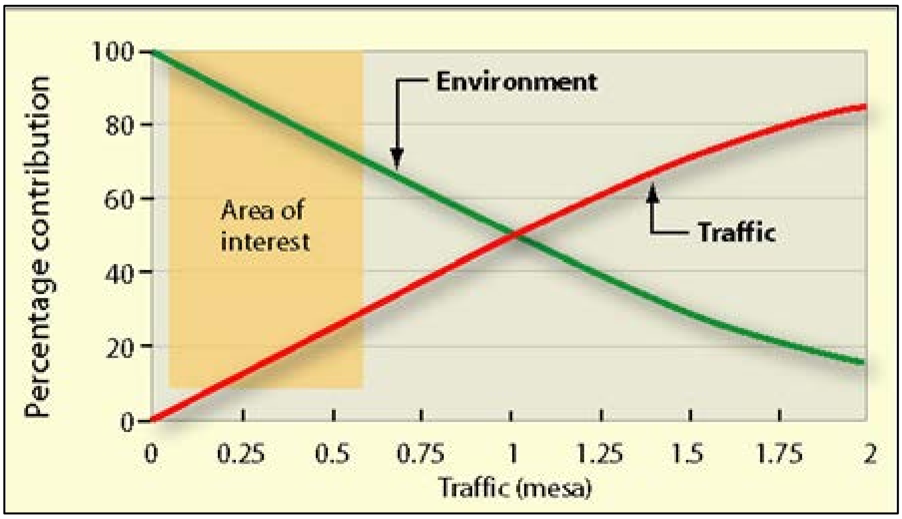


Figure 1. Effect of Traffic and Environment on Performance of Roads with Different Traffic Levels

The functional service level largely determines how the road user experiences the service provided by the road, and includes aspects of safety and ride comfort. The geometric design also contributes to the road-user's perception of the level of service, but falls outside the scope of this manual.

The structural service level reflects the condition of the pavement system and is determined by the level of structural deterioration. The structural condition of a pavement is generally affected by cracking, i.e., fatigue damage, and permanent deformation (or rutting) of flexible pavements, rutting of concrete block pavements and defects such as joint spalling and faulting, slab cracking and punch-outs of rigid pavements. The different distress types in pavements are discussed in Chapter 14, Section 3. The most common source of problems in a pavement is the ingress of moisture.

> **Functional Capacity**
>
> The functional capacity or service life is defined as the period from an initial sound functional condition to a predefined terminal (unacceptable) functional condition. This is determined by functional deterioration in terms of skid resistance, loose material, potholes and riding quality. The functional condition may often be improved through maintenance alone, but the improvement of a poor riding quality may require light rehabilitation in the form of an overlay.

A distinction is drawn between structural distress, terminal structural condition and structural failure.

- **Structural distress** occurs from the day the pavement is opened to traffic. Incremental permanent deformation and fatigue occur throughout the service life of the facility, but initially the level of distress is insignificant.
- A **terminal structural condition** is reached when the level of distress reaches a predefined, unacceptable level. Examples of terminal conditions are 20 mm of rutting for flexible pavements and 5 mm of faulting for rigid pavements. Although a terminal (unacceptable) level of distress is reached, the pavement has not necessarily failed in the strict sense of the word. For example, the rut may be rectified with a levelling course and the pavement may continue to provide years of excellent service.
- **Structural failure** occurs if the pavement layer or layers lose strength. For example, if rutting results from unstable, rapid shear failure, the layers do not have sufficient strength and the pavement requires structural strengthening.

> **Structural Capacity**
>
> The structural capacity or service life is defined as the period from an initial sound structural condition to a predefined terminal (unacceptable) structural condition. During this period only routine maintenance is performed, and at the end of this period heavy rehabilitation or reconstruction is normally required.

The functional capacity or service life is defined as the period from an initial sound functional condition with subsequent routine maintenance only, to a predefined terminal (unacceptable) functional condition. For flexible pavements, this is determined by functional deterioration in terms of skid resistance, loose material, potholes and riding quality. The functional condition may often be improved through maintenance alone, but the improvement of a poor riding quality may require light rehabilitation in the form of an overlay. In many cases, functional distresses are manifestations of structural deterioration.

Structural capacity or service life is defined as the period from an initial sound structural condition to a predefined terminal condition. The structural condition is determined by structural deterioration in terms of deformation, cracks, faulting and a loss of load bearing capacity. Structural failure can often only be corrected through heavy rehabilitation, requiring the structural strengthening or rebuilding of the "failed" pavement structure.

Routine Road Maintenance

Pavement design methods assume routine maintenance is done on all roads. A good reference for routine maintenance is:

- SANRAL. 2009. **Routine Road Maintenance Manual**. Second Edition.

The structural and functional capacities are measured in terms of the number of load repetitions to reach the terminal condition, or the time period required to reach the terminal condition. These structural and function capacities are affected by the timing of maintenance and rehabilitation. This is discussed further in Section 3.3.

There are a number of factors that impact on the functional and structural performance, capacity and service life of a pavement. These are the construction quality, maintenance schedule and pavement design. Together, these three aspects determine the performance potential of the pavement. The actual performance is determined by the environment in which the pavement operates. A pavement with a given design, construction quality and maintenance schedule performs differently in different environments.

The balance between the potential capacity of the pavement and the demand on the pavement eventually determines the performance and service life of the pavement. If there is an oversupply of capacity for the given environment, the pavement will perform exceptionally well and has a service life that exceeds the originally intended design life. Although this is a safe design approach, it has high initial costs and is probably not economically warranted in the long-term. Similarly, if the capacity is insufficient for the given environment, the pavement performs poorly, is unlikely to reach the intended design life, and requires regular maintenance at high costs, making such an approach unsustainable.

Structural Pavement Design

Structural pavement design matches the structural capacity of a pavement with the traffic demand for the given environment. It is inherently assumed that the design assumptions are met during construction and a proper maintenance schedule is followed. See Section 3.2 and 3.3 for further discussion.

Structural pavement design matches the structural capacity of a pavement with the traffic demand for the given environment. It is inherently assumed that the design assumptions are met during construction and a proper maintenance schedule is followed. Structural design therefore requires two components:

- A method to calculate and express the **traffic demand** in numerical terms.
- A method to estimate the **structural capacity** of a potential pavement design.

These two components of the structural pavement design process are presented in this chapter. The intention of this chapter is to provide an overview of sound engineering practice and procedures, as well as the techniques used during pavement design. Reference should be made to the guideline documents for additional details.

Table 1 provides a summary of the main elements of new and rehabilitation design. The sections in this chapter where the topics are covered are also given.

Guidelines for Pavement Design

A number of national and industry guideline documents are useful for the design of new and rehabilitated pavements. These are:

- **Flexible pavements**
  - TRH4 (1996) for new design
  - TRH12 (1997) for rehabilitation design
  - TRH14 (1985) for material classification
- **Rigid pavements**
  - M10 Manual (1995)
  - Extensive documentation in the help file of cncPave. The M10 Manual was superseded by cncPave.
- **Block pavements**
  - UTG2 (1987)
  - Concrete Block Paving Book 2 – Design Aspects (CMA, 2007)
- **Traffic estimation**
  - TRH16 (1991) for flexible, concrete block and rigid pavements

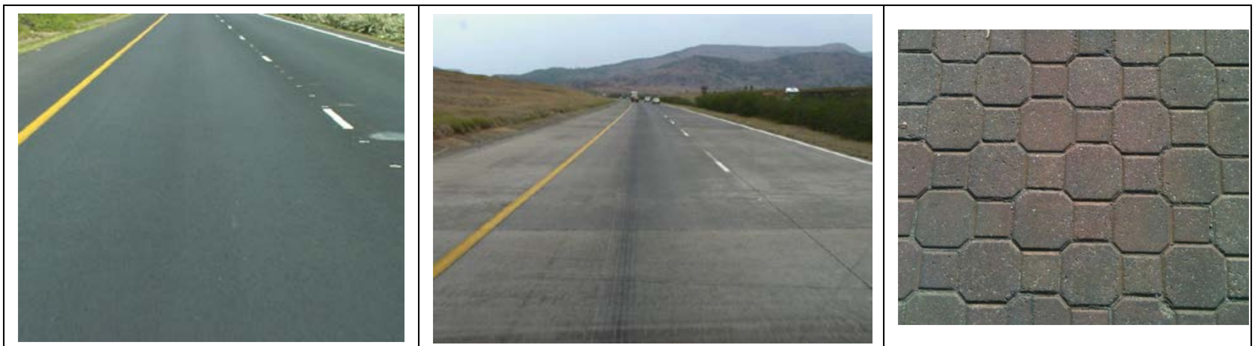
These documents set out sound engineering design principles and procedures to follow during the design phase, supplemented by the other guidelines mentioned in this chapter and listed in the references.

Table 1. Elements of New and Rehabilitation Pavement Design

<table>
<tr><th colspan="2">Element</th><th>New Design</th><th>Rehabilitation Design</th></tr>
<tr><td colspan="2" rowspan="2">Design Principles (Section 2)</td><td colspan="2">• Use material of sufficient strength and stiffness in the structural layers to withstand the loading
• Provide sufficient cover to the subgrade to ensure it is outside the area of high stress
• Provide a pavement foundation of adequate support for the structural layers
• Create a gradual transition in strength from the structural layers to the subgrade for flexible pavements
• Provide adequate support in the subbase for rigid pavements
• Maximise material strength, with proper compaction and by keeping the pavement dry:
– Proper drainage design and maintenance
– Maintain surfacing integrity</td></tr>
<tr><td></td><td>• Eliminate obvious problems, e.g., repair badly cracked local areas before overlay
• Select appropriate remedial measures for areas not requiring structural strengthening
• Maximise the use of the existing pavement structure in providing the required structural capacity</td></tr>
<tr><td colspan="2">Design Considerations (Section 3)</td><td colspan="2">• Road category
• Analysis and structural design periods
• Life-cycle strategy
• Pavement balance (for flexible pavements)
• Pavement behaviour
• Materials
• Environmental considerations
• Practical considerations
• Pavement type selection</td></tr>
<tr><td colspan="2">Design Techniques (Sections 4 & 5)</td><td colspan="2">• Design traffic calculations
• Structural capacity estimation</td></tr>
<tr><td rowspan="3">Pavement Design Process</td><td>Design Investigation (Section 5)</td><td>Data collection:
• Available data
• Additional testing
• Identify uniform subgrade sections
• Assess material availability, quality and cost
• Traffic investigation and design traffic estimate</td><td>Structured approach for investigation:
• Initial assessment:
– Condition assessment
– Homogeneous/uniform sections
– Past and future traffic
– Initial structural capacity assessment
• Detailed assessment:
– Identify distress mechanisms and remedial measures
– Characterise the existing structure for utilisation in the design
– Generate uniform section reports</td></tr>
<tr><td>Structural pavement design (Sections 6, 7, 8 & 9)</td><td>• Match structural capacity of the pavement with future traffic demand
• Use appropriate structural design techniques
– CBR, AASHTO Structural Number, DCP, Catalogues, SAMDM, Pavement Number, cncPAVE, Lockpave
• Adhere to design principles</td><td>• Match structural capacity of the pavement with future traffic demand
• Use appropriate structural design techniques
– Deflection based methods, AASHTO Structural Number, DCP, SAMDM, Pavement Number, cncPAVE, Lockpave
• Adhere to design principles</td></tr>
<tr><td>Economic analysis (Section 5)</td><td colspan="2">• Link to the life-cycle strategy
• Consider construction, maintenance and rehabilitation cost
• Select the most economically viable design</td></tr>
</table>

2. DESIGN PRINCIPLES

Typical examples of the three types of pavements; flexible, rigid and block pavements, are shown in Figure 2. This section discusses the design principles associated with these three pavement types.



Flexible Pavement | Rigid Pavement | Block Pavement

Figure 2. Typical Pavements

2.1 Flexible Pavements

The basic principles of structural design of a flexible pavement are explained using Figure 3, which shows a multi-layered pavement system loaded by a dual-wheel, half-axle load. The stresses imposed by the tyres at the tyre-pavement interface, also called the tyre-pavement contact stress, are applied to a relatively small contact area. These stresses are dissipated or spread over an area that increases with increasing depth in the pavement structure. The stress concentration and shear stress, therefore, reduce with increasing depth. This is indicated by the red/yellow shaded area. The materials in the upper region of the pavement structure therefore need high shear strength to resist the imposed shear stress conditions. Deeper down in the pavement structure, less shear strength is required.

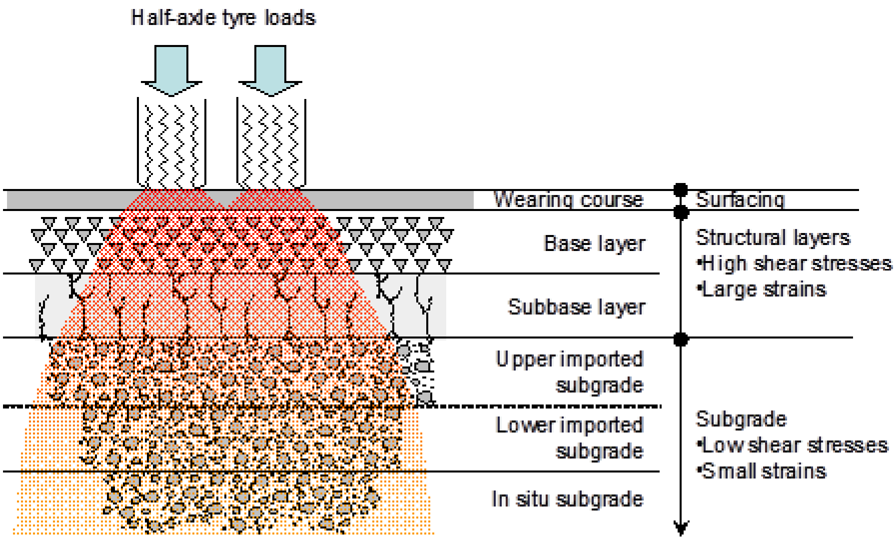


Figure 3. Stress Distribution in a Typical South African Flexible Pavement

Given this stress dissipation and material strength requirement, the five principles of the structural design process for flexible pavements are:

- Select materials of sufficient **strength and stiffness** for the pavement structural layers, to resist the high imposed shear stresses and to dissipate the high imposed stresses.
- Select sufficient **structural layer thickness** to ensure that the subgrade material is out of the high stress region.

- Provide a subgrade that adheres to the minimum **subgrade strength requirement,** to act as a proper support for the structural layers. If the in situ subgrade material is of an insufficient quality, material should be imported.
- The transition from material with high shear strength and stiffness at the top of the pavement structure to relatively lower shear strength and stiffness in the subgrade should be a gradual transition, resulting in a **balanced pavement structure** (see Section 3.4.1).
- The strength potential of all layers should be maximised through proper **compaction** to achieve a high density, and by keeping unbound granular layers as dry as possible. This is achieved with proper surface and subsurface drainage **design** and **maintenance** as well as maintaining the integrity of the surfacing layer. Drainage is not discussed in SAPEM, and is adequately covered in SANRAL's Drainage Manual (SANRAL, 2006).

Although the subgrade cannot be improved at reasonable cost for rehabilitation projects, the principle that the subgrade (and other weak layers in the pavement) should be outside the region of high shear stress applies equally to new and rehabilitation design. Additional design principles apply to rehabilitation design to eliminate obvious problems. For example:

- Repair **badly cracked areas** before overlay, to prevent the existing cracks reflecting through the overlay.
- Select appropriate **remedial measures** for areas not requiring structural strengthening.
- Maximise the use of the **existing pavement structure** to provide the required structural capacity.

2.2 Rigid Pavements

The principles for concrete or rigid pavement design are similar to those for flexible pavements, except that the transition from material with high shear strength and stiffness at the top to lower shear strength and stiffness material in the subgrade is rapid, not gradual. The primary load supporting element of a concrete pavement is the rigid layer or concrete slab. The shear strength and stiffness of concrete is high in relation to asphalt or crushed stone road bases, and the imposed stresses are dissipated quickly in the rigid layer. A thin layer of concrete thus protects the subgrade in a similar way as thicker layers and combinations of asphalt, crushed stone and gravel materials. This is illustrated in Figure 4.

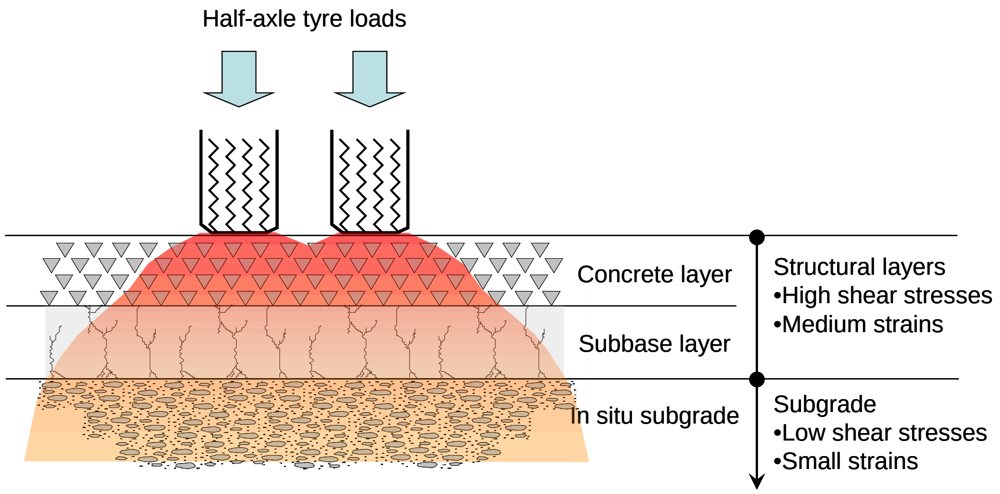


Figure 4. Typical Stress Distribution in a Concrete Pavement

The essential elements of concrete pavement design are to design the slab length, slab thickness, and subbase support type. The slab length is important to mitigate shrinkage cracking. In Plain Jointed Concrete Pavements (PJCP), shrinkage cracking is controlled by providing joints at regular and relatively short intervals.

Failures in concrete pavement generally occur at joints and cracks. Design, therefore, focusses on joints and cracks, with the aim of ensuring proper load transfer. Dowels are often installed at joints to improve load transfer across the joints and the concrete pavement is then referred to as a dowel jointed plain concrete pavement. The width of shrinkage cracking may also be controlled by installing

Subbase in Concrete Pavements

Moisture enters concrete pavements through joints and cracks. This causes erosion of the subbase, causing more vertical movement of the slab. The design of the subbase is essential to mitigate erodibility. This is generally achieved by selecting good material and by stabilising the subbase.

reinforced steel in the concrete pavement. This type of concrete pavement is generally referred to as Continuously Reinforced Concrete Pavements (CRCPs). Hybrids such as Jointed Reinforced Concrete Pavements (JRCP) also exist. The different types of concrete pavements are illustrated and discussed in Chapter 2, Section 6.1.2 or Chapter 9, Section 12.2.2.

In concrete pavements, moisture enters the pavement through joints and cracks. This causes erosion of the subbase, loss of support of the slab and slab failures. The design of the subbase is essential to mitigate erodibility. This is generally achieved by good material selection and by stabilising the subbase.

2.3 Block Pavements

Concrete block pavements fall somewhere between flexible and a rigid pavements. The layer of paving blocks provides a surfacing that is stiffer than those provided by bituminous materials (asphalt or seals), but not as stiff as concrete. The load spreading characteristics of a concrete block pavement are thus between that of flexible and rigid pavements.

Block pavements are considered as very cracked concrete pavements, with the cracks filled with sand to enhance load transfer between blocks. In essence, they are similar to thin concrete slabs with discontinuities. As with concrete pavements, therefore, the quality and strength of the blocks must be adequate to carry the load. The bedding sand should be permeable and not erodible, comprising of a, preferably, high quality crushed material.

3. DESIGN CONSIDERATIONS

There are many aspects to consider in the design process, before the actual pavement design can be performed. This section discusses some of these considerations.

3.1 Road Category

In South Africa, roads are categorized according to the importance and level of service required. Table 2 gives details of the four road categories used (TRH4, 1996; TRH12, 1997 and cncPAVE). The "approximate design reliability" is linked to the category. Photographs of typical roads in each category are given in Figure 5.

For new design, the design reliability is usually introduced when estimating the structural capacity of a potential design. This process is discussed in more detail in Theyse et al (1995 and 1996) for flexible pavements. For rehabilitation design, the road category not only determines the approximate reliability for the future structural capacity of the rehabilitated road, but also provides guidance on the percentile levels to use during the assessment of condition data for the existing road.

Table 2. Definition of the Four Road Categories

	Road Category			
	A	B	C	D
Description	Major inter-urban freeways and major rural roads	Inter-urban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Rural access roads
Importance	Very important	Important	Less important	Less important
Level of service	Very high	High	Moderate	Moderate
Typical Pavement Characteristics				
Approximate design reliability (%)	95	90	80	50[1]
Length of road exceeding terminal distress condition at end of structural design life	5	10	20	50
Total equivalent traffic loading (E80/lane)	3 – 100 million over 20 years	0.3 – 10 million Depending on design strategy	< 3 million Depending on design strategy	< 1 million Depending on design strategy
Typical pavement class[2]	ES10 – ES100	ES1 – ES10	< ES0.03 – ES3	ES0.003 – ES1
Daily traffic (evu)	> 4000	600 – 10 000	< 600	< 500
Riding quality:				
Constructed				
PSI	3.5 – 4.5	3.0 – 4.5	2.5 – 3.5	2.0 – 3.5
IRI	2.4 – 1.6	2.9 – 1.6	3.5 – 2.4	4.2 – 2.4
Terminal				
PSI	2.5	2.0	1.8	1.5
IRI	3.5	4.2	4.5	5.1
Rut level for flexible pavements (mm)				
Warning	10	10	10	10
Terminal	20	20	20	20
Area of shattered concrete for rigid pavements (%)[3]				
CRCP and UTCRCP				
Warning	0.2	0.3	0.4	0.5
Terminal	0.5	0.7	0.8	1.0
JCP and DJCP				
Warning	2	3	4	5
Terminal	5	6	8	10

Note
1. Although 50% reliability is stated for Category D, this essentially implies designing for an average situation. Great caution should, however, be taken when designing for an average situation with average values.
2. Traffic classes given in Section 4.1.5.
3. These criteria from cncPave.

	
Category A	**Category B**
	
Category C	**Category D**

Figure 5. Typical South African Roads for Road Categories

3.2 Analysis and Structural Design Periods

The analysis period incorporates one or more structural design periods and is often related to the geometric design life of the facility. If the geometric design is expected to be appropriate for a long period, because the traffic demand can be forecasted with a high degree of confidence, then the analysis period can be as long as 30 years. With an uncertain geometric design life, in a changing traffic situation, a shorter analysis period, such as 10 years, can be considered. In many instances, however, budget constraints dictate the analysis and structural design periods. The economic analysis of the life-cycle cost of the road is done for the analysis period.

The following guidelines apply to the selection of a structural design period for the different road categories:

- **Category A roads:** A structural design period of 25 years, ranging between 15 and 30 years, is recommended because:
  - The road alignment is fixed for a long period with a high certainty.
  - It is not acceptable to road users to have heavy rehabilitation on recently constructed important roads.
  - The cost of disrupting the high volumes of traffic offsets any advantage of a shorter structural design period.
- **Category B roads**. A structural design period of 20 years, ranging between 15 and 25 years, is recommended. Factors that may encourage a shorter design period are:
  - A changing traffic situation, which may cause the geometric design to become outdated.
  - A shortage of funds for the initial construction cost.
  - A lack of confidence in the design assumptions.

  If a shortened structural design period is selected, the design should be able to accommodate staged construction. See Section 3.3 for further discussion.
- **Category C roads.** Financial constraints may dictate the selection of a shortened structural design period of 10 years. If it is foreseen that structural rehabilitation will be difficult, a longer period of 20 years may be appropriate.

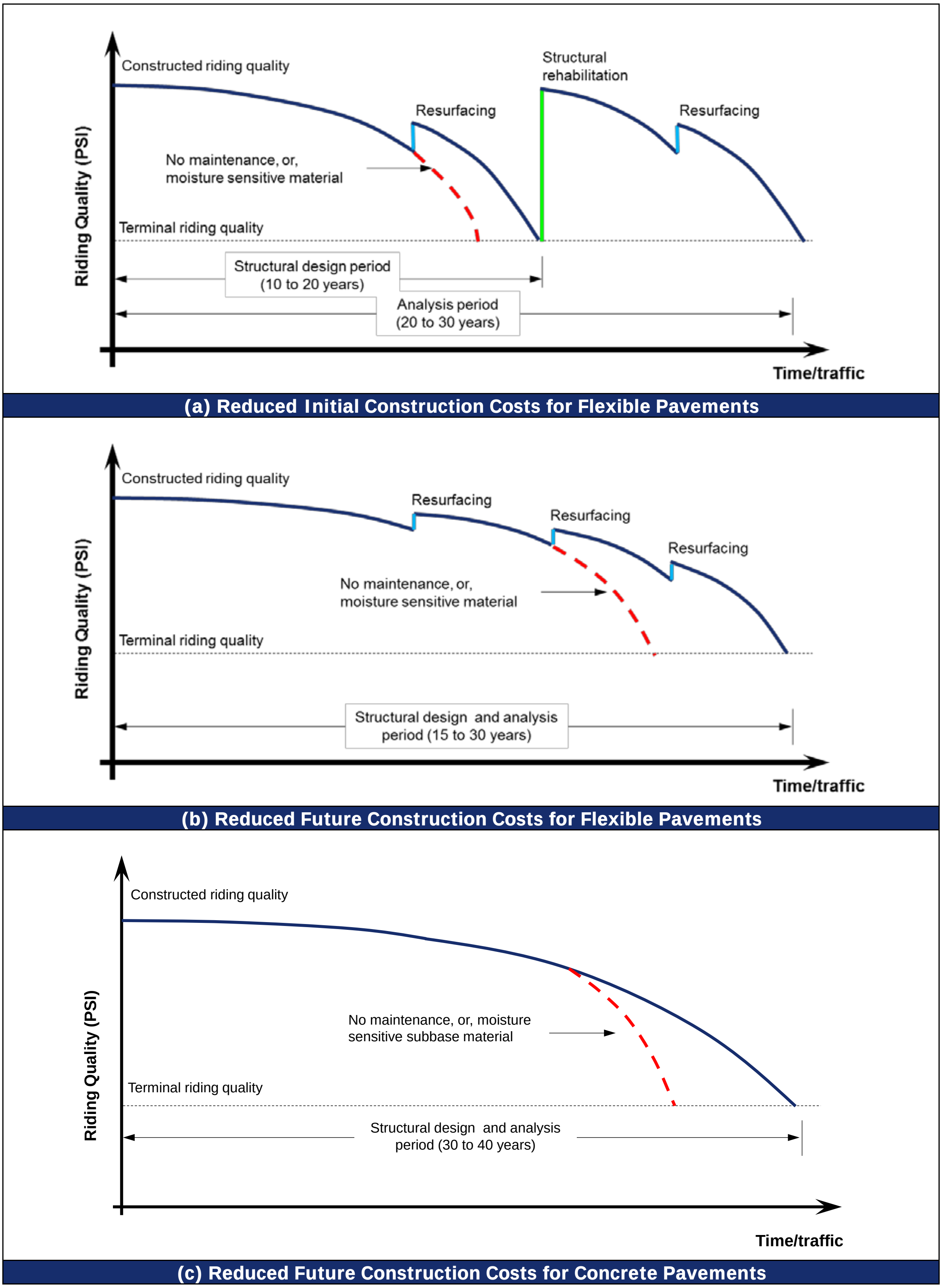
- **Category D roads.** Low volume and experimental roads are often also classified as Category D roads. The traffic growth on category D roads can be rapid and unpredictable. A short structural design period enables changes in the initial life-cycle strategy to adapt to changing circumstances, without any major financial implications. A design reliability of 50 percent implies that half the pavement is distressed at the end of its design period.

3.3 Life-Cycle Strategies

The purpose of pavement design is to identify the most economic design, with sufficient structural capacity, for the particular design situation. Various alternative designs are initially identified, considering the availability of materials, structural capacity demand of the actual traffic spectrum, and the service level of the facility. The ultimate selection of a particular design, however, depends on an economic analysis of the design options over the full life-cycle of the facility. The strategy for some of the design alternatives may call for more capital investment for the initial construction to ensure a longer structural design period, while others may have a bigger demand for maintenance funds.

The design strategy and economic analysis of the full life-cycle of the facility require an understanding of the behaviour of different pavement types, and the type and timing of maintenance and rehabilitation expected during the life cycle. An understanding of material and pavement behaviour is therefore critically important.

Figure 6 illustrates three possible design strategies for the design of a new facility. Alternative strategies are normally compared on the basis of the least Present Worth of Cost (PWOC). The successful realization of any design strategy depends on the constructability, maintainability and salvage value of the design. See Section 5.4 for more discussion on economic analyses.



(a) Reduced Initial Construction Costs for Flexible Pavements

(b) Reduced Future Construction Costs for Flexible Pavements

(c) Reduced Future Construction Costs for Concrete Pavements

Figure 6. Examples of Three Possible Design Strategies

3.4 Pavement Balance

3.4.1 Flexible Pavements

One of the design principles for flexible pavements is that the material quality gradually, and smoothly, increases from the in situ subgrade up to the structural layers and surfacing. Such a pavement structure is referred to as a well-balanced pavement. Pavement balance may be quantified from the analysis of Dynamic Cone Penetrometer (DCP) data for which pavement balance parameters have been defined, or by calculating the modular ratio of the stiffnesses of successive layers. Refer to Section 7.3 for DCP analyses and Section 7.2.2 for modular ratios.

> **Pavement Balance**
>
> One of the design principles for flexible pavements is that the material quality gradually, and smoothly, increases from the in situ subgrade up to the structural layers and surfacing. Such a pavement structure is referred to as a well-balanced pavement.

If the strength of a pavement is concentrated in one or two strong layers, those layers initially carry most of the loading, but deteriorate to achieve balance with the rest of the pavement structure. This process of the relative strength of the pavement layers being forced into balance is referred to as "traffic moulding". If the bulk of the strength of a pavement is concentrated in only the uppermost layers, the pavement is referred to as a shallow pavement. If the pavement strength is distributed throughout the depth of the pavement, the pavement is referred to as a deep pavement. Deep, well-balanced pavement designs are normally less sensitive to high loads than where all the strength is concentrated towards the top of the pavement, with poor support from below.

> **Deep and Shallow Pavements**
>
> - In **shallow** pavements, the strength of the pavement is concentrated in the uppermost layers.
> - In **deep** pavements, the strength is distributed throughout the pavement.
>
> Deep, well-balanced flexible pavement designs are normally less sensitive to high loads than designs where all the strength is concentrated towards the top of the pavement, with poor support from below.

Excellent performance has been obtained from deep, well-balanced granular pavement structures in South Africa using only thin asphalt surfacing layers, provided water is prevented from entering the base through the surfacing. Shallow pavements with a cemented or a hot mix asphalt base layer on a subbase or subgrade of much lower stiffness should be avoided. Such a pavement not only deteriorates rapidly under normal traffic, but is also more sensitive to overloading than a well-balanced, deep pavement structure.

> **Avoid Shallow Flexible Pavements**
>
> Shallow pavements with a cemented or a hot mix asphalt base layer on a subbase or subgrade of much lower stiffness should be avoided. Such a pavement not only deteriorates rapidly under normal traffic, but is also be more sensitive to overloading than a well-balanced, deep pavement structure.

The exception to well-balance pavements is inverted pavements. This is where a granular base is placed on a stronger, lightly cemented base. These pavements are commonly, and very successfully, used in South Africa. See Section 7.1.2 for further discussion.

Structural design methods do not generally yield a balanced pavement design automatically. Care must be taken to ensure that a design is balanced.

3.4.2 Concrete (Rigid) Pavements

The principle of pavement balance does not apply to concrete pavements. The strength of concrete is significantly higher than that of asphalt, granular and lightly cemented layers used in flexible pavements. Concrete has a very high modulus of elasticity, which results in great load spreading in the top layer and hence low stresses in the underlying substructure, consisting of the subbase and subgrade. The concrete layer thus carries the majority of the applied load, and the distribution of stresses to the lower layers is low. Concrete pavements can therefore be constructed on poor subgrades, and generally have fewer pavement layers than flexible pavements.

3.4.3 Concrete Block Pavements

The behaviour of concrete block pavements is somewhere between the behaviour of flexible and concrete (rigid) pavements. The blocks carry much of the load, but some load is distributed to the underlying layers. Pavement balance should be considered for the support layers.

3.5 Pavement Behaviour

A proper understanding of basic pavement behaviour is required before attempting a pavement design. This includes the behaviour under loading, load sensitivity, long-term pavement behaviour and materials.

3.5.1 Behaviour Under Loading

The effect of vehicle loading on a pavement is relatively small, when considering each vehicle or loading individually. However, the cumulative effect of many such loads causes distress in the pavement. An understanding of the short term effect of loading on a pavement provides a good background for how the cumulative affects manifest, and are modelled.

Under the action of a moving vehicle load, the pavement deflects, and rebounds when the load has moved away. The effect of a heavy vehicle load generally extends over an area of 1 to 2 metres from the point of loading, in all three directions. This deflected area tends to form a circular, deflected indentation known as a deflection bowl. The size and shape of deflections bowls vary and depend on the pavement structure, the strength and stiffness of the materials, pavement balance, temperature and of course, the loading magnitude, duration and contact area. For flexible pavements in a good condition, the maximum deflection is typically less than 500 microns under a standard axle load.

The most common method of measuring pavement deflections is with the Falling Weight Deflectometer (FWD), shown in Figure 7. The FWD measures the deflections with sensors placed on the road surface. A good reference for most aspects of FWD measurements is "Guidelines for Network Level Measurement of Pavement Deflection", (COTO, 2009)

Figure 7. Falling Weight Deflectometer (FWD)

The pavement layers influence the deflection bowl. This is illustrated in Figure 8 for a simple three layer pavement structure. The stress distribution through the pavement from the FWD loading is represented by the black curve. As the load is distributed into the pavement, the stress distribution increases, although the intensity of the stress is reduced. The deflection bowl is represented by the white line, and the blue arrows indicate the locations of the FWD sensors. The sensor immediately underneath the FWD load measures the largest deflection. All the layers contribute to that deflection. The sensor furthest away from the load measures a deflection that is generated from the subgrade. The closer the sensor to the load, the more layers contribute to the deflection. The deflection bowl, therefore, provides information about the individual layers, by investigating the shape of the bowl. For example, deflection bowl from a pavement with a weak base layer shows an increased deflection at the first three sensors. Section 7.5 presents some guidelines for using deflections to analyse and characterise pavements, and also to identify problem layers in the pavement.

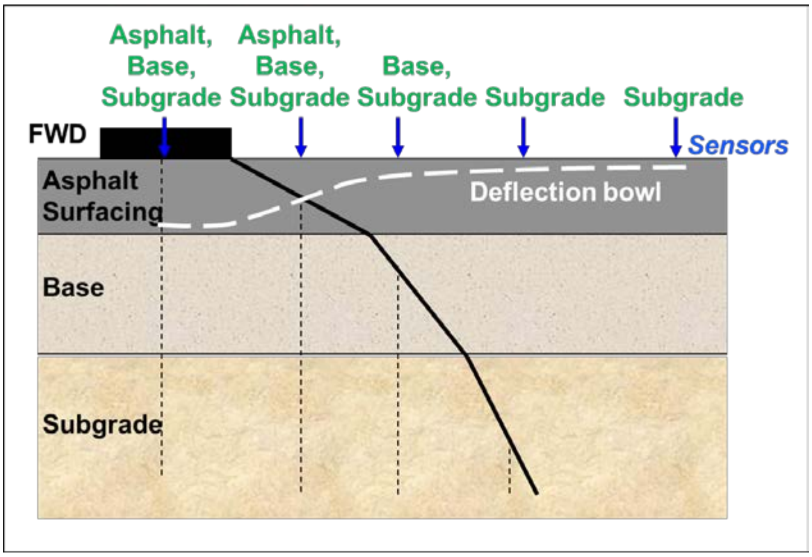


Figure 8. Stress Distribution Versus Deflections

The influence of the pavement structure on the deflection bowls is illustrated by three different scenarios shown in Figure 9:

- **Scenario 1** is a **stiff pavement**, with a relatively stiff and strong cemented subbase layer. The deflection is relatively low, and the bowl wide in comparison to its magnitude.
- **Scenario 2** is a pavement that is **relatively old**, but has **good quality materials**. The deflection is higher than Scenario 1, because the pavement is less stiff.
- **Scenario 3** is an **old pavement with that has poor quality materials**, and has a moist subbase and subgrade. The deflection is large, and the width of the bowl is narrow.

Section 7.5 later in this chapter, and Chapter 2, Section 9 contain discussion on the behaviour of various pavement layer types under vehicle loading.

3.5.2 Load Sensitivity

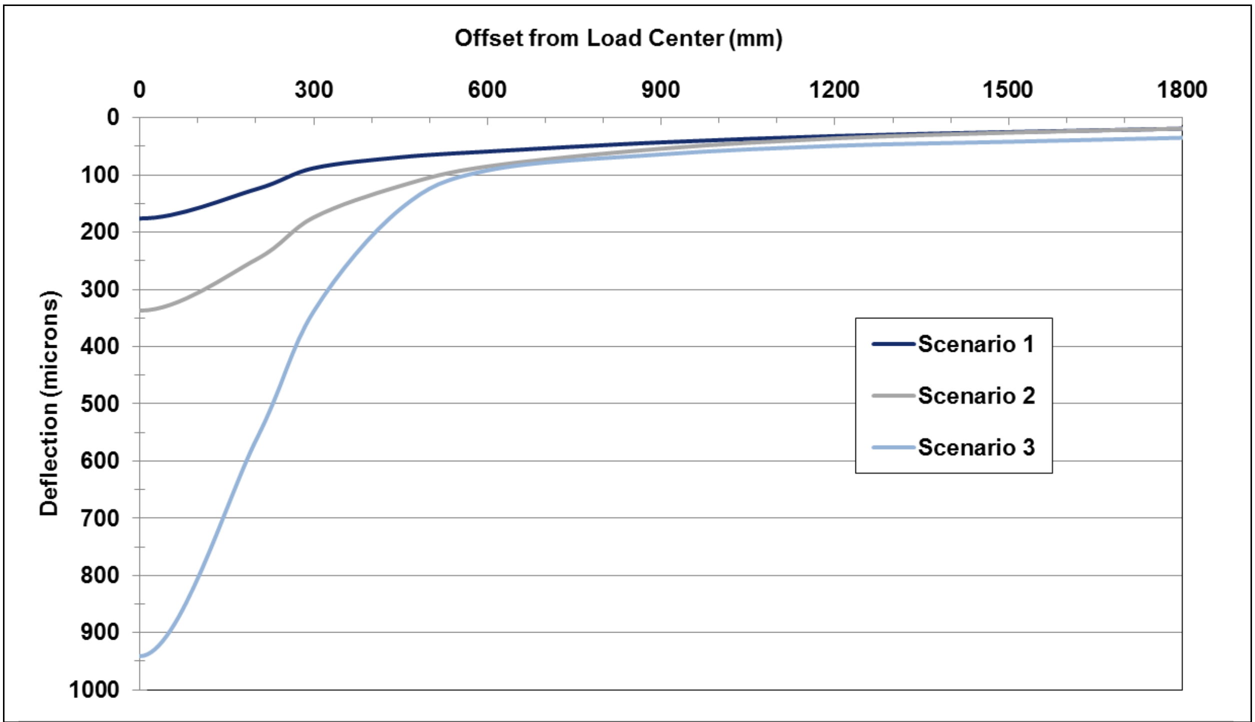
The load sensitivity of a pavement is typically used in the conversion of the traffic axle load spectrum to an equivalent design traffic estimate in terms of standard axles. The type of pavement and pavement balance determine the load sensitivity of a particular design. Hence, the same traffic load spectrum may convert to different equivalent standard axle values, depending on the load sensitivity. Load sensitivity and the conversion of actual axle loads to standard axles are discussed in Section 4.1.3.

3.5.3 Long-Term Pavement Behaviour

The typical long term behaviour under loading of flexible, rigid and concrete block pavements is different, as described below.

3.5.3.1 Flexible Pavements

The long term behaviour and distress of the different types of flexible pavement share some general phases and trends, as illustrated in Figure 10.



Scenario 1		Scenario 2		Scenario 3	
40 mm	Asphalt	60 mm	Old asphalt with seals	30 mm	Old asphalt
125 mm	G1 Base	250 mm	G2 Base	150 mm	G4 Base
250 mm	C3 Subbase	250 mm	G4 Subbase	250 mm	G5 Subbase
300 mm	G7 Selected	150 mm	G6 Selected	450 mm	G7 Selected
	G9 Subgrade		G9 Subgrade		G7 Subgrade

Figure 9. Example Deflection Bowls

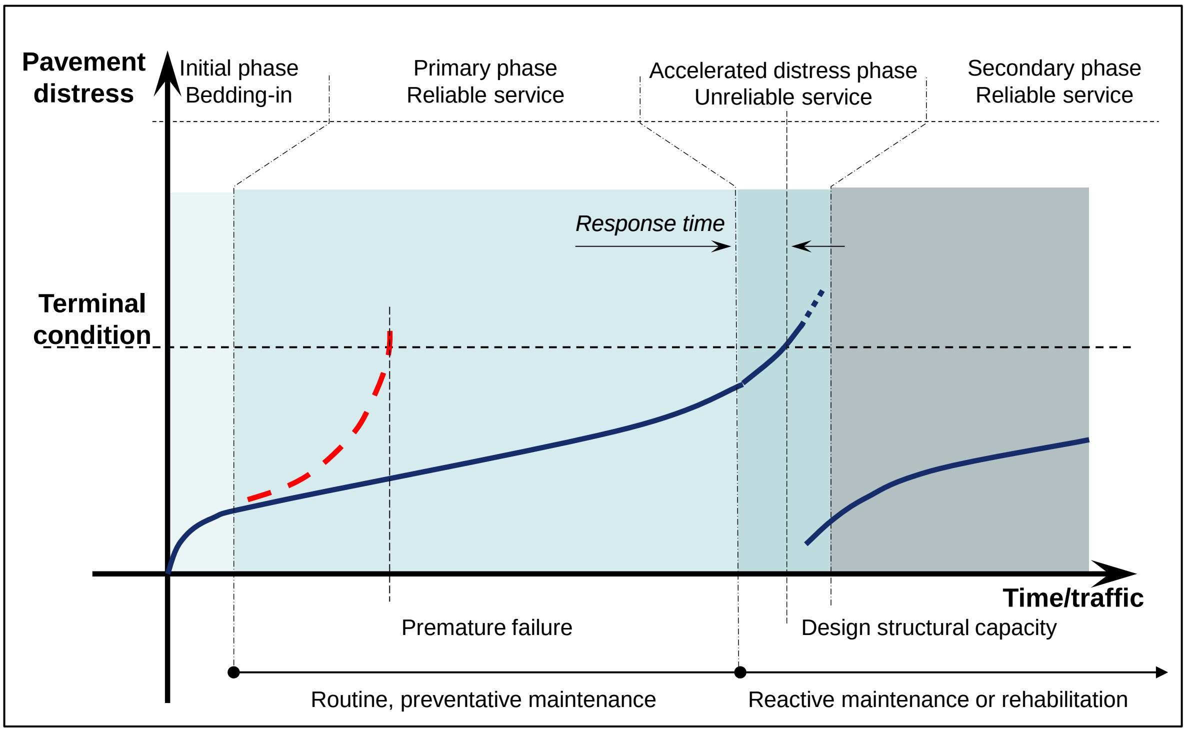


Figure 10. Generalized Behaviour of Flexible Road Pavements

- **Initial phase.** During the initial phase, some bedding-in occurs. The rate at which any particular type of distress increases may be high. The absolute value of the distress is, however, still well below the limits normally indicating a terminal condition.
- **Primary phase.** The rate at which the distress increases normally reduces fairly rapidly to an almost constant rate during the primary phase. Reliable service is expected during the primary phase as long as the appropriate routine, preventative maintenance is done. Premature failure may, however, occur due to poor construction, a lack of maintenance, extreme overloading or unexpected deterioration of the materials used in the pavement.
- **Accelerated distress phase.** During this phase, the rate of increase in distress becomes unstable. A terminal condition may be reached if the response time of the Pavement Management System (PMS) is too long, and reactive maintenance and rehabilitation is not done in time.
- **Secondary phase.** If reactive maintenance or rehabilitation is done in time during the accelerated distress phase, a secondary stable condition may be entered, extending the life of the facility beyond the initial structural design life.

The common flexible pavement types used in South Africa are pavements with the following base layers:

- Unbound granular
- Lightly cemented
- Bitumen stabilised
- Hot mix asphalt

These base layers are supported by granular or lightly cemented subbase layers. In addition to crushed stone, high quality natural gravel may be used in base layers on roads carrying light traffic. Figure 11 shows the general long-term distress behaviour of these pavement types. Flexible pavements generally deteriorate gradually over time. Pavement structures with lightly cemented bases initially show little distress, but deteriorate rapidly once distress initiates. Remedial action on pavements with lightly cemented bases is therefore urgent, once signs of distress are noticed.

In addition to the changes in distress, the elastic response of pavements with lightly cemented layers also changes with time, as illustrated in Figure 12.

Equivalent Granular State

The equivalent granular state is when a lightly cemented layer has cracked or weakened to the extent that the effective stiffness is similar to that of an unbound granular layer. The "cracked" state does not imply the material has reached the consistency of a granular material, or that it has necessarily visibly cracked into smaller, granular like pieces. The cracks are generally micro-cracks that are not that visible, but result in a loss of stiffness.

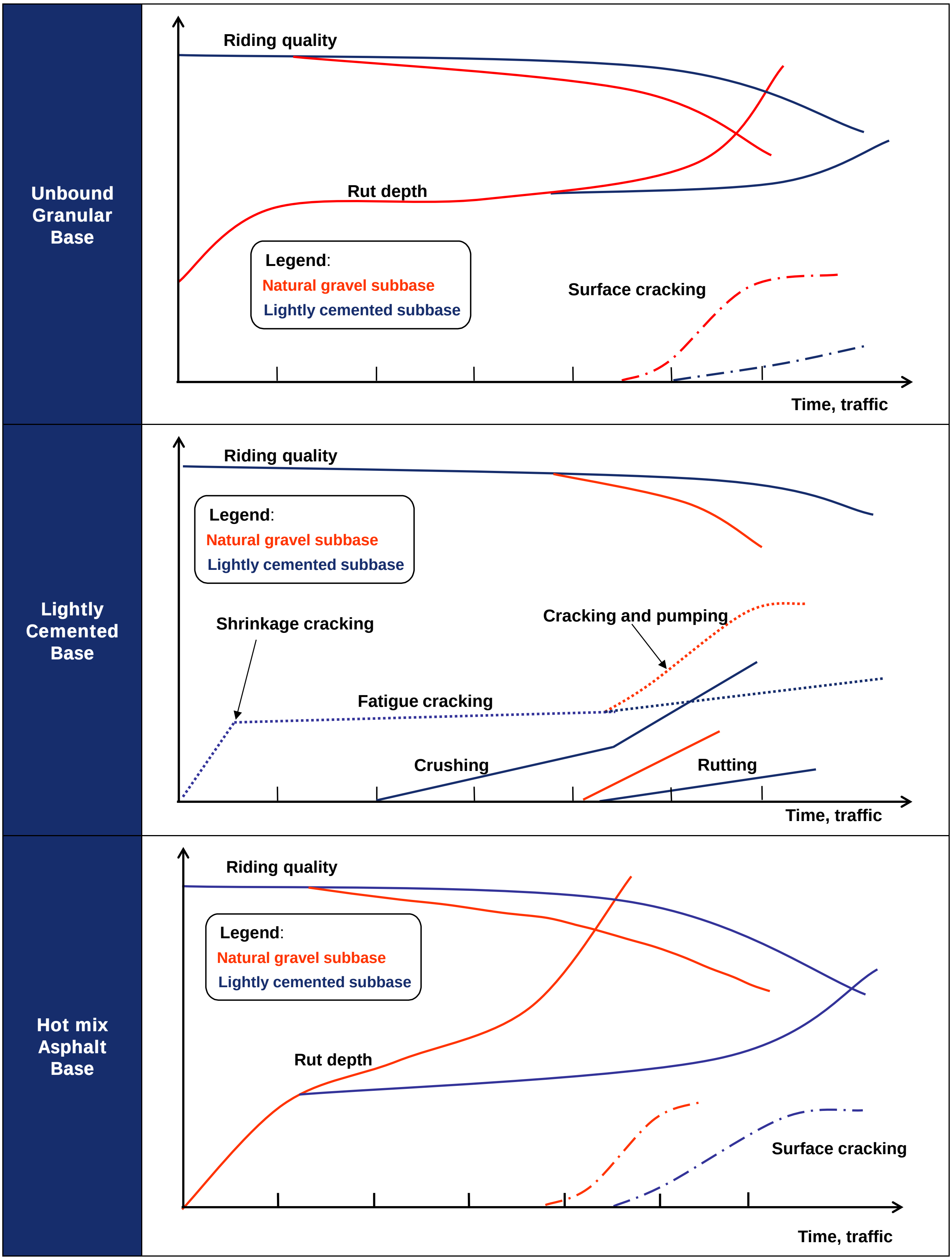


Figure 11. Long-Term Changes in the Functional and Structural Indicators of Flexible Pavements

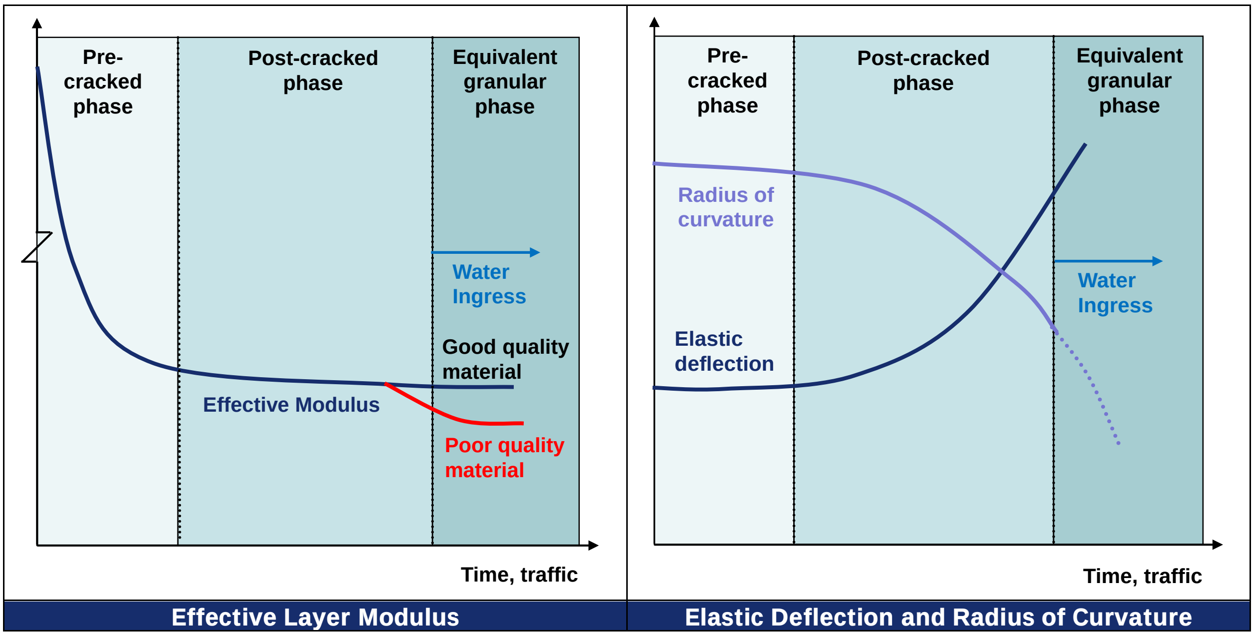


Figure 12. Long-Term Changes in the Elastic Response of Flexible Pavements with Lightly Cemented Layers

3.5.3.2 Concrete Pavements

The structural performance of concrete pavements is defined in terms of the percentage of road with the following types of distress:

- Shattered slabs
- Faulting
- Pumping
- Crack spacing in the case of continuously reinforced concrete pavements (CRCPs)

Shattering of slabs normally initiates at joints in plain jointed concrete pavements and jointed reinforced concrete pavement, or at closely spaced transverse cracks in CRCP, resulting in punch-outs. The percentage of the road surface with shattered slabs is the percentage of slabs or areas with two or more interconnected cracks, with pumping possible in bad cases, and the area that needs to be repaired.

The performance of CRCPs also depends on crack spacing. If the crack spacing is less than 0.5 m, the pavement loses stiffness, stress on the subgrade support increases, and punch-outs can develop. Areas of the pavement with less than 0.5 m crack spacing are a high risk area for punch-outs.

Areas that exhibit faulting in excess of 5 mm are a function of the erosion and pumping of the subbase, as well as the deflection at the joints or cracks under traffic loading, which is dependent on the effectiveness of load transfer between the slabs.

The long term behaviour of concrete pavements is different for plain jointed, jointed reinforced or continuously reinforced concrete pavements. The long term changes in the functional and structural indicators of rigid pavements are shown schematically in Figure 13.

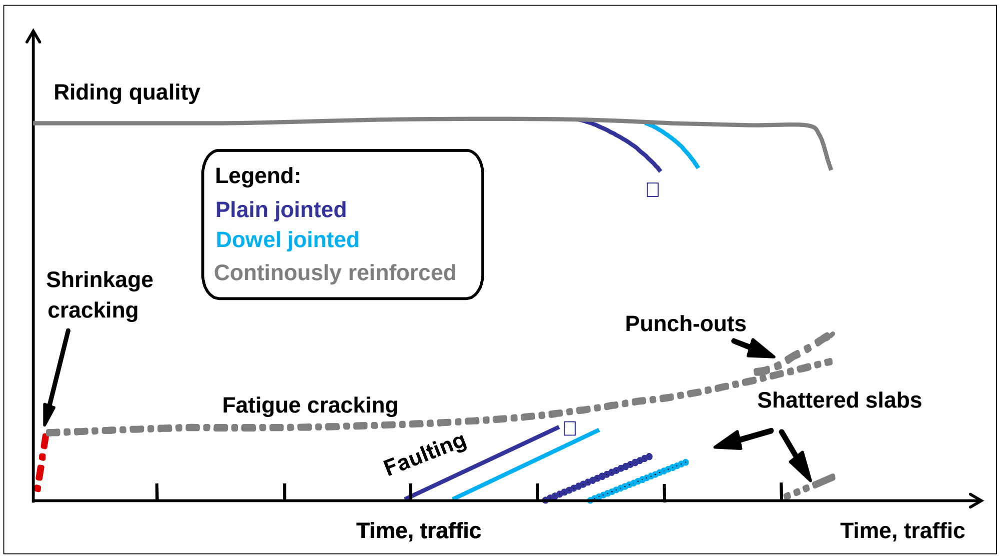


Figure 13. Long-Term Changes in Functional and Structural Indicators of Rigid Pavements

3.5.3.3 Concrete Block Pavements

Under traffic, block pavements tend to gradually accumulate rutting. In this respect, the performance of block paving is similar to that of conventional flexible pavements (refer to rut depth in Figure 11).

The blocks spread concentrated loads over a wide area of earthworks layers. This means that blocks do not merely act as a wearing course, but also as a load bearing course. The blocks have significant structural capacity when properly installed. The blocks themselves are generally hardly affected by high surface stresses. However, wear or abrasion of the blocks has been observed in some applications.

The performance of block pavements is highly dependent on interlocking of blocks, narrow joints between blocks, relatively thin bedding sand and a stabilised subbase. The sand used in the joints should be of high quality, preferably crushed material, and impermeable. The bedding sand should be continuously graded, but permeable crushed material. A stabilised subbase is preferred, because of increased support to the bedding sand, its structural stiffness, and its protection of the sub-layers. All these aspects are discussed in more detail in Chapter 9, Section 14.

Under traffic, concrete block pavements tend to stiffen, provided the blocks are "locked" in between curbs or beams on the edges to prevent widening of the joints between the blocks. This leads to the pavements achieving a quasi-equilibrium or 'lockup' condition, beyond which no further deformation occurs. Often the increase in stiffness in the block layer that accompanies lockup is substantial. After lockup, it may be possible to increase the loads applied to the pavement without causing damage. The development of lockup is contingent upon careful control of construction standards and layer works quality. For example, subbase layers of low bearing capacity do not permit the development of interlock during the early life of the pavement. Where conditions are favourable for achieving interlock, it can be allowed to develop gradually under traffic or may be more rapidly induced by proof-rolling.

3.5.4 Materials

Structural layers are designed to resist the high stress conditions imposed by external loads and to provide sufficient protection of the pavement foundation during the structural design period. Material of appropriate quality therefore needs to be sourced for the imported subgrade and structural pavement layers. Chapters 6, 8 and 9, and TRH3 (2007), TRH14 (1995) and TG2 (2009) provide detailed information on the behaviour of specific material types. The use of a materials checklist indicating the availability and cost of materials in the vicinity of the project is recommended.

3.6 Environmental Considerations

The environment in which the pavement is situated may be divided into:

- **Traffic** environment
- **Natural** (geological and meteorological) environment
- **Population** (rural or urban) environment

The traffic environment is the primary demand for which the pavement is designed. However, the natural and population environments also affect pavement behaviour and performance, and must therefore be considered during pavement design. The effect of environmental conditions on pavement performance is particularly important for light pavement structures.

The differences between rural and urban environments usually do not have a significant impact on the structural design of the pavement, but impact on the selection and design of the surfacing layer and drainage. Surfacing layer selection and design is a specialised activity, for which the following documents provide guidance:

- TRH3, 2007
- Guidelines for Upgrading Low Volume Roads (SARB, 1993a)
- Towards Appropriate Standards for Rural Roads: Discussion Document (SARB, 1993b)

Details on subsurface drainage are provided in:

- Drainage Manual, 5th edition (SANRAL, 2006)

3.6.1 Geology and Climate

The geology and climate of a region largely determines the characteristics of the in situ subgrade for new roads. Geological issues are discussed in much detail in Chapter 6, Road Prism and Pavement Investigations, Section 4 and Chapter 7, Geotechnical Investigations and Design Considerations, Sections 2 and 3.

3.6.1.1 Flexible Pavements In Situ Subgrade

(i) Material Depth

The material depth denotes the depth below the surface of the finished road that soil characteristics significantly affect pavement behaviour. Above this depth, the pavement strength must be sufficient for the traffic imposed stresses. Below this depth, the traffic imposed stress conditions are assumed to have dissipated and the material quality exceeds strength requirements. The moisture condition above the material depth has a major influence on the material strength, and needs to be controlled by providing adequate surface and subsurface drainage. The material below the material depth must be competent to support the pavement structure. The material depths shown in Table 5 are recommended for the different road categories for flexible pavements. Chapter 9, Section 2 provides additional discussion on the material depth.

Table 3. Recommended Material Depth for Each Road Category

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

Material depths are normally not considered for concrete pavements, provided that the support is consistent.

(ii) Minimum Subgrade Strength and In Situ Subgrade Delineation

A key pavement design principle is that the subgrade provides an adequate foundation for the pavement layers. A minimum Californian Bearing Ratio (CBR) of 15% is generally required for flexible pavements.

To determine the adequacy of the in situ subgrade, it is divided into sections based on the CBR, using the ranges in Table 4. The 10th percentile CBR value is determined for each region, and must exceed the minimum value in the range, i.e., 90% of the CBR results should exceed the specified lower limit. If the strength of a particular subgrade section does not meet the minimum strength requirement, layers of increasing quality are imported to ensure that the minimum is achieved. Following the recommendations from Table 6, the pavement foundation is built-up, as illustrated in Figure 14. When the road is constructed on a fill, the material should be controlled to at least the minimum material depth specified in Table 3.

Table 4. In Situ Subgrade Delineation for Flexible Pavements

CBR (%)[1] of delineated subgrade sections	Action
> 15	• In situ subgrade of a G7 standard and of sufficient strength to support structural layers. • Rip and recompact to 93% of modified (mod.) AASHTO density.
7 to 15	• In situ subgrade of a G9 standard. • Rip and recompact in situ material to 93% of mod. AASHTO density. • Import a 150 mm thick layer of G7 standard material.
3 to 7	• In situ subgrade of a G10 standard. • Rip and recompact in situ material to 93% of mod. AASHTO density. • Import a 150 mm thick layer of G9 standard material. • Import a second 150 mm thick layer of G7 standard material.
< 3	• Chemical/mechanical stabilisation • Or, remove and import new material. • Or, add additional cover to place poor quality in situ material below material depth.

Note
1. CBR at 93% modified AASHTO density

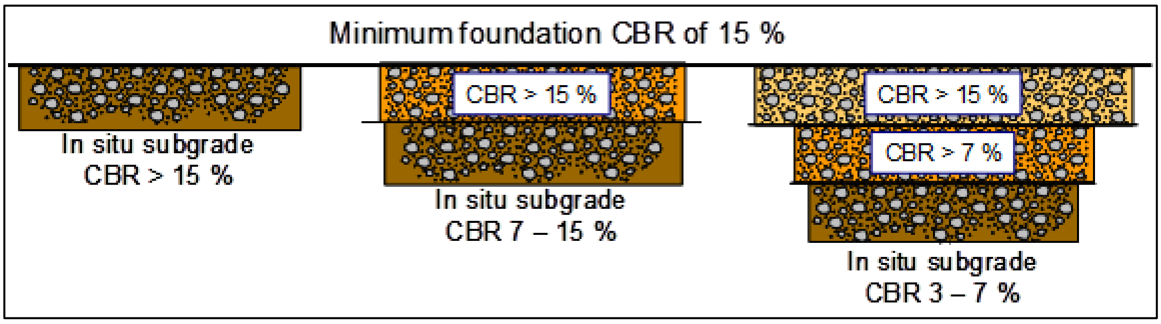


Figure 14. Importing Layers to Obtain Minimum Subgrade Strength

3.6.1.2 Concrete (Rigid) Pavements

Slab support influences the performance of rigid pavements. It is more important to have a uniform slab support than a strong, but variable, support. The prevention of voids under the slab due to erosion, plastic deformation and subsidence and swelling clays is also important. A strong foundation support is not necessarily required, because of the high stiffness, and therefore, good load spreading ability of concrete.

3.6.1.3 Block Pavements

The in situ subgrade requirements for block pavements are the same as flexible pavements.

3.6.2 Meteorological Environment

The meteorological environment is divided into macro-climatic regions with different moisture and temperature conditions. Figure 15 shows the macro-climatic regions for moisture conditions based on Thornthwaite's Moisture Index (Paige-Green, 2012). Thornthwaite is a function of evapo-transpiration, i.e., dependent on vegetation and rainfall. It determines the moisture deficit and is more accurate than Weinert's N value, which is season dependent. Thornthwaite also provides a more sensitive differentiation in the climatic regions than Weinert. The index is interpreted as shown in Table 5.

Table 5. Interpretation of Thornthwaite's Index

Index Range	Climatic Region	
< – 40	Arid	Dry
– 40 to – 20	Semi-Arid	
– 20 to 0	Dry Sub-humid	Moderate
0 to 20	Moist Sub-humid	
> 20	Humid	Wet

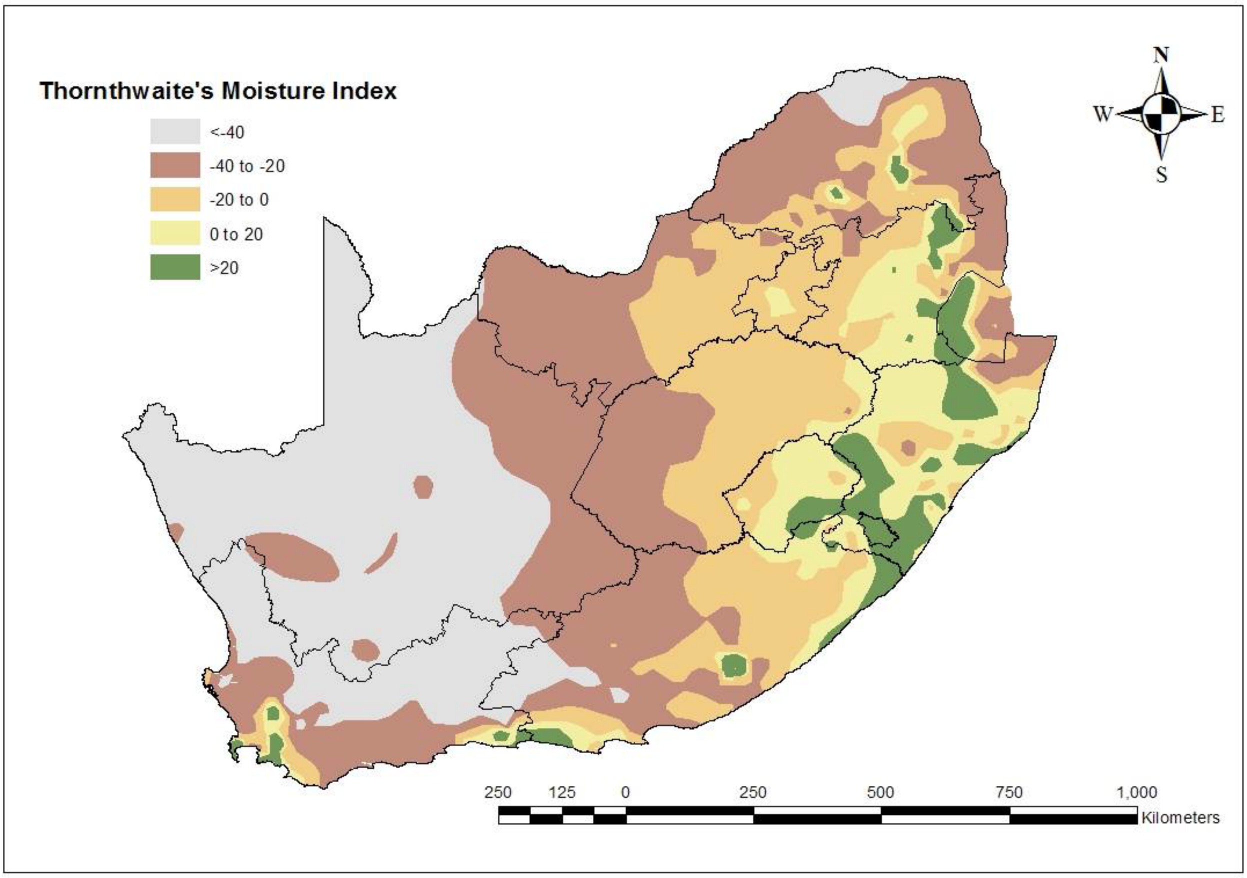


Figure 15. Southern Africa Macro-Climatic Regions Based on Thornthwaite's Moisture Index

The moisture condition affects the weathering of rock, the durability of weathered material and in combination with drainage conditions and the surfacing layer integrity, the moisture content and stability of unbound granular layers. Cognisance must also be taken of local micro-climates within the macro-climatic regions, as these may deviate substantially from the macro-climatic norm. An example is where orographic rainfall occurs in mountainous areas.

Temperature conditions largely affect the design of surfacing layers, particularly hot mix asphalt. Prediction of the temperature in these layers using ambient temperatures and solar radiation, for South African conditions, has been developed (Viljoen, 2001, Denneman, 2007a and b).

3.7 Practical Design Considerations

Poor drainage and insufficient compaction are probably responsible for more pavement failures than poor structural and material design. Problems related to the subgrade below the material depth must be considered. Other practical considerations that may impact on the design are variable cross-sections and paved shoulders. These considerations are discussed in the following sections.

3.7.1 Drainage

Efficient drainage of a pavement structure plays a vital role in the performance. It is, therefore, imperative that the drainage design of the pavement structure is completed in great detail. Adequate surface and subsurface drainage should be provided. SANRAL's Drainage Manual (2006) provides guidelines for drainage design.

Drainage Design

Efficient drainage is an essential part of good pavement performance. The following guideline provides details for drainage design:

- **Drainage Manual**, 5th edition, SANRAL, 2006. Download at www.nra.co.za.

Effective drainage should be provided to at least the material depth. Special care should be exercised in cuttings. Cuts and fills on side-slopes often lead to drainage problems. Proper drainage facilities on the embankment side are well worth the investment. See Chapter 7, Sections 4 and 5 for more on cuts and fills.

3.7.2 Compaction

Insufficient compaction may result in field densities below the minimum required. In such cases, the strength of the material is not fully utilised, and densification or failure may occur under traffic. Quality control on site must ensure that the design specifications are met through proper construction and compaction practices. TRH14 (1985) and COLTO (1998) provide recommendations on the minimum density requirements for pavement layers. Any special density requirements should be discussed with the relevant road authority. Chapters 12 and 13 discuss many aspects of compaction.

Poor Drainage and Insufficient Compaction

Poor drainage and insufficient compaction are probably responsible for more pavement failures than poor structural and material design.

Compaction problems may result from material grading deficiencies or poor construction practices, such as not compacting at the optimum moisture content, poor mixing of the material and compaction fluid, or insufficient or inappropriate compaction energy. Blending of material from different sources, to improve the grading and compaction potential of the material, may be better than trying to achieve density with excessive rolling.

When compacting a layer, the support layer needs sufficient support to act as an anvil, otherwise the compaction energy is transmitted and lost through the pavement structure. The use of impact rollers can improve the strength and support from the subgrade substantially. Impact rollers are discussed in Chapter 12, Section 2.10.1.

3.7.3 Variable Cross Sections and Paved Shoulders

On multi-lane roads or roads with climbing lanes, the traffic loading may be significantly different between the lanes. In these cases, the pavement design may be adjusted accordingly. For example, the slow lane of heavily trafficked roads is often constructed in concrete and the remainder of the pavement in granular, cemented, BSM or asphalt layers. Care should be exercised not to trap water in the pavement when using layers of different thickness or material type across the width of the road, including the shoulder. The complexity of implementing such a design should also be considered, with reference to construction sequences, accommodation of traffic, and the construction period.

Category A and B roads normally have paved shoulders. There is a zone of seasonal moisture content variation towards the edge of the pavement (Emery, 1984 and 1985). The recommended minimum paved shoulder widths in Table 6 should be used to prevent the zone of influence of the outer wheel-path overlapping with the seasonal moisture content variation zone. A paved shoulder is more important in wet regions than moderate or dry regions. Low volume roads seldom justify a surfaced shoulder. The decision to pave road shoulders also depends on the traffic expected on the road, locations where vehicles pull of the road, and erosion protection requirements.

Different Pavement Structures Across the Width of a Road

When the traffic levels are different across different lanes, the use of varying pavement structures is sometimes justified.

However, care should be exercised not to trap water in the pavement when using layers of different thickness or material type across the width of the road, including the shoulder.

Table 6. Recommended Minimum Paved Shoulder Widths

Road Category	Paved Shoulder Width (mm)
A	1 200
B	1 000
C	800

If a paved shoulder is provided, the structural capacity of the paved shoulder should be sufficient to carry traffic that may use the shoulder. See Section 4.4.1 on design traffic estimation for lane distribution factors, including paved shoulders.

3.8 Pavement Type Selection for New and Rehabilitated Pavements

Certain pavement types may not be suitable for some road categories or traffic classes. Table 7 gives recommended pavement types for road categories and traffic classes. Brief reasons why certain pavement types are not recommended for some combinations are also given. As a general rule, pavement structures with thin rigid or stiff layers at the top (shallow structures) should be avoided if many overloaded vehicles are expected. An exception to this general rule is concrete pavements, where the concrete thickness is calculated to be able to carry the expected heavy traffic.

Table 7. Recommended Pavement Types for Road Category and Design Traffic Class

Pavement Type		Road Category & Design Traffic Class								Reasons for Exclusion
		A		B			C			
Base	Subbase	ES100	ES30	ES10	ES3	ES1	ES3	ES1	< ES0.3	
Concrete	Granular	✗	✗	✓	✓	✓	✓	✓	✓	Granular subbases prone to erosion at joints and cracks
	Cemented	✓	✓	✓	✓	✓	✓	✓	✓	
Granular	Granular	✗	✓	✓	✓	✓	✓	✓	✓	Uncertain behaviour for high traffic demand
	Cemented	✓	✓	✓	✓	✓	✓	✓	✓	
Hot mix asphalt	Granular	✓	✓	✓	✓	✗	✓	✗	✗	Cost effectiveness
	Cemented	✓	✓	✓	✓	✗	✓	✗	✗	
Cemented	Granular	✗	✗	✗	✗	✗	✗	✗	✓	Cracking, crushing, rocking blocks and pumping unacceptable
	Cemented	✗	✗	✗	✓	✓	✓	✓	✓	
BSMs	Granular	✗	✓	✓	✓	✓	✓	✓	✓	Cost effectiveness, permanent deformation

There is no design method that accurately predicts the condition of a length of road 10 to 20 years in the future. However, certain distresses can be expected in particular pavement types by the end of the structural design period. Rehabilitation options for these distresses need to be considered during the original design. Table 8 provides guidance for expected terminal conditions. The distress mechanisms are illustrated and discussed in Chapter 14, Section 3.

Table 8. Typical Terminal Conditions and Rehabilitation Options

Pavement Base Layer	Terminal Distress Condition[1]	Possible Rehabilitation Options[2]
Concrete	• Shattering • Pumping • Faulting • Punch-outs	• Local repairs, slab replacement and seal or overlay • Crack and seat with seal and/or overlay • Treatments of joints important for the overlay of plain and dowel jointed concrete pavements
Granular (Crushed Stone and Natural Gravel)	• Base stable • Top-down distress	• Local repairs and seal or overlay, or mill and replace surface
	• Crocodile cracking • Loss of surfacing • Pumping of fines • Potholes • Patching • Permanent deformation (rutting) • Unstable	• Recycle with stabilisation for excessive distress and seal or asphalt overly • Pre-treatment patching with overlay • Recycle layer as subbase and construct new base with surfacing
Hot Mix Asphalt	• Fatigue cracking • Pumping	• Recycle with stabilisation and seal or overlay • Pre-treatment patching with overlay • Utilise recycled layer as subbase and construct new base and surfacing • Mill and replace (inlay)
	• Permanent deformation (rutting)	• Mill and replace with stable material including concrete
Cemented	• Cracking • Crushing • Rocking blocks • Pumping • Carbonation or durability problems with deformation and shear	• Recycling and stabilise and new surfacing • Utilise recycled layer as subbase and construct new base and surfacing
Bitumen Stabilised Materials	• Cracking • Permanent deformation	• Recycling with stabilisation and surfacing • Utilise recycled layer as subbase and construct new base and surfacing

Note
1. Distress types illustrated and discussed in Chapter 14, Section 3.
2. Overlays and inlays of all pavements can be asphalt or concrete.

4. DESIGN TRAFFIC ESTIMATION

The composition of vehicle traffic on roads is highly varied, ranging from light passenger vehicles to buses and heavy vehicles transporting commercial goods. The flow of the combined light and heavy vehicles is important for traffic engineering. However, the pavement design process focuses on the response of pavements to loading, specifically the volume and loading of heavy vehicles.

The study and quantification of traffic loading is complicated by its extremely variable nature. There are two modes of land-based transport for commercial goods, rail and road. Shifts in the preferred mode of transport from rail to road results in changes in the traffic loading. Economic growth requires good transport infrastructure, which also affects the long-term growth in traffic volumes. Sporadic construction activity causes an increase in specific heavy vehicle types for the duration of the construction project. Seasonal activities such as harvesting affect the traffic on certain routes.

Changes to legal axle load limits, and the level of enforcement also affects the loading and overloading per vehicle, as do changes in the mechanical design and load carrying capacity of vehicles. Therefore, a host of factors are considered to properly assess the volume, loading, seasonal fluctuation and long-term changes in traffic, specifically heavy vehicle traffic.

The consequence of an incorrect design traffic estimate is serious and costly. Although the calculations used in the process of estimating the design traffic are fairly simple, the high uncertainty associated with the inputs often leads to low confidence in the results.

The national guideline document for design traffic estimation is TRH16: Traffic Loading for Pavement and Rehabilitation Design (1991). TMH3: Traffic Loading Load Surveys for Pavement Design (1988) specifies the methods for axle load surveys. This section provides an overview of the design traffic estimation process, loosely following TRH16.

Abbreviations used in Traffic Analyses

- **AADE:** annual average daily equivalent traffic
- **AADT:** annual average daily traffic
- **ADE:** average daily equivalent traffic
- **ADT:** average daily traffic
- **ADTT:** average daily truck traffic
- **B_E:** lane distribution factor
- **CTO:** Comprehensive Traffic Observations Stations
- **E80:** standard axle load (80 kN per axle)
- **E80/HV:** E80 per heavy vehicle, considering all axles
- **ES0.003 to ES100:** TRH4 design traffic classes
- **ET:** cumulative equivalent traffic
- **$F_{7/5}$:** adjustment of 5 day to 7 day traffic count
- **F_E:** adjustment for exceptional periods
- **F_S:** adjustment for seasonal variation
- **HVV:** heavy vehicle volume
- **LEF:** load equivalency factor
- **MESA:** million equivalent standard axles
- **n:** damage exponent in power damage law
- **sdp:** structural design period
- **SHV:** short heavy vehicle
- **WIM:** weigh-in-motion

The components of the process are presented in the following sequence:

- **Concepts** and terminology
- Traffic **investigation** for pavement design
  - General considerations
  - Loading surveys
- Design traffic **calculations**

4.1 Concepts and Terminology

The concepts and terminology associated with design traffic terminology, include:

- Legal axle loads
- Axle load histograms
- Standard axles and equivalent standard axles
- E80s per heavy vehicle (E80/HV)
- Traffic classes

References for Traffic Measurement and Estimation

Good references for Traffic Measurement and Estimation for Pavement Design are:

- TRH16, **Traffic Loading for Pavement and Rehabilitation Design** (1991)
- TMH3, **Traffic Loading Load Surveys for Pavement Design** (1988)

TRH4 (1996) and TRH12 (1997) also include detailed sections on handling traffic.

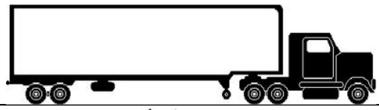
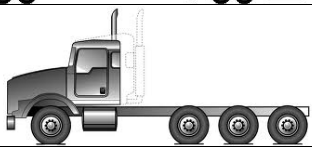
All of these guidelines can be downloaded from www.nra.co.za.

4.1.1 Legal Axle Loads

The permissible axle load limits, in terms of the average static axle mass for different axle groups, are set by Regulations 234 to 240 of the Road Traffic Act of 1996, and summarized in Table 9. Note that the latest version of TRH16 was published in 1991, and is based on the 1989 Act.

TRH16 Based on 1989 Road Traffic Act
The latest version of TRH16, published in 1991, is based on the 1989 Road Traffic Act. The permissible axle load limits were increased in the 1996 Road Traffic Act. See Table 9 for the changes.

Table 9. Permissible Axle Loads

Vehicle	Axle Group		Wheel Configuration	Permissible Static Mass (kg) Pre-1996[1]	Permissible Static Mass (kg) Post-1996[2]
Truck	Steering		Single	7 700	7 700
	Single		Dual-wheel	8 200	9 000
			Single	7 700	8 000
	Tandem		Dual-wheel	16 400	18 000
			Single	15 400	16 000
	Tridem		Dual-wheel	21 000	24 000
			Single	21 000	24 000
Bus	Steering		Single	7 700	7 700
	Single		Dual-wheel	10 200	10 200

Note
1. Based on 1989 Act, and used in TRH16.
2. After the increase in legal axle loads in 1996.

The permissible load for single wheels on axles other than steering axles, is less than for dual-wheels for single and tandem axles. The use of wide-based super single tyres is limited in South Africa. No limits are included in the legislation for tyre pressures. As a result, tyre pressures are dictated by tyre specifications and vehicle fleet operators.

Tyre Pressures
The Road Traffic Act does not provide limits for tyre pressures. As a result, tyre pressures are dictated by tyre specifications and the vehicle fleet operators.

In enforcing the legislation, a 5% grace is allowed on top of the permissible loads before prosecution. Poor enforcement of the legislation often results in actual axle loads significantly exceeding the legal limits. Although the permissible loads and the level of enforcement are two important factors that determine the distribution of actual axle loads on roads, these two factors are not directly included in design traffic calculations. Their influence is therefore indirectly accounted for through the distribution of actual axle loads.

Traffic Investigation and Design Traffic Investigation
Traffic investigation and design traffic estimation are specialised activities. Incorrect traffic data and inaccurate design traffic estimates contribute to design risk as much as an inaccurate structural capacity estimate. Traffic surveys and design traffic estimation are, unfortunately, often neglected during the pavement design process, as most of the effort is focussed on the structural capacity estimation component. Details on traffic surveys and design traffic estimation techniques are presented in Sections 4.2 and 4.7.

4.1.2 Axle Load Histograms

The permissible axle loads and their enforcement set an upper bound for in-service axle load distributions. Operating conditions, such as the vehicle load carrying capacity and overloading, determine the in-service distribution of axle loads. This distribution is therefore normally wide, and cannot be represented by a single value.

The details of axle load surveys are covered in Sections 4.6.4. Such surveys result in thousands of individual axle mass observations. Axle load histograms summarise this huge volume of axle load data into a limited set of categorised data by counting the number of observations within predefined categories, as illustrated Figure 16. The distribution histogram represents the actual data, but does not contain the actual data.

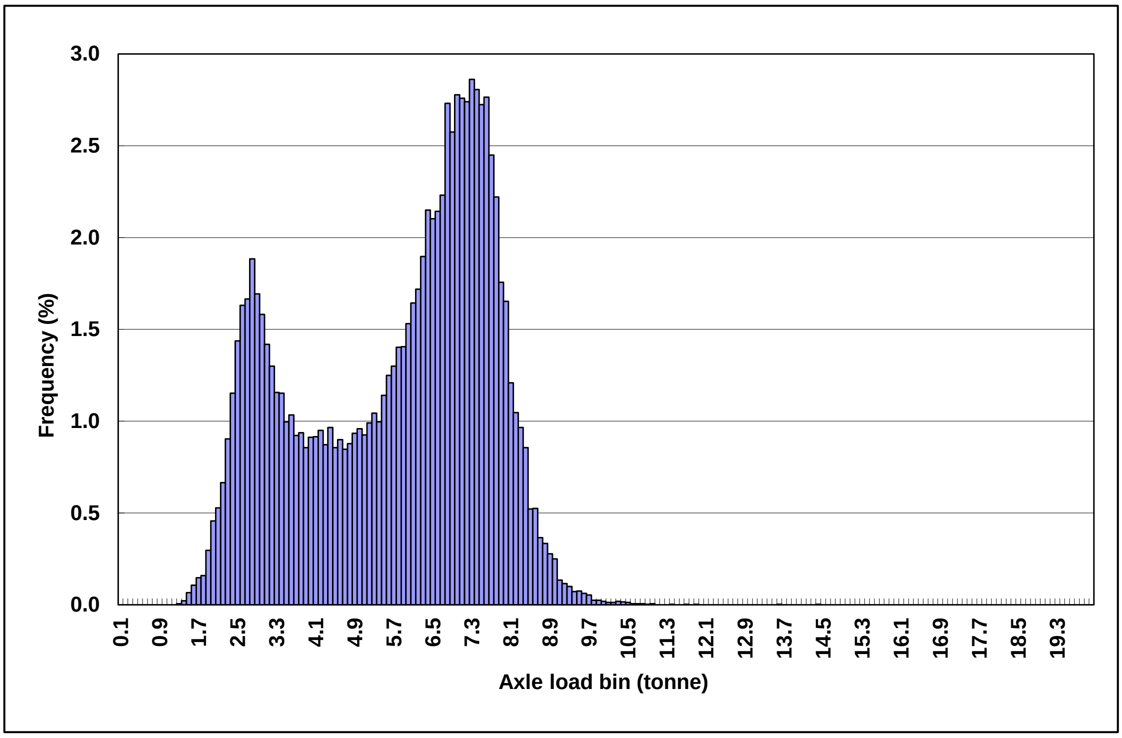


Figure 16. Example of an Axle Load Histogram

4.1.3 Standard Axles and Equivalent Standard Axles

The standard axle load concept originated from the AASHO road test (Highway Research Board, 1961 and discussed in Chapter 2, Section 2.2.1). It is based on the principle that any load may be converted to an equivalent number of standard axle loads, based on the damage done by the load in relation to the damage done by a standard load. Results from the AASHO test indicated that the amount of pavement damage is not proportional to the axle load, but increases according to a power law. In South Africa the standard axle load is an 80 kN single axle load with a dual wheel configuration. The centres of the wheels are 350 mm apart. The standard axle load bears no relation to the permissible axle loads, but is a design standard.

Standard Axle Load

The standard axle load for South Africa is an 80 kN dual-wheel, single axle load. Any other load P may be converted to its equivalent number of standard axles (E80s) based on the damage done by load P in relation to the damage done by the standard load.
Equation (1) is used to calculate the relative damage, with a value of 4 or 4.2 for n.

Load sensitivity is generally expressed by the damage law, often called the fourth power law. The load equivalency factor (LEF) of load P is calculated using Equation (1), which is known as the power damage law, or the 4th power law. The unit of the LEF is equivalent standard axles (ESA) or E80s. The abbreviation MESA is often used for million equivalent standard axles. A load equivalency factor of 10 ESAs for load P therefore indicates that load P does damage equivalent to the damage done by 10 80 kN standard axles.

$$\frac{\text{Structural capacity for reference load}}{\text{Structural capacity for load P}} = \text{LEF} = \left(\frac{P}{80}\right)^n \qquad (1)$$

where
LEF = Load Equivalency Factor (E80)
P = Any axle load for which the load equivalency is required (kN)
80 = Reference axle load, typically 80 kN (standard axle)
n = damage exponent

The damage exponent n was originally determined as 4.2 from the AASHO Road Test, although a value of 4 is often used. Heavy Vehicle Simulator testing in South Africa has shown that depending on the pavement type, pavement balance and distress mechanism, n may vary from 2 to 6, as shown in Table 10

Table 10. Suggested Values for the Relative Damage Exponent (TRH4)

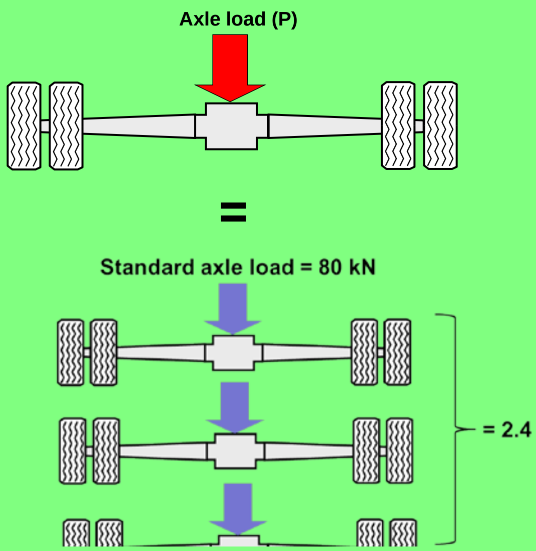
Base/Subbase Combination	Range of Values (Recommended Value)
Granular/granular	3 – 6 (4)
Granular/cemented	2 – 4 (3)
Cemented/granular	
pre-cracked	4 – 10 (5)
post-cracked	3 – 6 (5)
Cemented/cemented	
pre-cracked	3 – 6 (4 –5)
post-cracked	2 – 5 (4 – 5)
BSM/granular	2 – 6 (4)
Hot mix asphalt/cemented	2 – 5 (4)
Concrete	(4.5)

By applying the power damage law, the axle load histogram for mixed traffic can be converted to an axle load histogram consisting of only 80 kN axle loads with a frequency equal to the equivalent number of standard axles, as illustrated in Figure 17. This value for the equivalent standard axles is now used to calculate the daily equivalent traffic parameters (Section 4.2).

Equivalent Standard Axle Load Calculation: Example

A 100 kN axle load is equivalent to 2.4 single axle loads:

$$[100/80]^4 = 2.4$$



Standard Axle & Legal Axle Loads

In South Africa, an 80 kN dual wheel axle configuration constitutes a standard axle. This is below the legal axle load limit of 88 kN.

4.1.3.1 Effect of Axle Configuration on a Standard Axle

The effect of tandem, tridem and other axle configurations on the LEF is generally not considered in South Africa. The effect can be evaluated using Figure 18 (Havens et al, 1981). The standard 18 kips (80 kN) axle load applied as a single axle has a damage factor of 1. Therefore, when six 18 kips axle loads are applied (108 kips total) as single axles, the number of equivalent axle loads is 6 x 1 = 6. However, when the 108 kips are applied through a group of six axles on a truck, the damage factor is 0.85 and the number of equivalent axles thus 6 x 0.85 = 5.1. It follows that the number of equivalent axles are overestimated by 15 percent when the effect of multi-axles is not considered. This overestimation is not considered significant, especially considering the wide limits of the design traffic classes, especially at the higher end of the load spectrum.

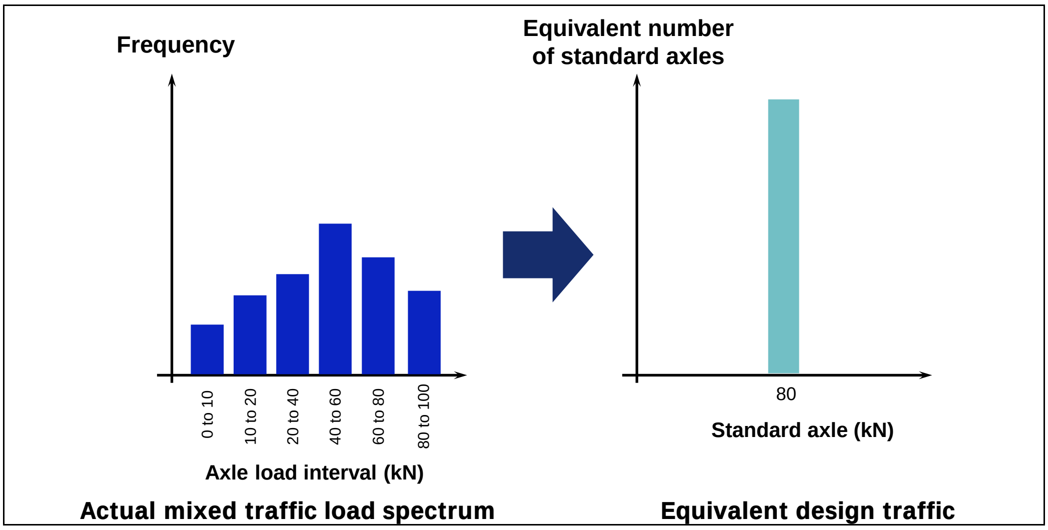


Figure 17. Representative Axle Load Histogram for Mixed Traffic by Equivalent Standard Axle Load Histogram

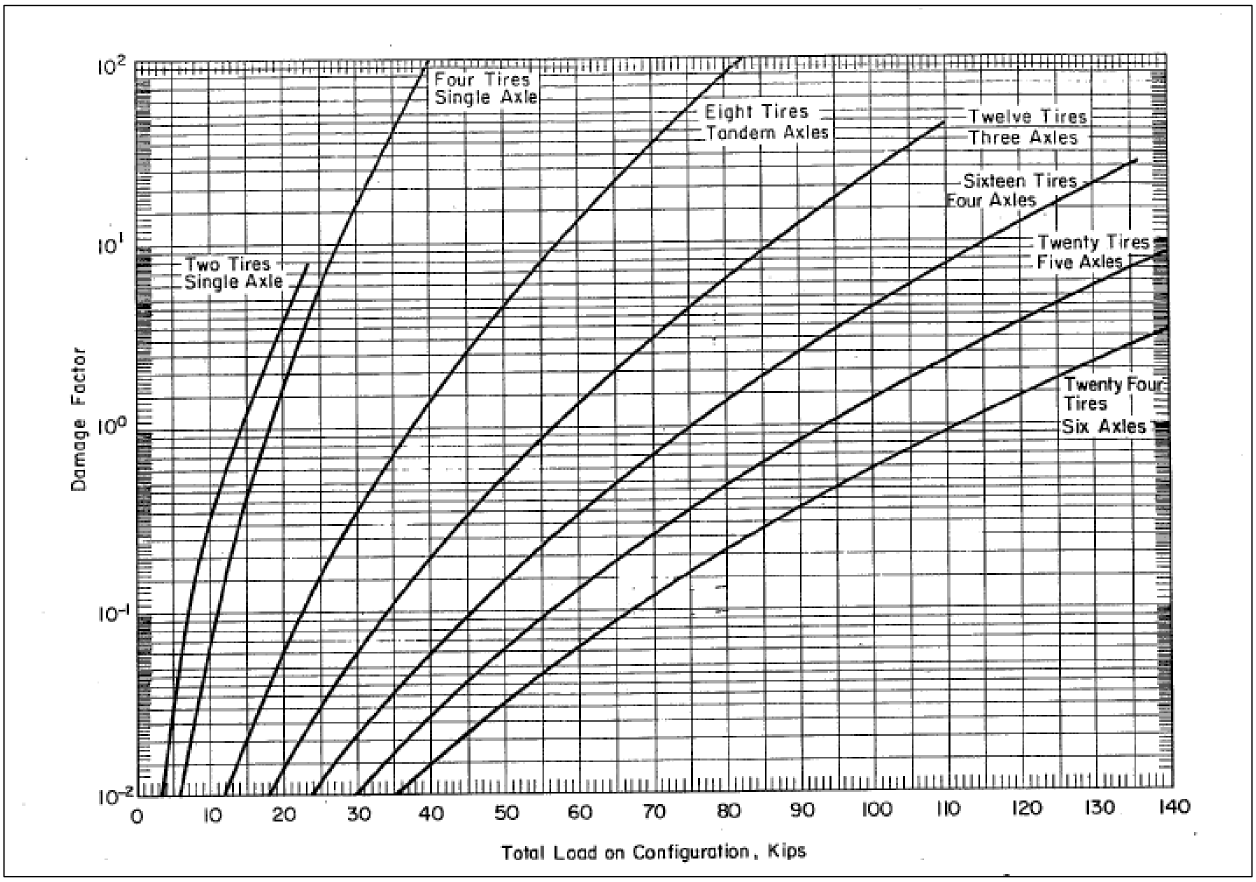


Figure 18. Damage Factors for Various Axle Configurations as a Function of Total Load

4.1.3.2 Load Equivalency of Grouped Axle Load Data

The conversion from a mixed traffic spectrum to an E80 value requires using the load equivalency formula for the lower and upper limits of each load interval, followed by the calculation of the average load equivalency. Equation (2) is used to calculate the LEF for each load interval, using the lower and upper limits of the interval.

$$LEF_i = \frac{\left(\frac{P_l}{80}\right)^n + \left(\frac{P_u}{80}\right)^n}{2} \qquad (2)$$

where LEF_i = Load Equivalency Factor (E80) for the load interval i
P_l = Lower limit of the axle load interval (kN)
P_u = Upper limit of the axle load interval (kN)
80 = Reference axle load (80 kN)
n = Damage exponent

4.1.3.3 Using the Full Traffic Spectrum in Pavement Design

Modern pavement design methods are moving away from using load equivalency and standard axle loads by incorporating the axle load histogram in the design analysis, using an incremental damage approach. The simplest form of this utilises Miner's Law with the current failure criteria, as defined in Section 7.1. A more detailed, and theoretically correct, approach requires a recursive pavement design method.

To use Miner's Law with existing failure criteria or transfer functions, the following process is followed for each layer:

- **Determine the number of ultimate load repetitions of each axle group** or vehicle type that can be accommodated, denoted $N_{1,max}$, $N_{2,max}$ ... $N_{j,max}$. For example, determine the number of fully loaded tandem axle trucks can be accommodated before the asphalt layer fails in fatigue using the asphalt fatigue transfer function (Section 7.1.1). Note that the transfer functions use the initial condition of the pavement to estimate the terminal condition.
- For each load applied of each axle group ($n_{j,i}$), **determine the proportion of damage** done by that load using Equation (3).
- **Add the damage from each axle load and axle load group**. When the total damage reaches one, the layer has reached the end of its life (or phase).

$$\text{Damage} = \sum \frac{n_{j,i}}{N_{j,max}} \leq 1 \qquad (3)$$

where $n_{j,i}$ = Number of load applications of load j
$N_{j,max}$ = Repetitions to failure of load j, as determined by transfer function

This simple method is often performed using a specific time period, such as a 3 months season. The damage for all the applications of each axle load group carried during that period is summed, and failure occurs when the damage equals, or exceeds, one.

This Miner's Law approach is linear in that the damage develops linearly between the initial and terminal condition, as those are the only condition states. Each load application of an axle group is considered to do the same amount of damage, regardless of the current condition of the pavement when the load is applied. This is a simplification of how damage develops. This method is fairly widely used, but should be used with caution as the failure criteria utilised were not developed and calibrated for such an application.

A full recursive pavement design method re-evaluates the condition of the pavement, and the changing stress state of the materials, to determine the effect of damage on the pavement system from each application of each axle load group. An overloaded truck will therefore not do the same amount of damage when applied at the beginning than at near the end of the life of the layer. The revised SAMDM (see Section 7.1) will incorporate recursive analysis techniques.

4.1.4 E80s per Heavy Vehicle (E80/HV)

For many projects, the full axle load distribution is not available for a particular project. In these cases, the concept of an E80 per heavy vehicle (E80/HV) is used. An E80/HV is a factor that converts different truck loads to an equivalent number of standard axles. The concept is illustrated in Figure 19, where the truck axle loading is first converted to an equivalent standard axle (using Equation (1) in Section 4.1.3), and summed for all the axles to obtain the E80/HV for that vehicle.

The E80/HV for each vehicle type is, however, not that useful, considering the many different vehicle types on any one route. Therefore, the average E80/HV for a network or section of road is generally used. This average is a weighted average of the E80/HV of the vehicle types and the number of the vehicles per day, and is specific to a network route or project section. The value of the average E80/HV factor gives an indication of the loading on a road, with roads with a higher number of heavy vehicles having a higher E80/HV. For example, the N3, which carries large number of heavy trucks, typically assumes an E80/HV of 3.1 whereas the N2 between Port Elizabeth and Grahamstown uses 1.5. When the term E80/HV is used, it typically refers to the weighted average of the E80/HV for the specific network. Changes to the legal axle load and the level of enforcement generally affects the E80/HV for a network.

E80/HV

An E80/HV is a factor that converts different truck loads to an equivalent number of standard axles.

When the term is used, it typically refers to the weighted average of the E80/HV for the specific project section.

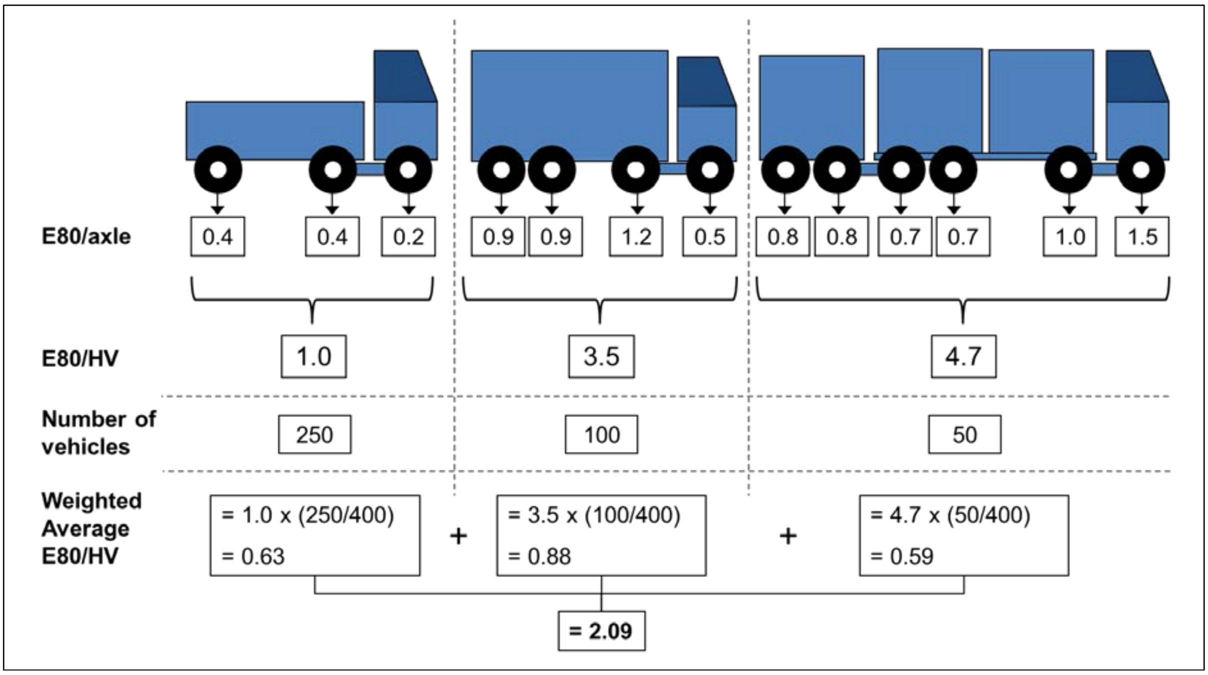


Figure 19. E80/Heavy Vehicle

4.1.5 Traffic Classes for Structural Pavement Design

TRH16 (1991) lists seven design traffic classes. However, TRH4 (1996) makes provision for ten pavement classes. It is recommended that TRH4 classes, as shown in Table 11, are used.

Table 11. TRH4 Traffic Classes

Pavement Class	Structural Capacity (million standard axles/lane)	Traffic Volume and Description of Traffic[2]	
		Approximate vpd[3]	Description
ES0.003[1]	< 0.003	< 3	Very lightly trafficked roads with very few heavy vehicles. Includes roads transitioning from gravel to paved roads and may incorporate semi-permanent and/or all weather surfacing layers.
ES0.01	0.003 – 0.01	3 – 10	
ES0.03	0.01 – 0.03	10 – 20	
ES0.1	0.03 – 0.1	20 – 75	
ES0.3	0.1 – 0.3	75 – 220	
ES1	0.3 – 1	220 – 700	Lightly trafficked roads carrying mainly cars, light delivery and agricultural vehicles with very few heavy vehicles.
ES3	1 – 3	> 700	Medium traffic volume roads with a few heavy vehicles.
ES10	3 – 10	> 700[4]	High traffic volume roads or roads with many heavy vehicles.
ES30	10 – 30	> 2200[4]	Very high traffic volume roads with a high proportion of fully laden heavy vehicles.
ES100	30 – 100	> 6500[4]	

Notes

1. ES = Equivalent Standard Axle (80 kN) class.
2. Traffic demand converted to Equivalent 80 kN axles.
3. vpd = vehicles per lane per day. The approximate vpd per lane for ES0.003 is estimated from the structural capacity and thus pavement class based on the following: 10% heavy vehicles, 1.2 E80s per heavy vehicle, 4% growth rate in E80s over a design period of 20 years.
4. For ES10 to ES100 the vpd is total per direction with 20% heavy vehicles and 2 E80s per heavy vehicle.

4.2 Methods of Traffic Measurement and Classification

4.2.1 Traffic Measurement

Traffic counts and classification are done manually or are automated using traffic counting stations installed on the road. Traffic counting is performed on a special or project basis (short term), or on a temporary (medium term) to permanent (long term) basis by road authorities, as part of their road management system data collection strategy. Whilst permanent stations provide a continuous traffic record from one year to the next, temporary stations are used on a sampling or periodic basis to collect data over a minimum specified time period.

Depending on the application, different equipment and combinations of equipment or systems are used to provide the required data in the most cost-effective way. Installation, implementation and maintenance of these systems are normally done by specialist service providers. The information provided here is therefore introductory.

Traffic counting and classification is performed at various levels of detail. Primary sensors are the essential sensors to detect vehicle presence and are also used for basic vehicle classification. By adding secondary sensors, more information is recorded and a more detailed classification undertaken.

Pneumatic tubes, induction loops, and piezo-electric sensors are typical primary sensors. Induction loops are the most common primary sensor used and detect the presence of a vehicle by electromagnetic induction. Inductive profiles are used to distinguish light from heavy vehicles using a single loop. A dual loop system provides inductive profiles plus vehicle length, and can therefore be used to classify vehicles as short, medium and long.

Piezoelectric sensors, or axle sensors, can be used as traffic counters and vehicle classifiers. They are normally used as secondary sensors together with induction loops. Classification is based on axle spacing profiles, therefore the number of axles per vehicle.

Weight measurement and classification is done through weigh-in-motion (WIM) systems. These devices measure the dynamic axle mass of a moving vehicle to estimate the corresponding static mass. WIM systems can be divided into high speed weigh-in-motion (HSWIM) and low speed weigh-in-motion (LSWIM). The function of these systems differs in that HSWIM attempts to quantify actual loads applied by moving vehicles and is an effective way to measure the entire spectrum, but at lower accuracy. LSWIM can be used to measure samples or pre-screened vehicles at a higher accuracy, and is, therefore, used for law enforcement.

Different WIM technologies exist. The most common types of mass sensors are piezoelectric cables, load cells, and bending plates. A complete WIM installation typically consists of at least one mass sensor and two induction loops. A typical WIM setup is shown in Figure 20. The mass sensors are briefly described below, in order of increasing life expectancy, accuracy and cost.

- **Piezoelectric sensors:** These strips essentially consist of copper wire surrounded by piezoelectric material, which produce a charge when subjected to pressure. The waveform current is proportional to the axle mass.
- **Bending plates:** A steel plate is instrumented with strain gauges at critical positions to measure strain induced by a vehicle. The strain is analysed to determine axle load.
- **Load cells:** These sensors typically consist of one or more steel platforms, beams or plates that are simply supported by a load cell on each corner to measure the applied force.

Site selection and location characteristics are fundamental to the performance of WIM systems and the selection of an appropriate site is based on experience and intuition. Apart from pavement characteristics (installation site and approach), vehicle and climatic characteristics impact on the reliability and accuracy of measurements. Furthermore, to determine the equivalent static axle mass of a moving vehicle, the system output needs to be properly calibrated. Whilst calibration requirements vary among different WIM systems, routine calibration is needed.

Figure 20. Typical WIM setup

4.2.2 Vehicle Classification

The appropriate vehicle classification system depends on the traffic data required and the capabilities of the traffic monitoring equipment. Heavy axle loads, associated with heavy vehicles, do most of the damage on pavements. For pavement design, traffic should therefore be split between light and heavy vehicles.

A number of heavy vehicle classification systems are available. The common systems used in South Africa are summarised in Table 12. For classification systems using the number of axles per vehicle, traffic measurements are made using an axle load sensor in combination with loop sensors (see Section 4.2)

The extended heavy vehicle classification system is based on the length of the vehicle rather than the number of axles. The definition of the heavy vehicle classes is:

- **Short heavy** vehicle: length < 10.8 metres
- **Medium heavy** vehicle: length between 10.8 and 16.8 metres
- **Long heavy** vehicle: length > 16.8 metres

The Extended Light/Heavy vehicle classification system is commonly used for estimating design traffic.

Table 12. Heavy Vehicle Classification Systems Commonly Used in South Africa

<table>
<tr><th>Classification System</th><th>Light/Heavy</th><th>Extended Light/Heavy</th><th>SANRAL</th><th>Toll</th></tr>
<tr><td>Equipment requirement[1]</td><td>Single loop</td><td>Dual loop</td><td>Dual loop with axle load sensor</td><td>Dual loop with axle load sensor</td></tr>
<tr><td rowspan="17">Vehicle classes</td><td rowspan="3">Light vehicles</td><td rowspan="3">Light vehicles</td><td>Motorcycle</td><td rowspan="3">Toll class 1 (light vehicles)</td></tr>
<tr><td>Light motor vehicle</td></tr>
<tr><td>Light motor vehicle and trailer</td></tr>
<tr><td rowspan="14">Heavy vehicles</td><td rowspan="5">Short heavy vehicles</td><td>Two axle bus</td><td rowspan="2">Toll class 2 (2 axle heavy vehicles)</td></tr>
<tr><td>Two axle single unit</td></tr>
<tr><td>Buses with maximum 4 axles (including trailer)</td><td rowspan="4">Toll class 3 (3 and 4 axle heavy vehicles)</td></tr>
<tr><td>Single unit with maximum 4 axles (including trailer)</td></tr>
<tr><td>Two axle single unit with or without trailer (maximum 4 axles)</td></tr>
<tr><td rowspan="5">Medium heavy vehicles</td><td>Single trailer (maximum 4 axles)</td></tr>
<tr><td>Buses with 5 or more axles</td><td rowspan="8">Toll class 4 (5 and more axles heavy vehicles)</td></tr>
<tr><td>Single unit and trailer with more than 4 axles</td></tr>
<tr><td>Five axle single trailer</td></tr>
<tr><td>Six axle single trailer</td></tr>
<tr><td rowspan="4">Long heavy vehicles</td><td>Five axle multi-trailer</td></tr>
<tr><td>Six axle multi-trailer</td></tr>
<tr><td>Seven axle multi-trailer</td></tr>
<tr><td>Eight or more axle multi-trailer</td></tr>
</table>

Notes
1. See Section 4.2 for descriptions of the equipment.

4.3 Daily Equivalent Traffic Parameters

4.3.1 Average Daily Traffic (ADT)

Traffic is most often denoted by the Average Daily Traffic (ADT). The ADT represents the number of vehicles on a road in both directions and on all lanes, and includes heavy and light vehicles.

4.3.2 Average Annual Daily Traffic (AADT)

The AADT is calculated by extrapolating the ADT to the full year, and dividing by 365 days. The heavy vehicles are often given as a percentage of the total ADT or AADT.

4.3.3 Average Daily Equivalent Traffic (ADE)

The average daily equivalent traffic (ADE) is the average E80s per lane per day for the duration of the survey period. The ADE can only be calculated directly from data from permanent WIM stations. The ADE is calculated using Equation (4) from the total E80s for the survey period, divided by the survey duration and number of lanes on which the survey was done, if not already accounted for, or lane distribution factors are not available. The ADE is valid for the period of the survey alone. Where permanent WIM station data are not available, the ADE is estimated from the ADT and AADT, considering the directional splits and lane distributions.

Daily Equivalent Traffic Parameters

- **ADT** is the average daily traffic, and includes all vehicles (light and heavy) travelling in all directions.
- **AADT** is the average annual daily traffic and includes all vehicles and all directions. The total year's traffic is divided by 365 days. Heavy vehicles are often given as a percentage of the AADT or ADT.
- **ADE** is the average E80s per lane per day over the duration of the survey period, calculated using Equation (4).
- **AADE** is the total E80s per lane for one year, divided by 365 days. ADE is extrapolated over the year to get AADE. Care must be taken to ensure the ADE survey period is representative of the full year's traffic.

If axle load histogram available:

$$ADE = \frac{\sum_{i=1}^{k} n_i \text{ x } LEF_i}{\text{duration x lanes}}$$

If axle load histogram is not available and E80/HV values are used:

$$ADE = \frac{E80/HV \text{ x } HVV}{\text{duration x lanes}} \quad (4)$$

where n_i = Axle count in bin i (i = 1 to k)
LEF_i = Load equivalency factor of bin i (from Equations 4 or 5)
lanes = Number of lanes. Only included if the lane distributions have not already been taken into account, and lanes carry equal E80s.
E80/HV = E80 per heavy vehicle
HVV = Heavy vehicle volume for survey duration

4.3.4 Annual Average Daily Equivalent Traffic (AADE)

The annual average daily equivalent traffic (AADE) is the total E80s per lane for one year, divided by 365 days. The AADE is valid for the year during which the survey was done. The AADE is calculated from the ADE.

If the survey period is fully representative of the traffic conditions for the full year, then the ADE equals the AADE. However, if the survey period is not representative of the traffic conditions for the full year, the ADE needs to be carefully converted to the AADE. Figure 21 illustrates two different ADEs determined from different surveys on the same route. In situations like this, the ADE needs to be adjusted to account for the variations, as discussed in Section 4.4.

Definitions of Traffic Parameters

Different road authorities can use different definitions for the traffic parameters. Before embarking on the traffic calculations, check that the definitions you assume are the same as your client's.

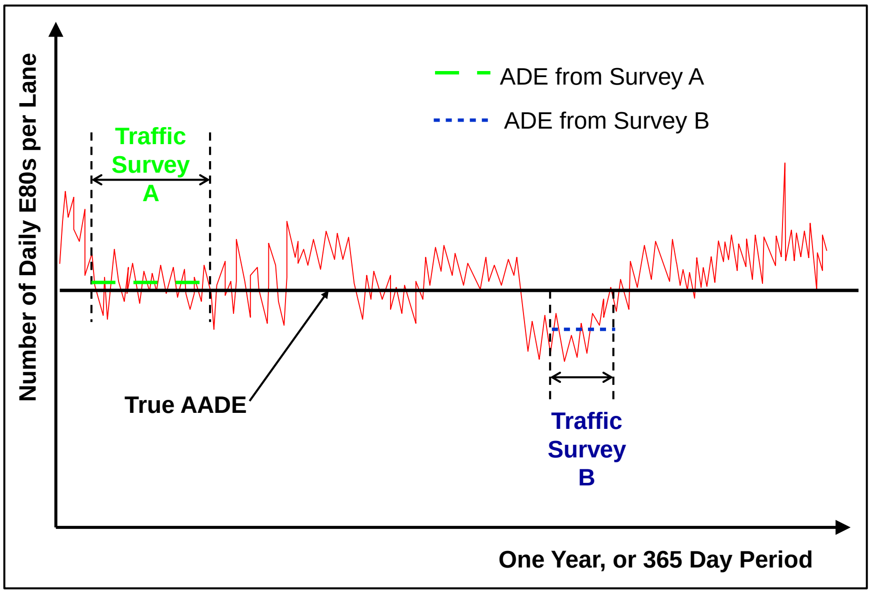


Figure 21. Potential Deviation of the ADE from the AADE

4.4 Traffic Loading Variation

Variations in traffic loading occur over the short or long-term:

- **Short-term variations** occur within a year. Seasonal variations are recurring periodic patterns occurring each year, and are usually related to weather or agricultural activities, such as planting and harvesting. Periods of exceptionally low traffic activity may also occur when economic activity is halted at a specific time of the year. Corrections for seasonal variation and exceptional periods are made using adjustment factors, but should only be applied if they are expected to occur each year for the duration of the design life of the facility.

Traffic Surveys

Great care must be taken to ensure that traffic during the survey period is representative of the traffic during the year.

- **Long-term variations** include changes in traffic volumes and loading of vehicles over a period longer than a year. Traffic volume changes are affected by a host of factors, for example, population growth or decline, establishment of new industries such as mines and quarries, long-term construction programmes and economic recessions. Traffic volumes and vehicle loading are also influenced by changes in the way the preference shifts between different transportation modes, particularly road, rail and pipeline.

Adjustments for Long-Term Variations

Adjustments for long-term variations are complex. Adjustment factors should be developed over at least a five year period.

Three main factors influence the equivalent standard axles per heavy vehicle:

- Improved **vehicle design** leads to changes in the type and size of heavy vehicles.
- **Efficient management** of road transport, i.e., fewer vehicles running empty and better utilisation of vehicle capacity.
- Level of **overloading control.**

Changes in Government regulations and policies, such as changes in the legal axle load or gross vehicle mass of vehicles, and deregulation of road transport have a significant effect on traffic loading. The degree to which law enforcement is applied to overloading also has a significant effect on the loading per vehicle.

In South Africa, a general decrease in the number of short heavy vehicles has been observed from the late 1980s to 2003, while the number of long heavy vehicles has increased (De Bruin and Jordaan, 2004).

Provision must be made for long-term changes in traffic volumes and vehicle loading by:

- **Traffic volumes:** Apply a growth rate to the heavy vehicle traffic. Road geometric capacity restrictions are essential to contain volume growth to realistic levels.
- **Loading per vehicle:** Apply an E80 per heavy vehicle growth rate in addition to the volume growth rate. Care should, however, be exercised not to grow E80/HV to practically impossible levels. Guidelines on the E80/HV growth rates are given in Table 18.

Adjustments for long-term variations are complex. Care should be taken to not overestimate the long-term changes in the traffic volume and vehicle loading based on cyclic fluctuations lasting several years. These are usually related to economic activity of the country. It is not advisable to correct for these cyclic variations as their average impact corresponds with the trend. Consequently, any adjustment factors should be developed over at least a five year period.

4.4.1 Lane Distribution

If traffic data is collected and processed on a lane by lane basis, a lane distribution is not required for multilane facilities. However, if the ADE is calculated from traffic counts, or from the results of transportation planning models, a lane distribution is required. The distribution between lanes may vary along the length of a road depending on geometric design, climbing lanes and ramps. The distribution of traffic volume and loading is also normally not the same. Suggested lane distribution factors are given in Table 13. The design ADE for each lane is calculated by multiplying the total ADE per direction by the lane distribution factor, often termed B_e.

Table 13. Recommended E80 (ADE) Lane Distribution Factors (B_e)

Total Number of Lanes in Both Directions	E80 Lane Distribution Factor				
	Surfaced Slow Shoulder	Lane 1[1]	Lane 2	Lane 3	Surfaced Fast Shoulder[2]
2	1.00	1.00	–	–	N/A
4	0.95	0.95	0.30	–	0.30
6	0.70	0.70	0.60	0.25	0.25

Notes
1. Lane 1 is the outer or slow lane
2. For dual-carriageway roads

4.4.2 Adjustment for Short-Term Variation

If a traffic survey is conducted over a long enough period during a year, the influence of weekends, public holidays and peak periods are averaged out, and the ADE provides a direct indication of the AADE. However, if the ADE is determined from a relatively short survey, then the influence of night time, weekends and unusual periods is large, and adjustments are needed for the following:

- Variation between **day and night traffic**, where data was only collected for a 12 hour period.
- Variation between **weekday and weekend** traffic.
- Variation between periods of exceptionally low traffic, such as **holidays** when businesses, factories and the construction industry are closed, and periods of normal traffic.
- Variation between periods of exceptionally high traffic, such as **harvesting season** in agricultural areas, and out-of-season periods.

The ADE is corrected with adjustment factors. Where applicable, the adjustment is done by calculating weighted average values for the AADE, considering the duration of the normal and exceptional traffic in a year. The adjustments must only be applied if:

- Variations **occur year after year** on the specific road.
- Traffic survey results **do not represent the conditions** for a full year.
- The adjustments have a **significant impact,** resulting in serious error if the adjustment is not applied.

4.4.2.1 Variations between Day and Night Traffic

If a twelve hour traffic survey was done, typically between 06:00 and 18:00, the counts are adjusted for night-time traffic. Typically about 70% of the daily traffic loading uses the road during the 12 hour period. However, this proportion varies between 40% for rural routes and 90% for urban freeways (TRH16, 1991). Projections of the 12 hour survey to 24 hour data should done using local knowledge, and if possible, by using classification counts over a 24 hour period. Classification counts usually provide the number of vehicles of each type for each hour of the day and night. Assuming that the vehicles are similarly loaded during the day and night, the survey data can be extended to give the ADE.

4.4.2.2 Variations between Weekday and Weekend Traffic

Traffic loading is normally lower during weekends than on weekdays. If the ADE is determined from a survey done during the 5 day work week only, then the ADE needs to be adjusted to a full 7 day week. This is done using the factor $F_{7/5}$, given in Equation (5).

$$F_{7/5} = \frac{ADE_7}{ADE_5} \tag{5}$$

where ADE_7 = ADE for a full 7 day week
ADE_5 = ADE for a 5 day work week

Unfortunately, in a design situation, the ADE_7, and therefore $F_{7/5}$, is unknown. Therefore, published values of $F_{7/5}$ are generally used. TRH16 recommends a value between 0.7 and 1, where:

- $F_{7/5}$ = 0.7: **no additional heavy vehicle traffic over a weekend**.
- $F_{7/5}$ = 1.0: **traffic volume during the weekend and work week is the same**.

Since TRH16 was published, some additional $F_{7/5}$ values have been recommended (Theyse, 2008c), and are given in Table 14.

Table 14. Recommended 5 to 7 day ADE Conversion Factors ($F_{7/5}$)

Route Description	Weekend Traffic Volume (%)		$F_{7/5}$ Factor
	Range	Recommended Value	
Routes carrying mainly agricultural traffic in rural areas and urban routes with high traffic during weekdays	14 to 16	15	0.85
Routes carrying industrial and mining traffic	13 to 19	17	0.86
Routes connecting major centres and export routes	18 to 21	19	0.89

4.4.2.3 Adjustment for Exceptional Periods

The adjustment for exceptional periods is done with the exceptional period factor, F_E. F_E is less than one. The calculation of F_E, and AADE using F_E, are shown in Equations (6), (7) and (8) in Table 15.

Table 15. Adjustment for Exceptional Periods

Adjustment Factor F_E
$F_E = \dfrac{ADE_E}{ADE_N}$ (6)
where ADE_E = ADE for exceptional days ADE_N = ADE for normal days
Traffic Survey During Normal Period
$ADE_N = ADE_7$ $ADE_E = F_E ADE_7$ $AADE = \dfrac{(D_N ADE_7 + (365 - D_N) F_E ADE_7)}{365}$ (7)
where D_N = Number of days with seasonally high traffic in a year
Traffic Survey During Exceptional Period
$ADE_E = ADE_7$ $ADE_N = \dfrac{ADE_7}{F_E}$ $AADE = \dfrac{\left(\dfrac{D_N ADE_7}{F_E} + (365 - D_N) ADE_7\right)}{365}$ (8)
where D_N = Number of normal days in a year

Based on weigh-in-motion data collected at various stations in South Africa, the only period of exceptionally low heavy vehicle traffic activity is from middle to end December. Depending on which day the Christmas and New Year holidays fall, the duration of this exceptional period varies between one and two weeks. Investigations showed that the error in ignoring this exceptional period is small for both AADE and when projected over a 20 year period. (Theyse, 2008c)

Adjustments for Exceptional Periods

In South Africa, the only period with exceptionally low traffic is over the Christmas and New Year period. Investigations showed that the error in ignoring this exceptional period is small for both AADE and when projected over a 20 year period.

4.4.2.4 Adjustment for Seasonal Variation

The adjustment for seasonal variation is done similarly to the adjustment for exceptional periods, but with the seasonal adjustment factor F_S. The calculation of F_S is given in Equation (9) in Table 16, along with the calculation of the adjusted AADE (Equations (10) and (11)). F_S is normally equal to or less than one.

The adjustments for exceptional periods and seasonal variation are done consecutively. The AADE output from the adjustment for exceptional periods is the ADE input of the sample period (ADE_7) for the seasonal variation correction.

(i) Study of the Effect of Harvesting Season

In the study investigating the effect of short-term traffic variations (Theyse, 2008c), extensive data were not available to calculate the value of the seasonal adjustment factor F_S and the duration of seasonally high traffic. A yearly set of WIM data was, however, available for one location where sugarcane is harvested. The E80/HV was 2.5 for the seasonal harvesting period, compared to 2.0 for the rest of the year. The traffic volume increased from 419 to 449 HV/lane/day during the harvesting season.

If a 7 day traffic survey is done during the non-harvesting season and the ADE is not corrected for the higher traffic volume and vehicle loading during the harvesting season, the result is a 9% underestimation of the design traffic over a 20 year period (assuming 6% traffic growth rate). Such an error may result in the road lasting for only 18 years and not the intended 20 years. Again, the difference is not dramatic, but it may be more severe for other cases. It is recommended that the potential for seasonal traffic loading be carefully investigated. If it is likely to occur, the traffic survey should be conducted in and out of season. Alternatively, if the adjustment for seasonal traffic is neglected, the survey should be done during the high season, to ensure a conservative error.

Table 16. Adjustment for Seasonal Variation

Adjustment Factor F_S

$$F_S = \frac{ADE_O}{ADE_I} \tag{9}$$

where ADE_O = Out of season ADE
ADE_I = In season ADE

In Season Traffic Survey

$$ADE_I = ADE_7$$
$$ADE_O = F_S ADE_7$$

$$AADE = \frac{(D_S ADE_7 + (365 - D_S)F_S ADE_7)}{365} \tag{10}$$

where D_S = Number of days with seasonally high traffic in a year

Out of Season Traffic Survey

$$ADE_O = ADE_7$$
$$ADE_I = \frac{F_S}{ADE_7}$$

$$AADE = \frac{\left(\frac{D_S ADE_7}{F_S}_7 + (365 - D_S)ADE_7\right)}{365} \tag{11}$$

where D_S = Number of normal days in a year

4.4.3 Adjustment for Long-Term Variation (Heavy Vehicle and E80/HV Growth Rates)

Adjustment for long-term traffic variation is probably the most likely cause of incorrect design traffic estimates. This is because it is extremely difficult to anticipate how the traffic volume and loading per heavy vehicle change with time. In general, a growth in heavy vehicle volume has been observed in South Africa (De Bruin and Jordaan, 2004 and Theyse, 2008c). The general trends show:

- Number of **long heavy vehicles increasing.**
- Number of **short and medium heavy vehicles increasing at a much lower rate**, or remaining constant with time.
- End result is that **the total number of heavy vehicles is increasing with time.**

There are always exceptions to the general trend. Cases were found where the heavy vehicle volumes remained fairly constant over a period of a few years. These stations are, however, in areas where the economic activity in the area has stabilised, such as fully developed agricultural and forestry areas.

The growth rate for heavy vehicle volumes may be derived from past data using Equation (12), if data are available for a sufficient duration of time, at least 5 years. Typical heavy vehicle volume growth rates were calculated from historical data collected at selected permanent traffic observation stations in South Africa and the results are summarised in Table 17 (De Bruin and Jordaan, 2004). The period for which data was available varied from 4 to 9 years. Although the data in the table only cover a very limited set of conditions, it does provide an indication of the extreme values that may be expected for traffic growth in South Africa.

If the trend in traffic volume growth is difficult to anticipate, the trend in heavy vehicle loading is almost impossible to anticipate. Generally, because there is a shift toward using long heavy vehicles, the average number of E80/HV should increase. However, the enforcement of legal axle load limits (or the lack thereof) has a significant effect on the long-term trend for the E80/HV, combined with the starting E80/HV:

- If the **E80/HV starts from a low base** on a specific route and there is a **lack of overload control**, the E80/HV increases with time.
- If there is a **lack of overload control** on a route, the **E80/HV starts from a high base**, but may even reduce if effective policing is applied.

$$r = \left[\left(\frac{HVV_n}{HVV_1}\right)^{1/n} - 1\right] \qquad (12)$$

where
r = Heavy vehicle volume growth rate (%)
HVV_n = Heavy vehicle volume at year n
HVV_1 = Heavy vehicle volume at year 1

Table 17. Recommended Traffic Growth Rates Based on Historical Data for South Africa

Route Description and Economic Activity	Heavy Vehicle Volume Growth Rates (%)	
	Range (pre 2007)	Typical Long-term Values
Major routes connecting centres of major economic development	9 to 19	6 to 10
Routes connecting centres of moderate economic development	6 to 8	4 to 7
Routes in areas of stagnant economic development	0.5 to 1.5	0 to 2

TRH16 uses a technique developed by Bosman (1989 and 2004) to distinguish between routes with different heavy vehicle compositions. Routes are classified according to the percentage of 2 axle heavy vehicles in the total HV traffic. Although the extended vehicle classification does not separate out 2 axle heavy vehicles, the short vehicle class should consist mostly of 1 and 2 axle heavy vehicles. Using the percentage of short heavy vehicles (SHV) as an indicator of heavy vehicle composition, recommended E80/HV values are given in Table 18. These values should only be used for new design, or, where there are good reasons for not conducting an axle load survey on an existing road. The values are based on a damage law exponent of 4.2 and could be higher for more load sensitive pavements (see Section 4.1.3).

Potential E80/HV growth rates are shown in Table 18. These growth rates only apply if the freight movement demand increases on a route with an existing low or moderate freight movement demand, under normal overloading control levels. The E80/HV at the end of the analysis period must not exceed realistic levels when using these growth rates. Under strict overloading control, the E80/HV is limited by the legal axle loads and no growth occurs. When the freight movement demand is high, with normal overloading control, the E80/HV is limited by the vehicle capacity. Increases in demand are accommodated by increasing heavy vehicle volumes.

Table 18. Recommended E80/HV Values and Growth Rates for Heavy Vehicle Classes for Design Purposes

Overloading control by law enforcement	Normal			Strict
Freight movement demand	Low (SHV[1] > 45%)	Moderate (SHV = 20 – 45%)	High (SHV < 20%)	All levels
Heavy Vehicle Class	**Recommended E80/HV Values**			
Short heavy	0.5 – 1.5			0.4 – 0.8
Medium heavy	1.0 – 4.0	2.0 – 5.0		1.0 – 3.0
Long heavy	1.5 – 4.5	3.5 – 6.0	4.5 – 7.0	3.0 – 5.0
All heavy	0.8 – 2.8	2.0 – 4.5	3.0 – 5.0	2.0 – 3.0
Heavy Vehicle Class	**E80/HV Growth Rates**			
Short heavy	0		0	0
Medium heavy	3	0		
Long heavy	5	2		
All heavy	6	2		

Notes
1. SHV = short heavy vehicle, 1 and 2 axle heavy vehicles

The heavy vehicle volume growth rate (Table 17) and the E80/HV growth rate (Table 18) are combined into a single growth rate for the AADE using Equation (13).

$$i = [(1 + h)(1 + v) - 1] \tag{13}$$

where i = yearly AADE growth rate as a decimal fraction
h = yearly heavy vehicle volume growth rate as a decimal fraction
v = yearly E80/HV growth rate as a decimal fraction

4.5 Sequence of Events and Cumulative Design Traffic

The chronological sequence of events is important when determining the design traffic. The following three dates are necessary:

- Date of the **traffic investigation**
- Date of **design**
- Date of **opening**

The process of projecting the AADE from the survey date to the date of design, followed by the accumulation of the equivalent traffic for the duration of the structural analysis period is illustrated in Figure 22.

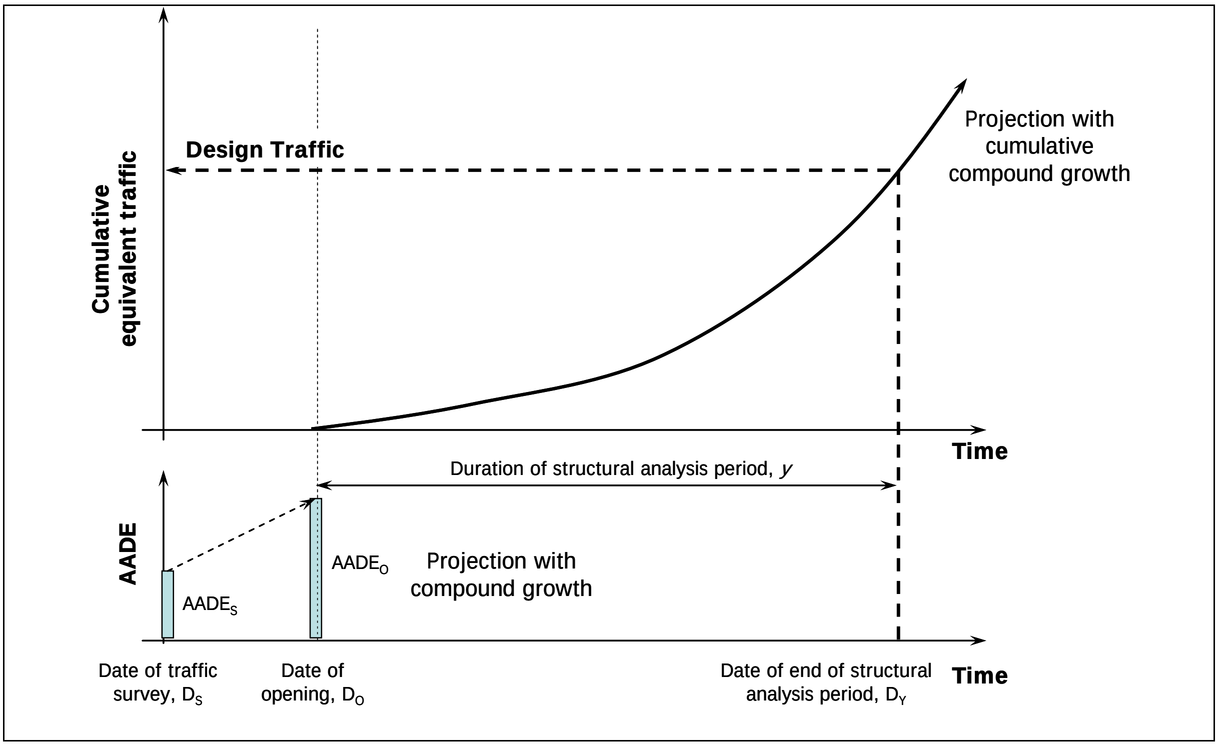


Figure 22. Sequence of Events

The date of the traffic survey and design may coincide, but if historical traffic data is used for the design, these dates differ. The AADE determined from the traffic survey data is projected to the AADE at the date of opening, using a compound growth calculation given by Equation (14).

$$AADE_O = AADE_S(1 + i)^n \tag{14}$$

where $AADE_O$ = AADE at the time of opening the facility
$AADE_S$ = AADE at the time of the traffic investigation
i = yearly AADE growth rate as a decimal fraction
n = Time (years) from the date of the traffic investigation to the date of opening

Once the facility is opened to traffic, the cumulative equivalent traffic (ET) is calculated for the duration of the structural design period using the cumulative compound growth equation (Equation (15) in Table 19). The cumulative equivalent traffic at the end of the structural design period is the design traffic or traffic demand that must be provided by the structural pavement design.

Table 19. Equivalent Traffic Calculation for New and Rehabilitation Design

Cumulative Equivalent Traffic: $$ET = 365 \text{ x } AADE_O(1+i)\frac{[(1+i)^y - 1]}{i} \qquad (15)$$

where ET = Cumulative equivalent traffic
i = Yearly AADE growth rate as a decimal fraction
AADE = AADE at the beginning of the calculation period
y = Time in years for the calculation period since the opening of the facility, the structural design period for the design traffic determination

Inputs			
		AADE	**Years (y)**
New Design	**Design (future) traffic**	$AADE_O$ at time of opening facility.	Years since the opening of the facility, the structural design period.
Rehabilitation Design	**Past traffic**	$AADE_O^I$ at the date of the past opening of the existing road. Calculated using Equation 13 and the number of years from the survey date to the opening date, D_I (a negative number).	Age of the existing road in years from the past opening (D_I) to the date of the investigation (D_{INV}).
	Design (future) traffic	$AADE_O$ at time of opening the rehabilitated road. Calculated using Equation 13 from AADE at the survey time and the time to opening.	Structural design period (sdp) in years after the opening of the rehabilitated road.

Rehabilitation design requires that the past cumulative equivalent traffic and future anticipated traffic are estimated. Figure 23 illustrates the time-based relationship between the events used in the traffic calculation process for rehabilitation design. The formula for calculating the past traffic is also used for projecting the future traffic from the date of the traffic survey, Equation (15) with different input values. The input variables for Equation (15) for both future and past equivalent traffic are given in Table 19.

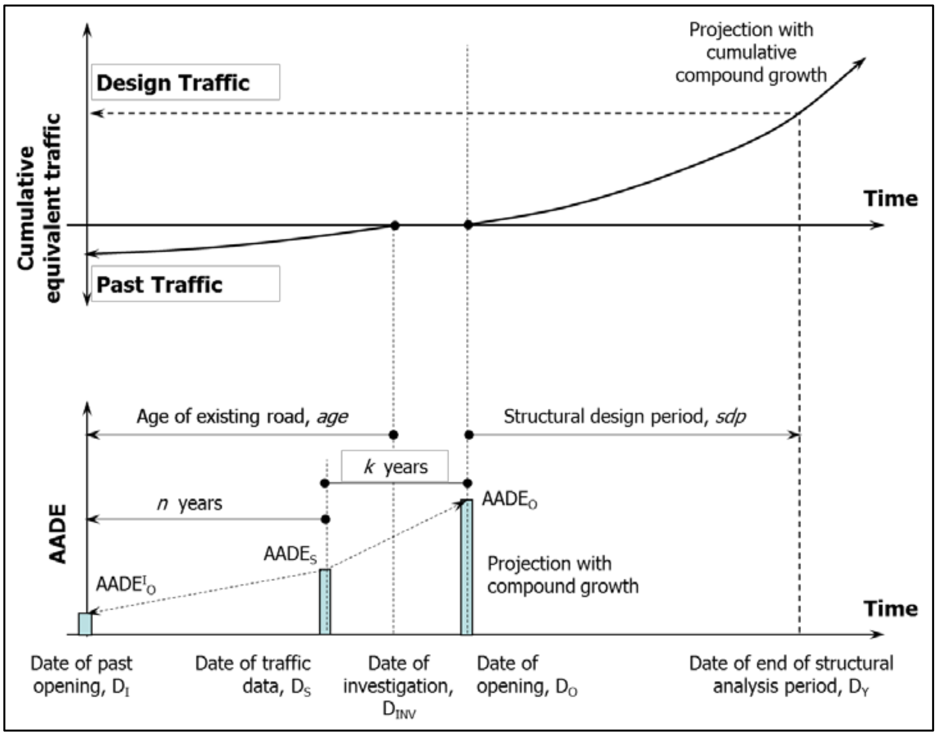


Figure 23. Chronological Events for Rehabilitation Design Traffic Calculations

4.6 Traffic Investigations for Pavement Design

Traffic data are required for network level planning and management, and project level pavement design. This guideline focuses on project level analyses. One-day traffic counts, often done at municipal level for planning purposes, may be tempting for project level design, but are not recommended. An incorrect design traffic estimate results in the same design risk as a pavement structure of inadequate structural capacity. The allocation of funds to pavement and material investigations often enjoy preference to spending funds on traffic surveys. Funds spent on a project specific traffic survey are, however, well worth the investment for reducing the design risk.

One-day Traffic Counts

One-day traffic counts, often done at municipal level for planning purposes, may be tempting for project level design, but are not recommended.

Four traffic parameters have a dominant effect on the accuracy of the design traffic estimate:

- **Daily heavy vehicle volume** per lane for the base year
- Heavy vehicle volume **growth rate**
- **E80/HV** for the base year
- E80/HV **growth rate**

TRH16 (1991) provides for a number of methods of estimating or measuring these parameters:

- Published results or **results from other projects** related to the design project in terms of:
  - Similar loading of heavy vehicles
  - Linked to the heavy vehicle classification system
  - Based on a road stratification system
- Transportation **planning models**
- Estimations based on project specific **visual observations**
- Project specific **traffic surveys**

Road Stratification System

A road stratification system is used to classify roads into different strata, with each stratum essentially having the same traffic characteristics. Sampling can then be done by measuring the traffic characteristics of a few roads in each strata, and applying the measured characteristics to all the roads in that strata.

The level of effort and cost associated with these methods increases from the use of known results to the project specific surveys, but so does the value of the results obtained. The application of these techniques is linked to the importance of the road. Table 20 recommends the minimum level of investigation based on the road category.

Table 20. Recommended Traffic Investigation Levels for Road Categories

Road Category	Traffic Parameter			
	Base Year HV Volume	HV Volume Growth Rate	Base Year E80/HV	E80/HV Growth Rate
A	Traffic surveys[1]	Transportation models	Traffic surveys	Published results
B	Traffic surveys	Transportation models Published results	Traffic surveys	Published results
C	Visual observation[2]	Published results	Visual observation	Published results
D	Published results[3]	Published results	Published results	Published results

Notes

1. Project specific traffic surveys
2. Project specific visual observation and tabulated values
3. Published results, or results from other projects with similar traffic characteristics

Justification for Traffic Surveys

Given the relatively low cost of traffic surveys, compared to the construction and rehabilitation cost, it is highly recommended that project specific traffic surveys are done as part of rehabilitation investigations.

For a specific project, the necessary data to derive the base year HV volume, E80/HV, AADE growth rate and E80/HV growth rate are seldom available. If historical axle load data and traffic volume data are available for a route, these are used to derive the necessary input. Lacking such historical data, a project specific traffic survey should be done for Category A and B roads. However, given the once-off nature of the survey it is not possible to derive historical trends and estimate future growth rates. A stratification system, or results from other projects with similar traffic characteristics, needs to be used to derive growth parameters. Lacking this type of information, it is recommended that the historical growth rates listed in Table 17 are used as a guide.

The base year HV volume and E80/HV are easily derived from project specific traffic surveys. Given the relatively low cost of traffic surveys compared to the construction and rehabilitation cost, it is highly recommended that project specific traffic surveys are done as part of rehabilitation investigations. Detailed project specific traffic surveys are not necessary for Category C and D projects, and replaced with a visual observation of the traffic and vehicle loading characteristics. For Categories A and B, it is recommended that either a 7 day count with limited static weighing, or a full 7 day axle load survey is done. Figure 24, from the SADC Low-volume Sealed Roads Guideline (2003) Guideline, shows the effect of the count duration on the error in the traffic estimates.

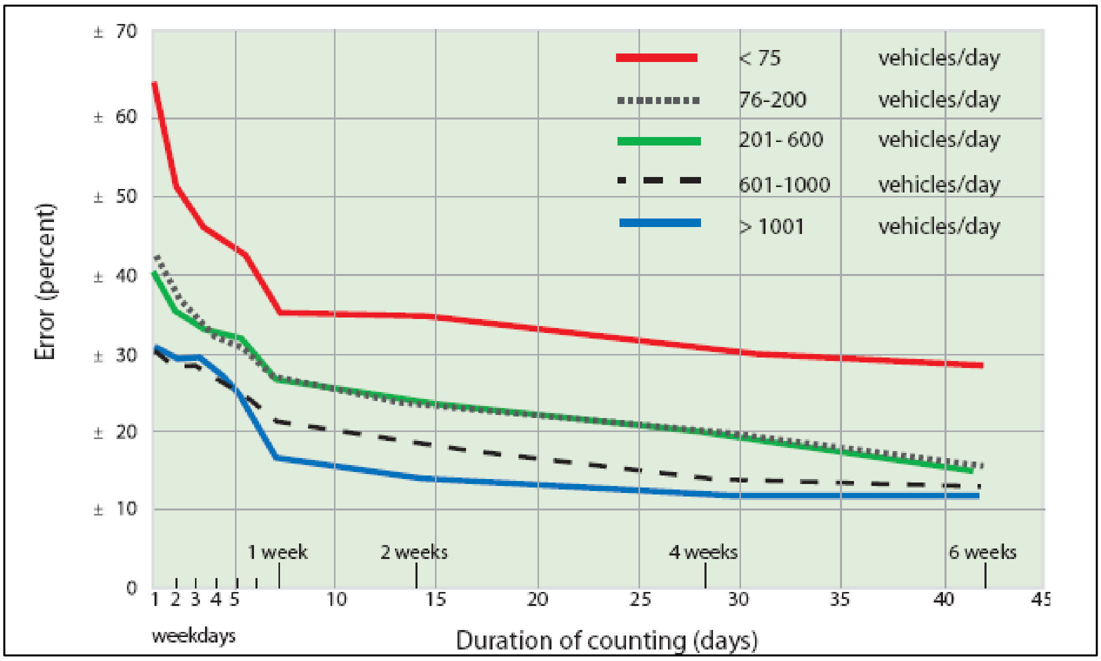


Figure 24. Effect of Counting Duration on Accuracy of Results

A particularly difficult problem is the design of a green fields project, where traffic data cannot be collected. In such cases, only published results or transportation planning models can be used. The preferred option is a transportation planning model, done by a transportation engineer.

4.6.1 Published Results

When using published results, the traffic conditions must be the same. To check this, compare the loading per heavy vehicle, the type/classes of heavy vehicles operating on the road, or using a stratification system.

When published results or those measured on other roads are used, TRH16 warns that significant errors in the design traffic estimate can result. This technique should therefore only be applied to Category C and D roads carrying little traffic (less than ES0.3). In such situations, a basic traffic count significantly enhances the design traffic estimation.

A stratification system classifies routes based on the function of the route, in addition to the characteristic of the heavy vehicles. A simple system considers the proportion of 2-axle heavy vehicles in the total heavy vehicle traffic spectrum as an indicator of the type of route (Bosman, 1989 and 2004). TRH16 lists the E80/HV based on the original work by Bosman. However, given the influence of the 1996 increase in legal axle load limits and the influence of the level of overloading control, those E80/HV values may be too low. It is therefore recommended that the E80/HV values listed in Table 18 for the short-medium-long heavy vehicle classification, considering the level of overloading control and the proportion of short heavy vehicles, are used. Because of the general lack of E80/HV growth data it is recommended that the growth rates in Table 18 are also used. In general, TRH16 is outdated, and needs updating.

TRH16 E80/HV Values

The TRH16 (1991) recommended values for the E80/HV and the growth rate were determined before the 1996 increase in the legal axle load, and do not consider the level of overloading control.
It is therefore recommended that the values given in Table 18 are used for E80/HV and the growth rates.

4.6.1.1 Comprehensive Traffic Observations (CTO) Stations

SANRAL have installed a number of Comprehensive Traffic Observation (CTO) stations on the national road network. Traffic is monitored for varying periods of time at these stations, but counts as long as one year on certain important routes are not uncommon.

The information available from the CTO stations includes calculation of the average daily E80s for the particular count. This information can be used to calculate the AADE. The information can also be used to calculate E80 and E80/HV growth rates if historic counts from the same station are available. An example of the output obtained from a station at Kinkelbos on the N2 in the Eastern Cape is shown in Figure 25.

4.6.2 Transportation Planning Models

Transportation planning models predict traffic patterns in terms of traffic distribution, growth, and future volumes. Various inputs are used, such as economic activity and geographical zones. The models are usually based on an urban modelling procedure, which looks at the region as a number of zones (van Zyl, 1986). Traffic is applied to the zones and the number of trips generated and attracted to each zone is modelled. The trips are assigned to each link of the network to obtain the traffic flows. This modelling technique is a powerful aid for judging the future traffic on existing links and, where possible, should be used in conjunction with axle load surveys.

4.6.3 Project Specific Visual Observation

The manual visual observation technique is simple and inexpensive, and gives acceptable results if a high level of accuracy is not required. The technique should only be applied to Category C and D roads with design traffic less than 1 million equivalent standard axles.

The method is based on an assessment of the loading of the heavy vehicles. The recommendations in TRH16 regarding the E80 loading per axle are repeated in Table 21 and are considered adequate. Using these values, the E80 for predominantly fully laden 7 and 8 axle vehicles is 6.3 and 7.2 E80/HV. These agree well with the values in Table 18 for long heavy vehicles under normal overloading control. TRH16 provides a template for recording the data during the visual observation survey.

Table 21. Recommended E80/Axle Loading for Visual Observation Technique

Description of Heavy Vehicle Loading	Percent of Vehicles		Axle Load Factors (E80/axle)
	Fully Laden (%)	Empty or Partially Laden (%)	
Predominantly lightly laden vehicles	< 35	> 45	0.3
Fully laden, partially laden and empty vehicles	40 – 45	34 – 45	0.5
Fully and partially laden vehicles	60 – 75	< 30	0.7
Predominantly fully laden vehicles		> 70	0.9

4.6.4 Project Specific Traffic Surveys

The main purpose of a traffic survey is to obtain estimates of the base year HV volume and E80/HV. A number of survey options are possible, each requiring different levels of effort and cost, and yielding different levels of detail.

4.6.4.1 Manual or Electronic Counts with Limited Static Weighing

This technique is applicable to Category C and B roads, when the traffic volumes are relatively low, i.e., less than 300 heavy vehicles per day. The traffic volume from electronic 7 day surveys is combined with limited static weighing of selected vehicles to determine the E80/HV. It must be determined whether the survey was done during seasonally high traffic or exceptionally low traffic. The static weighing is done using portable scales which should be placed in a safe, but accessible location. The static weighing process is limited to daylight hours for safety reasons, but should be done for at least a 12 hour period from 06h00 to 18h00. The team doing the survey must ensure the random selection of the heavy vehicles for static weighing, as there is normally a tendency to target the heavily laden vehicles. The methods of weighing are discussed in Section 4.2.

3076 **Kinkelbos WB WIM**

TRAFFIC HIGHLIGHTS OF SITE 3076			
1.1	Site Identifier		3076
1.2	Site Name		Kinkelbos WB WIM
1.3	Site Description		WB Screener between P.E. and N2/N10 I/C
1.4	Road Description		Route : N002 Road : N002 Section : 12X Distance : 5.3km
1.5	GPS Position		25.892166E -33.636581S
1.6	Number of Lanes		1
1.7	Station Type		Permanent WIM
1.8	Requested Period		2010/01/01 - 2010/12/31
1.9	Length of record requested (hours)		8760
1.10	Actual First & Last Dates		2010/01/01 - 2010/12/31
1.11	Actual available data (hours)		8760
1.12	Percentage data available for requested period		100.0
			Total
2.1	Total number of vehicles		1622070
2.2	Average daily traffic (ADT)		4444
2.3	Average daily truck traffic (ADTT)		682
2.4	Percentage of trucks		15.3
2.5	Truck split % (short:medium:long)		28 : 20 : 52
2.6	Percentage of night traffic (20:00 - 06:00)		13.2
3.1	Speed limit (km/hr)		120
3.2	Average speed (km/hr)		103.1
3.3	Average speed - light vehicles (km/hr)		106.7
3.4	Average speed - heavy vehicles (km/hr)		83.2
3.5	Average night speed (km/hr)		98.4
3.6	15th centile speed (km/hr)		83.7
3.7	85th centile speed (km/hr)		122.0
3.8	Percentage vehicles in excess of speed limit		16.6
4.1	Percentage vehicles in flows over 600 vehicles/hr		1.9
4.2	Highest volume on the road (vehicles/hr)	2010/09/26 18:00:00	875
4.3			
4.4			
4.5	Highest volume in a lane (vehicles/hr)	2010/09/26 18:00:00	875
4.6	15th highest volume on the road (vehicles/hr)	2010/03/26 16:00:00	699
4.7			
4.8			
4.9	30th highest volume on the road (vehicles/hr)	2010/09/23 17:00:00	646
4.10			
4.11			
5.1	Percentage of vehicles less than 2s behind vehicle ahead		19.3
6.1	Total number of heavy vehicles		248948
6.2	Estimated average number of axles per truck		4.9
6.3	Estimated truck mass (Ton/truck)		24.6
6.4	Estimated average E80/truck		1.5
6.5	Estimated daily E80 on the road		1004
6.6			
6.7			
6.8	Actual average daily E80 in the worst West lane		1008
6.9			
6.10	Actual average Axles/Truck (Short:Medium:Long)		(2.2 : 4.5 : 6.4)
6.11	Actual average Mass/Truck (Short:Medium:Long)		(8.7 : 21.6 : 34.1)
6.12	Actual average E80s/Truck (Short:Medium:Long)		(0.4 : 1.3 : 2.1)

Figure 25. Example Output from CTO Station

4.6.4.2 Dynamic Weighing or Weigh-In-Motion

> **WIM Calibration**
> A small bias in WIM measurements can result in a large error in the design traffic estimate. The proper calibration of WIM equipment is therefore of the utmost importance.

This method is recommended for Categories A and B roads carrying high volumes of traffic. The survey should be conducted for at least 7 days, and it must be assessed whether the survey was done during seasonally high traffic or exceptionally low traffic.

The axle load of individual axles is recorded under normal operating conditions. This type of survey is generally referred to as a weigh-in-motion (WIM) survey (see Section 4.2). The axle load measurements are done at the operating speed of the heavy vehicles, which impacts the results. Vehicles moving at speed tend to bounce and roll, while the wheels hop at a high frequency. This results in an axle with a fixed static weight causing a spread in results when weighed dynamically. For this reason, the approach to the WIM installation should be smooth and the pavement structure sound. The WIM must also be properly calibrated to get a reliable estimate of the static axle load. Referred to TMH3 (1988) for detailed survey requirements.

Two types of error are associated with the results from dynamic axle load surveys:

- **Systematic error or bias**: deviation of the observed dynamic axle load from the actual static load.
- **Random error**: spread of the observed dynamic axle load for a fixed actual static load.

Statistical techniques have been developed to correct the errors (Prozzi et al, 2006 and Theyse et al, 2007), but can only be applied if the level of error is known. The correction for random error has been shown to have a minimal effect on design traffic estimates. However, a 5% WIM bias may result in a 20% error in the design traffic estimate. The proper calibration of WIM equipment is, therefore, of the utmost importance.

4.7 Design Traffic Calculation

The steps involved in a design traffic calculation are shown in a flowchart in Figure 26.

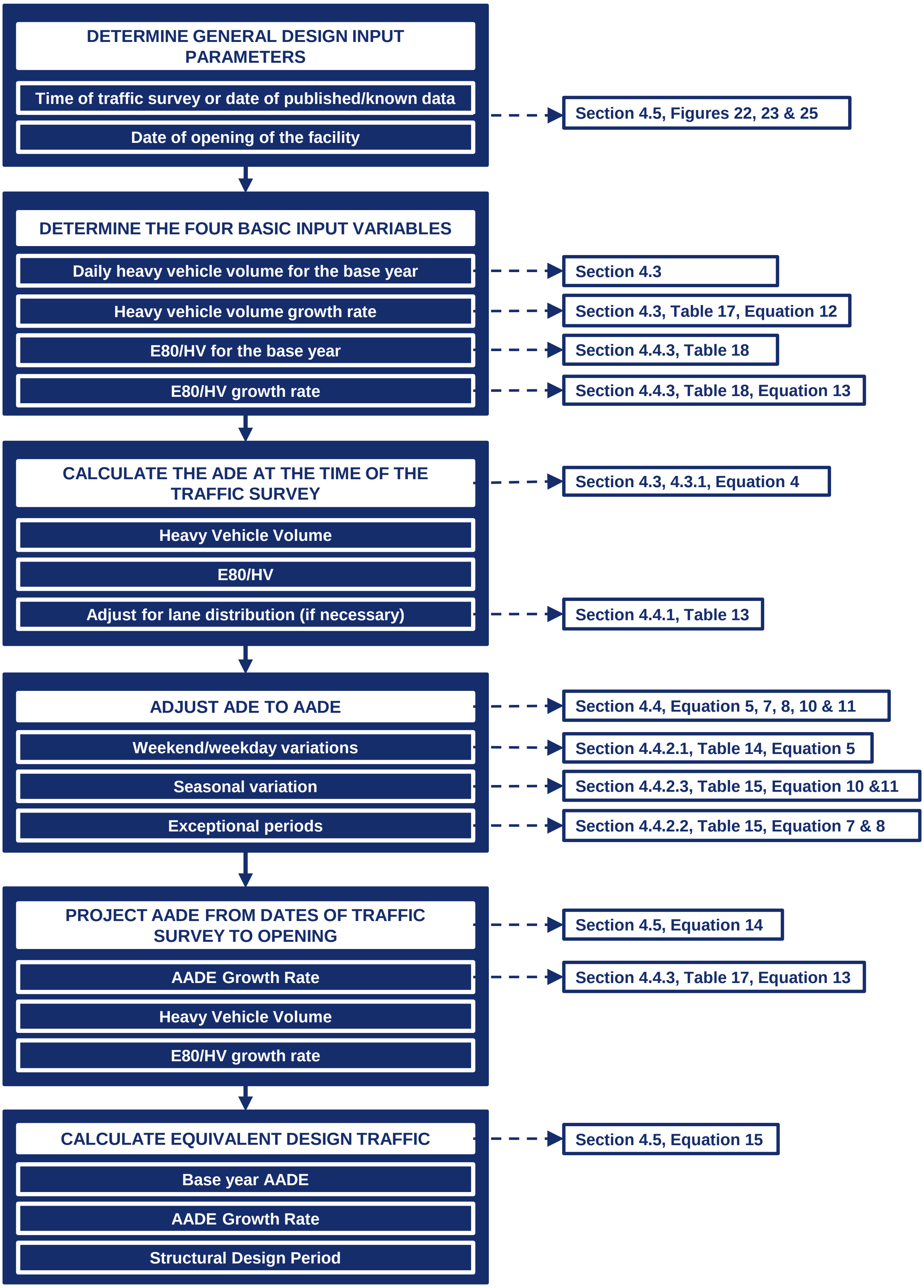


Figure 26. Calculation of Design Traffic

(i) Worked Example of Traffic Calculation

A worked example of calculating traffic from published data is shown in Table 22. The data are obtained from the CTO Station in Figure 25.

Table 22. Worked Example: Design Traffic Calculation using Published Data (CTO)

Parameter	Data	Comments
Step 1: Determine General Design Input Parameters		
Time of traffic survey or date of published/known data	2010/12/31	From CTO data (Figure 25).
Date of opening	01/01/2013	Assumed from project data.
Step 2: Determine the Four Basic Input Parameters		
Daily heavy vehicle volume per lane for the base year (HVV)	682	ADTT from CTO Data in Figure 25.
Heavy vehicle volume growth rate (h)	5%	Assumed from Table 17, range of 4 – 7% (routes connecting centres of moderate economic development, long-term values).
E80/HV at the survey time	1.5	Estimated average E80/truck from CTO Data in Figure 25.
E80/HV growth rate (v)	2%	Assumed from Table 18, for "all heavy".
Combined heavy vehicle growth and E80/HV growth	i = [(1+h)(1+v)] = [(1+0.05)(1+0.02)-1] = <u>0.07</u>	Equation (13), input from this step.
Step 3: Calculate the ADE at the Time of the Traffic Survey		
Adjust for lane distribution	N/A	CTO Data in Figure 25 indicates only 1 lane.
	$\text{ADE} = \dfrac{\frac{\text{E80}}{\text{HV}} \cdot \text{HVV}}{\text{duration x lanes}} = \dfrac{1.5 \cdot 682}{1 \cdot 1}$ = <u>1023</u>	Equation (4), input from Step 2. Data duration for full year.
Step 4: Adjust ADE to AADE		
Weekend/weekday variation	N/A	Data available for full year.
Seasonal variation		
Exceptional periods		
	ADE = AADE = 1023	No adjustments for variations.
Step 5: Project AADE from Date of Traffic Survey to Date of Opening		
Time from survey to opening date (n)	2 years	
AADE at time of opening	$\text{AADE}_O = \text{AADE}_S \cdot (1+i)^n$ $= 1023 \cdot (1+0.07)^2$ = <u>1171</u>	Equation (14), input from Steps 3 and 2.
Step 6: Calculate Equivalent Design Traffic		
Structural design period	20 years	Design assumption.
Equivalent traffic	$\text{ET} = 365 \cdot \text{AADE}_O(1+i)\dfrac{[(1+i)^y - 1]}{i}$ $= 365 \cdot 1171\,(1+0.07)\dfrac{[(1+0.07)^{20} - 1]}{0.07}$ = 18 748 635 = <u>**18 million E80s**</u>	Equation (15), input from Steps 6 and 2.

4.7.1 Sensitivity Analysis

The design traffic estimation relies on a number of input variables determined from traffic surveys or estimated. These inputs are therefore not exact and can vary. The sensitivity of the design traffic estimate to variation in the inputs must be evaluated. This is done with a simple sensitivity analysis. To do this, assign a low, medium and high value to each input variable, and repeat the design traffic calculation at different combinations of the inputs. A Monte-Carlo simulation technique may also be used where a statistical distribution is assigned to each of the inputs. The inputs are randomly generated from the distribution and the design traffic calculated. The process is repeated many times, and a distribution for the design traffic is generated.

4.7.2 Geometric Capacity Limitations

The volume of traffic that can be carried by a road is limited by its geometric capacity. The traffic volume at the end of the analysis period must therefore be compared with the road's capacity. The topography of the area and the level of service required also influence the capacity of the road, and must therefore be considered.

Geometric Capacity Limitations

The volume of traffic that can be carried by a road is limited by its geometric capacity. The traffic volume at the end of the analysis period must therefore be compared with the road's capacity.

5. PAVEMENT INVESTIGATION AND DESIGN PROCESS

The design investigation processes are different for new and rehabilitation design. Design investigation for new design focuses on the characterisation of the in situ subgrade and materials available from nearby sources, such as borrow-pits and quarries. Except for the utilisation of the in situ subgrade, the pavement is constructed from new. The design investigation for rehabilitation design aims to identify and remedy distressed and weak layers in the existing pavement structure, and attempts to maximise use of the existing pavement structure in the design of the new pavement structure.

For new pavements, the investigation delineates the subgrade into sections of similar quality. The rehabilitation investigation process divides the existing project road into a number of homogeneous or uniform sections, some of which only require remedial measures while others require structural strengthening to accommodate the anticipated future traffic. A structural pavement design needs to be done for those sections requiring structural strengthening similar to the structural design required for new pavements, to ensure adequate structural capacity to carry the traffic. When the structural design is actually being done, the differences between new and rehabilitation design are less significant. However, certain structural design techniques are better suited to rehabilitation design, while others are better suited to new design.

Pavement design for new and rehabilitation projects are based on the same principles, design considerations and techniques. However, the detail of the tasks performed during new and rehabilitation design is sufficiently different, especially during the design investigation phase, to warrant separate discussion. Figure 27 presents a diagram for new pavement design, and Table 23 provides a framework for rehabilitation design.

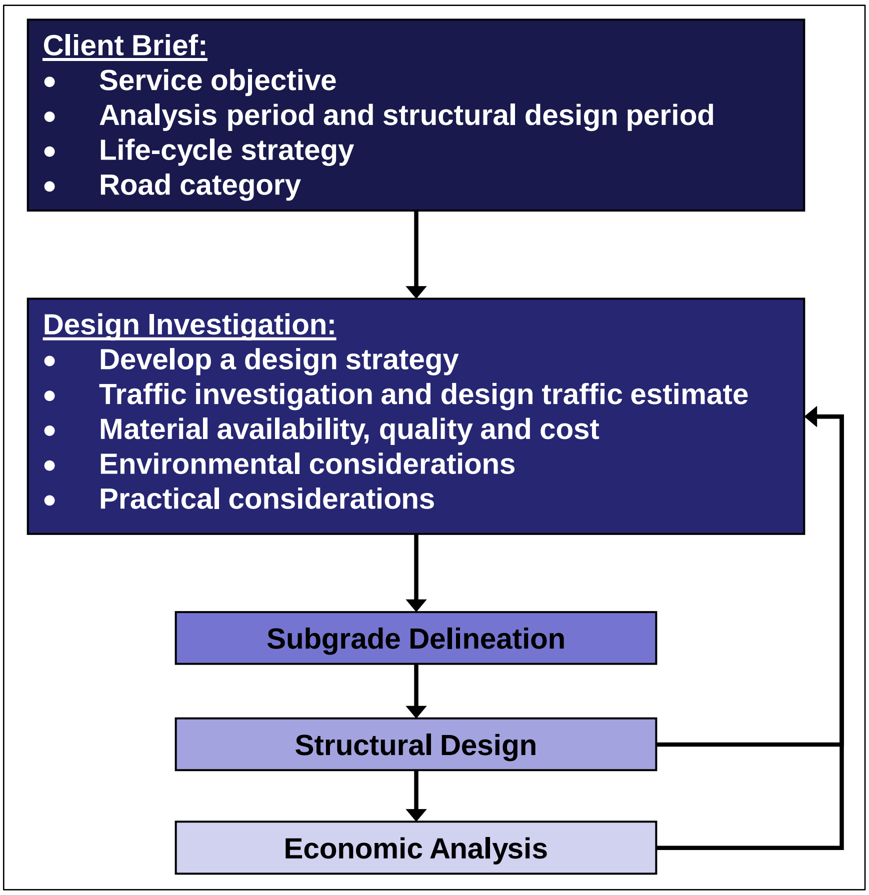


Figure 27. Flowchart for New Pavement Design

Table 23. Framework for Rehabilitation Investigation and Design

Main Component		Activity	Elements
Design Investigation	Initial assessment	Gathering information	• Client brief • Available information (traffic, environment) • Detailed visual inspection • Pavement surveillance measurements
		Data processing	• Traffic data • Visual inspection data • Surveillance data (rut, profile, deflection)
		Pavement evaluation and initial structural capacity estimation	• Condition assessment • Demarcation of uniform sections • Backcalculation of deflection data (if applicable) • Initial structural capacity assessment • Uniform section reports
	Detailed assessment	Additional testing	• Test-pits and bulk samples for laboratory testing • Relative movements at joints and cracks • Cores and DCP tests • Detailed traffic survey
		Cause and mechanism of distress	• Pavement condition and behavioural state • Distress manifestation and correlations
		Pavement situation description	• Update uniform section reports
	Design considerations	Practical and functional aspects	• Material availability and cost • Constructability • Performance adequacy • Maintainability
Rehabilitation Design		Identify rehabilitation options	• Eliminate weak layers and utilise existing strength • Refer to design considerations (Section 3)
		Structural pavement design	• Select appropriate design method(s)[1] • Prepare design method inputs • Structural capacity estimation and design
Economic Analysis		Present worth of cost (PWOC) analysis	• Decide on cost elements to consider • Develop life-cycle strategy for design alternatives • PWOC calculation
		Incorporation of uncertainty	• Decide on different strategies • Assign probabilities and outcomes • Calculate expected PWOC for each option • Calculate expected PWOC for each strategy

Note:
1. It is generally recommended that more than one design method is used to determine appropriate pavement structures.

5.1 Design Investigation

The first step for both new and rehabilitation design is the design investigation. The main purpose is to gain a comprehensive understanding of the design problem, the parameters governing the design and to obtain the necessary design inputs. The importance of the activities associated with the design investigation should never be underestimated. The eventual design is only as good as the information gathered during the design investigation, and incorrect design input from the design investigation may lead to significant design risk.

The design investigation begins with determining, in conjunction with the road authority, the design strategy. This includes determining the appropriate road category, design structural capacity, analysis and structural design periods and possible stage construction. The design strategy may also be influenced by the future maintenance and rehabilitation strategy for the road. For example, certain roads may need designs do not require frequent maintenance, to limit traffic disruptions. Therefore, the life-cycle strategy for the road needs to be determined. For this, the initial traffic demand is required, which may be obtained from past traffic surveys on the road and road management systems. It may be necessary to revisit the design and life-cycle strategies at times during the pavement design process, as when more accurate information becomes available.

5.1.1 New Design

For new design, the formulation of the design strategy is followed by these components of the design investigation:

- Detailed **traffic** investigation and design traffic estimate
- **Materials** investigation
- **Environmental** and **practical** considerations

5.1.1.1 Traffic Investigation and Design Traffic Estimate

Detail on the traffic investigation options and design traffic estimation process are given in Section 4. Traffic survey options are limited in the case of new pavement design because:

- The road is a **completely new facility** and a transportation study is required to estimate the future traffic.
- If the road is being **upgraded from unpaved to paved** standard, some indication of the future traffic may be obtained from the traffic using the existing gravel road. Provision must also be made for traffic attracted to the upgraded road.

5.1.1.2 Materials Investigation

The materials investigation potentially consists of three components for new pavement design:

- **Centre-line soils investigation** to delineate the in situ subgrade and to determine the subgrade class according to Section 3.6.1.1. See also Chapter 4, Appendix A and Chapter 6, Section 4.
- Confirmation of **material availability** and cost from excavations, borrow-pits and quarries. A materials checklist is useful. See Chapter 8, Materials Sources for more information.
- Assessment of the potential improvement of available materials through **mechanical or chemical stabilisation**. If the available material shows potential for improvement based on limited testing, and the structural design indicates that the improved quality is required, the initial tests are supplemented with a proper mix design.

The TRH4 (1996), for COLTO (1998) or TRH14 (1985) material classification systems are used to classify the subgrade. The TRH14 system is summarized in the Appendix to Chapter 4 and the materials are discussed in Chapter 4, Sections 2, 3 and 5 and Chapter 9. The material classification system for design, discussed in Chapter 9, Section 15, and in TG2 (2009) can also be used.

5.1.1.3 Environmental and Practical Considerations

The potential impact on the design of the environmental and practical considerations must be considered before proceeding with the structural design of the pavement. See Sections 3.6 and Section 5.2. This particularly applies to the climate and traffic environments, and their impact on the selection of an appropriate pavement type and surfacing, as well as drainage and compaction requirements.

5.1.2 Rehabilitation Design

The design investigation process for rehabilitation design is more involved than that of new design and is described in detail in TRH12 (1997) for flexible pavements. The principles are the same for rigid pavements. The aim of pavement rehabilitation is to restore deficient or failed pavements so that they can carry the future traffic expected over the design period with only routine maintenance. The scope of pavement rehabilitation, therefore, includes any of the following:

- Complete **pavement reconstruction**
- Partial reconstruction by **strengthening existing layers** before resurfacing
- Asphalt, granular or concrete **overlays**
- **Surfacing rehabilitation**
- **Drainage** provision or improvement of existing drainage facilities

The main components of a rehabilitation investigation are:

- **Initial** assessment
- **Detailed** assessment
- **Environmental** and **practical** design considerations

Rehabilitation Design

TRH12 is a good reference for most aspects of rehabilitation design for flexible pavements. The current version is from 1997, but is due for an update.

- TRH12: **Flexible Pavement Rehabilitation Investigation and Design**. 1997.

Although TRH12 is for flexible pavements, most of the principles discussed apply to rigid pavements.

The condition assessment, which is part of the initial and detailed assessments, forms the foundation of the rehabilitation investigation and design process. The aim is to determine uniform pavement sections at the project level that are similar in condition and have similar structural or functional improvement. This is done during the initial assessment. The cause and mechanism of distress on each uniform section is identified during the detailed assessment.

Uniform Sections

A uniform section is a section of road for which the condition and structure are similar. Appropriate rehabilitation options are therefore suitable for the whole section. The final design selected is used on the full uniform section.

Uniform sections should not be so short as to introduce construction problems.

The nature and detail of the condition assessment should take the following into consideration:

- Road category
- Available resources
- Nature, quantity and quality of data that is available at the onset of the investigation

The initial and detailed assessments are integral to the condition assessment, but are treated separately for ease of discussion. They are also discussed in Chapter 6.

5.1.2.1 Initial Assessment

The objectives of the initial assessment are:

- **Record and process data** into a format that may be readily used during subsequent detailed investigations.
- Benchmark the pavement condition against standard conditions. The condition limits given in TRH12 (1997) and Horak (2008) are useful for this.
- **Divide the road into uniform sections** that require different rehabilitation actions.
- Recommend appropriate measures for sections that obviously only exhibit **surfacing or localized distress**.
- Identify and recommend appropriate tests for the sections that need **detailed investigation** to determine the cause and mechanism of pavement distress.

The components of the initial assessment include the gathering of information, data processing and condition assessment, and an initial assessment of the structural capacity of the uniform sections.

(i) Information Gathering

The information gathered during the initial assessment is mostly obtained from three sources:

- **Preliminary investigation:** The preliminary investigation relies mostly on available data such as construction and pavement management system (PMS) data and a preliminary site visit. PMS data typically includes historic traffic data, surveillance measurements, and maintenance and rehabilitation records. Often the PMS surveillance measurements are sufficient, and only supplemental surveys are required. The preliminary investigation is followed by a detailed visual distress survey and non-destructive surveillance measurements.

Condition Assessment

These chapters discuss various elements of the condition assessment of roads, including the devices used for measurement:

- Chapter 6: **Road Prism and Pavement Investigation**, Section 5 and 6
- Chapter 14, **Post-Construction**, Section 2

- **Detailed visual inspection of the road:** Photographs are invaluable to the visual inspection process and create a permanent record of the visual distress. Photographs should be taken of the distress on the road, and the condition of road furniture such as guardrails, signposts and fencing. The detailed visual inspection is done to record and map:
  - **Position** of distress
  - **Mode and type** of distress
  - **Degree and extent** of distress
  - **Spacing** of distress
  - Pertinent **construction** details and deficiencies
  - **Topographical, geological and vegetation** clues to the cause of distress
  - Condition of **drainage** structures and facilities

- **Pavement surveillance measurements.** The automation of pavement surveillance for network level pavement management has led to the increased availability of valuable data, such as rut, riding quality and deflections, for project level preliminary investigations. In many situations, the Road Agency's PMS has sufficient data for the initial assessment phase. See Chapter 6, Section 5 and Chapter 14, Section 2 for discussion on the surveillance equipment.

TRH6 (1985) and TMH9 (1992) provide guidance on the rating of visual flexible pavement distress. Although TMH9 is primarily intended for network level evaluations, it contains useful photographic examples of flexible pavement distress. The degree rating system in TMH9 is valid for both network and project levels. The most critical aspect of the visual inspection is for the team doing the visual inspection to be properly trained and consistent in their evaluation of distress.

Guidance on the visual condition assessment for concrete pavements is provided in:

- **M3-1:** Visual Assessment Manual for Concrete Pavements (1998)
- **TRH19:** Standard Nomenclature and Methods for Describing the Condition of Jointed Concrete Pavements (1989).

Network vs Project Level Data Collection and Analysis

Data collection and analysis at the **network level** is intended to monitor the general performance of a network, and to highlight and prioritise sections for maintenance or rehabilitation.

Project level data collection and analysis is typically a lot more detailed than network level analysis. It is intended to gather sufficient information with which to assess the condition of the pavement, and to decide on appropriate maintenance and rehabilitation measurements.

For example, for network level monitoring, SANRAL measures FWD deflections at 200 metre intervals. For project level investigations, FWDs at least every 100 metres are recommended.

The most comprehensive method of recording distress is for inspection teams to walk the length of road (if safety conditions allow) and to map the chainage/longitudinal distance and transverse position of the distress. This process is, however, time consuming. If a competent team is doing the inspection, the visual distress is recorded in terms of a degree and extent rating similar to the process used for network level evaluations. However, for project level surveys, the segment lengths are normally reduced to 200 metres or less. The level of detail of the survey is also influenced by the perceived remedial or rehabilitation action required. For example, if pre-treatment such as patching followed by a surfacing is likely, then the comprehensive approach offers many benefits. If it is clear that some form of reworking is required, then the degree and extent rating system approach may suffice.

Additional information to allow for the selection of uniform sections is obtained from non-destructive testing, such as riding quality, rut depth and skid resistance measurements. These are used as measures of functional serviceability of the road. Deflection measurements, Dynamic Cone Penetrometer (DCP), rut depth measurements and trial pit results are used as indicators of the structural condition of the pavement. These are discussed in more detail in Chapter 6, Section 5. The length of uniform sections varies and the delineation of these sections is often based on a cumulative difference approach. Refer to Appendix J of the 1993 AASHTO Design Guide for details on the cumulative difference procedure (AASHTO, 1993).

(ii) Data Processing and Condition Assessment

Data processing of the visual inspection ratings, non-destructive test results and preliminary traffic data is done during the initial assessment phase. The effective evaluation of the collected data requires data to be categorized against set criteria and presented in a form that facilitates analytical comparison. Therefore, performance criteria are necessary for each type of measurement. A classification of the existing pavement condition and expected future behaviour as "sound", "warning" or "severe" is done based on each measurement type. The classification is in terms of current condition for the visual condition inspection data, and in terms of the current and expected future condition for measured pavement response.

Different performance criteria are applied to different road categories. Table 24 provides criteria for the percentile levels according to which the condition of the road should be classified for flexible pavements.

Measurement Types and Associated Criteria

Chapter 6: **Road Prism and Pavement Investigation**, Section 5, contains discussion on all measurement types and the associated criteria.

Table 24. Percentile Levels Recommended for Data Processing for Flexible Pavements

Road Category	Percentage of Road Allowed to Perform Unsatisfactorily at the End of the Design Period	Recommended Percentile Level for Data Processing
A	5	95
B	10	90
C	20	80
D	50	50

The classification system used for visual condition and surveillance measurements is based on the Road Category, and therefore the appropriate percentile value, and pre-defined criteria for each measurement type. These criteria are set out in Appendix 1 of TRH12 (1997). Assessment criteria for rigid pavements are included in Table 2 of this chapter. The length of pavement used to classify the pavement is generally a uniform section, for which the particular type of measurement seems consistent. Chapter 6, Section 5 contains discussion on all measurement types and the associated criteria.

The data processing, and specifically the classifications, is done with a view to collating the results to obtain a condition assessment.

The initial estimate of the past and future traffic demand is done in accordance with the calculations set out in Section 4.7. It is normally based on traffic data from network level observations.

(iii) Pavement Evaluation and the Initial Structural Capacity Assessment

All the information available from the initial assessment phase is used to identify uniform, or homogenous, sections with consistent problems and rehabilitation needs over the length of the uniform section. This includes the pavement structure information, probably from as-built data, visual condition survey data and non-destructive test results. All this information should be displayed in a concise manner similar to the example shown in Figure 28 for a flexible pavement. These types of plots are often called “stripmaps”.

The plot in Figure 28 contains the as-built pavement information and the pavement condition classification based on the visual condition, FWD deflection, rut and riding quality data. The information for this particular case shows that there is a change in both the visual condition and FWD deflection classifications at about km 11.77. At chainages lower than km 11.77 the pavement shows very little visual distress, except for the binder condition, and all the deflection, rut and riding quality parameters are low. The classification of the pavement condition at chainages less than km 11.77 is mostly sound. Higher than km 11.77, the pavement shows visual distress and the deflection and riding quality parameters are higher, resulting in warning to severe classifications for a number of parameters. The section of road shown in the example should be divided into two uniform sections at km 11.77.

Similar stripmaps can be prepared for concrete pavements by considering the failure mechanisms of concrete pavements as described in M3-1 (1998) or TRH19 (1989).

Once the project has been divided into uniform sections, an initial structural capacity assessment is done for each uniform section. It is recommended that this initial structural capacity assessment is done using simple, empirically derived relationships, such as (included in Section 7):

- Surface deflection methods
- DCP method, if DCP tests were done during the initial assessment and rut results are available
- AASHTO SN method, if riding quality results are available
- Pavement Number method

A more detailed and/or sophisticated structural capacity assessment is done during the rehabilitation design stage.

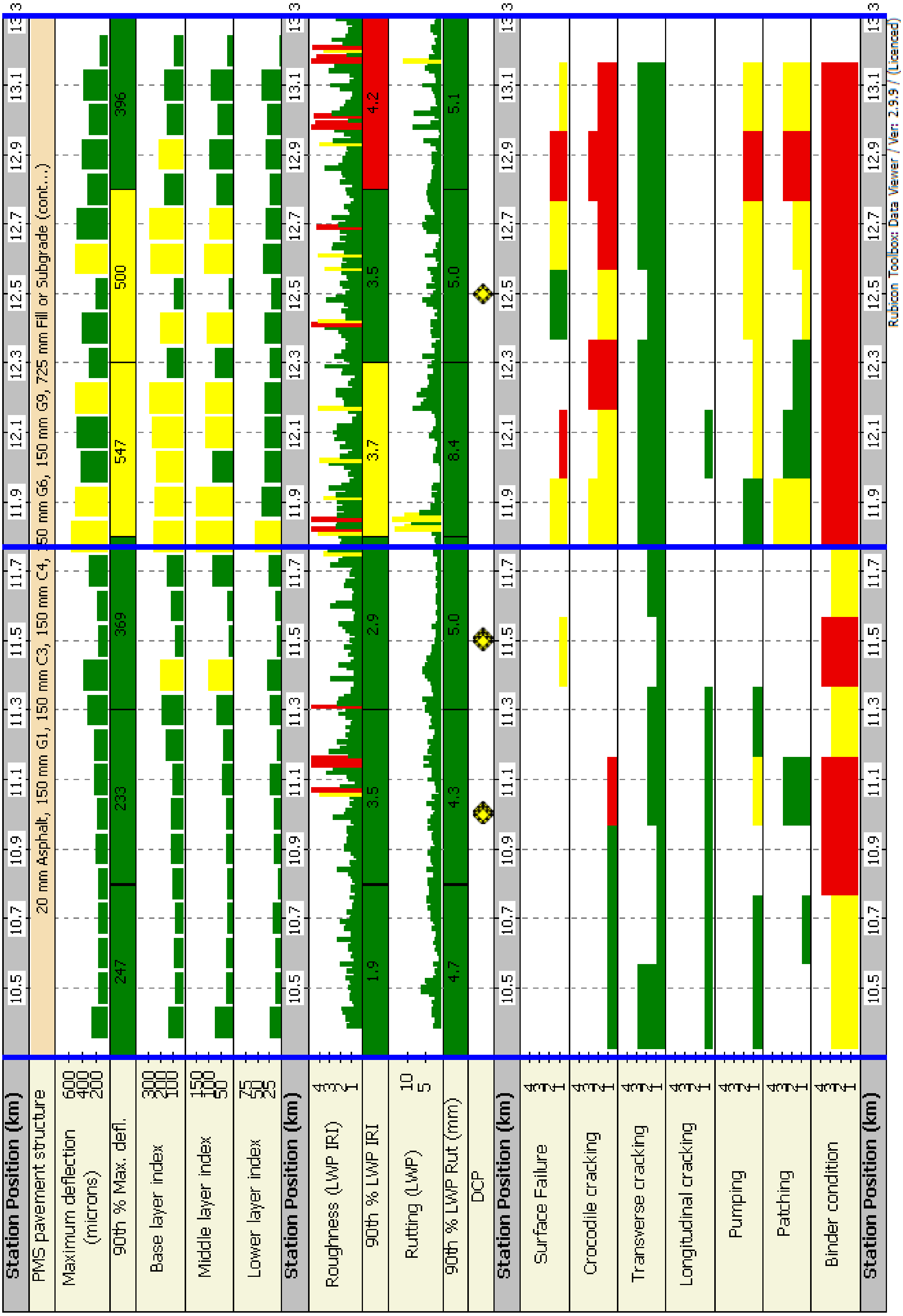


Figure 28. Concise Representation of Information from the Initial Assessment Phase

Once each lane of the full project length has been divided into uniform sections, and the initial structural capacity assessment completed, each uniform section is classified according to:

- **Sections with no significant problems** therefore requiring no action. Sections with no significant problems are those sections that can economically be kept at an acceptable level of service using routine maintenance. These sections can be often excluded from further analysis. The underlying assumption is that if the road looks fine, it is fine. However, if the future traffic demand is high, it is advisable to check the structural capacity of these sections as part of the detailed assessment phase.
- **Sections with only surfacing problems.** Ravelling, bleeding and polishing are classified as surfacing-only distress in the absence of other forms of distress. Further tests may be required on the surfacing material to establish appropriate rehabilitation alternatives, but these tests are not regarded as part of the rehabilitation design phase.
- **Sections with localized problems.** Areas of localised distress are identified and treated separately so as not to influence the assessment and design of the remainder of the project, and to ensure that appropriate measures are applied to these sections to prevent recurrence of the problem. In South Africa, inadequate drainage is by far the most common cause of localised distress. Vegetation close to the edge of the road is normally associated with drainages problems, as are blocked drains.
- **Sections requiring structural strengthening** for increased structural capacity where the estimated future traffic exceeds the remaining life.

Sections that do not fit in the above categories, or for which there is uncertainty regarding the cause of distress, or the initial structural capacity assessment indicates a low remaining structural capacity, require further analysis during the detailed assessment phase.

The final step of the initial assessment phase is to generate a uniform section report for each uniform section. This summarises the data collected during the initial assessment, with recommendations for:

- **Additional testing** required during the detailed assessment phase.
- **Remedial measures** for those sections not requiring structural strengthening.
- Sections that should be considered for **structural rehabilitation**.

These uniform section reports are updated during the detailed assessment phase, when more information becomes available.

Drainage

In South Africa, inadequate drainage is by far the most common cause of localised distress.

5.1.2.2 Detailed Assessment

The detailed assessment deals with those sections that are identified as probably requiring structural improvement. There are three activities typically associated with the detailed assessment phase:

- **Additional testing.** The purpose is to collect any additional information required, and to support tentative findings and recommendations from the initial assessment phase. Typically, the additional tests are destructive tests. These may include test pits and bulk material samples for laboratory testing, as well as cores from layers that are still effectively bound. If the traffic information available from the preliminary investigation is doubtful, a detailed traffic survey may be included during the detailed assessment to boost confidence in the design traffic estimate. Refer to Section 4.6 for traffic survey options.
- **Determination of the cause** and **mechanism of distress.** The establishment of the cause and mechanism of distress for each uniform subsection is crucial for the selection of appropriate remedial actions and effective rehabilitation design. A sound understanding of pavement behaviour is the key to identifying the origin and hence the cause and mechanism of distress. The aspects of pavement behaviour presented in Section 3.5 are useful for this analysis.
- **Pavement situation description**:
  - Type and thickness of layers
  - Traffic loading, both past and future
  - Environment, both temperature and moisture
  - Pavement performance
  - Present condition, visual as well as pavement tests
  - Cause and mechanism of distress

The final step of the detailed assessment phase is to update the uniform section reports from the initial assessment phase with more reliable information. Then, the cause and mechanism of distress can be pinpointed. The input for the structural pavement design should also be presented in a concise manner.

5.2 Design Considerations

Before embarking on the design, the designer should be aware of material availability and cost, as well as practical and functional considerations that influence the final rehabilitation decision.

5.2.1 Material Availability and Cost

The existing pavement is a major source of available material in a rehabilitation design. The potential utilisation of the existing pavement material through stabilisation must be investigated.

5.2.2 Constructability

Factors that need to be considered include:

- Traffic accommodation
- Widening of the paved surface
- Bridge and overpass clearance
- Layer thickness constraints
- Availability of resources
- Potential for deep in situ recycling
- Environmental impacts of the selected rehabilitation option

> **Reusing Materials in Existing Road**
> The existing pavement is a major source of available material in a rehabilitation design. The potential utilisation of the existing pavement material through stabilisation must be investigated.

5.2.3 Performance Adequacy

Drainage conditions and the selection of an appropriate surfacing or wearing course for specific characteristics should be kept in mind. For example, porous asphalt is used to reduce spray and noise.

5.2.4 Maintainability

Pavements have a long lifespan, including several maintenance and rehabilitation cycles. Maintainability considerations are often determined by the road authority's resources and policies, such as budget, maintenance capabilities, maintenance policies and political policies. It is unwise to opt for a rehabilitation option that impairs the future maintainability of the facility.

5.3 Structural Pavement Design

The structural design of a pavement, and subsequent economic analysis, should only proceed once sufficient information has been gathered during the design investigation.

5.3.1 New Design

The purpose of the structural design of a new pavement is to develop a number of potential designs with sufficient structural capacity to match the design traffic demand, while adhering to the principles of pavement design. Refer to Section 2 for the principles of design and Sections 7, 8 and 9 for structural design methods. Appropriate structural design methods for new design include:

- TRH4 (1996) catalogue and other industry catalogues
- DCP method, Section 7.3
- SAMDM for flexible pavements, Section 7.1
- Mechanistic-empirical design method for concrete pavements, Section 8.2
- Pavement Number method, Section 7.2
- Mechanistic-empirical design method for block pavements, Section 9.4

5.3.2 Rehabilitation Design

The philosophy of rehabilitation design is to fully utilise the remaining strength of existing layers. The structural design is done for each of the subsections requiring structural strengthening. From a practical point of view, it is not advisable to have short lengths of slightly different designs to be constructed under a single contract. Consideration should therefore be given to combine uniform sections in the design and documentation stage to ensure constructability.

5.3.2.1 Appropriate Rehabilitation Options: Flexible Pavements

Before suggesting an appropriate rehabilitation option, typical causes and mechanisms of distress are discussed.

> **Short Uniform Sections**
> From a practical point of view, it is not advisable to have uniform sections with short lengths of slightly different designs to be constructed under a single contract.

(i) Cause and Mechanism of Distress

The mode of distress, cracking or deformation, serves as pointers towards the origin and cause of distress in pavements. The types of distress that manifest in flexible pavements are illustrated and discussed in Chapter 14, Section 3.2.

Deformation in flexible pavements results from the following:

- Deformation caused by **surfacing inadequacies** may be identified by a poor rut-deflection correlation, and the DCP indicating a sound base/subbase combination.
- Deformation caused by a poor base and/or subbase normally exhibits a **narrow, deep rut with heaving** next to the rut. The DCP should also indicate a poor base and/or subbase quality.
- Deformation of the subgrade is normally characterised by a **wide rut** and a better rut-deflection correlation. Although deformation occurs in the subgrade, it may not necessarily indicate a poor subgrade quality and may be the result of inadequate protection or cover by the base/subbase layers.
- Post-construction compaction is normally characterised by the **absence of heaving next to the rut**. The width of the rut may depend on the depth at which the post-construction compaction occurred. DCP tests done in and between the wheel-paths should indicate an increased strength in the wheel-paths.
- Subgrade swell and collapse normally take the form of **transverse undulations** spaced along the length of the road, and are uniform across the width of the road. Knowledge on the geology of the subgrade also assists in determining the cause of distress as either swell of a clay subgrade or collapse of a sandy subgrade.

Cracking in flexible pavements results from the following:

- **Crocodile cracking** may occur for two reasons.
  - Crocodile cracks from fatigue cracking. In this case, the crocodile cracking may be confined to a region close to the wheel-tracks with an associated rut. High deflections with a small radius of curvature may point towards fatigue cracking. A soft base layer may also cause fatigue cracking and may be identified with DCP testing or radius of curvature measurements.
  - Aged, brittle asphalt surfacing. In this case, the cracking is distributed over the full width of the pavement. Rutting is not a prerequisite.
- **Block cracking** associated with shrinkage of treated/stabilised materials has a characteristic pattern, which is not confined to the wheel-tracks.
- **Longitudinal cracking** that is not confined to the wheel-tracks is indicative of construction joints or embankment instability.
- **Transverse cracking** is symptomatic of temperature associated distress, and can initiate structural problems if left unsealed.
- **Shrinkage cracking** of an asphalt surfacing is indicative of stiff, dry asphalt. The closing of these cracks in the wheel-tracks is an indication that the crack may be limited to the surfacing only.

(ii) Rectifying Deformation

Different remedial actions need to be considered, depending on the origin of deformation. Surfacing deformation is rectified by recycling or replacing the surfacing.

- A **thick overlay** may not be the most appropriate rehabilitation option for rectifying deformation originating from a poor base or subbase. The recycling of the base layer material, with the addition of low percentages of bituminous or cementitious binder should be considered. The deep, in situ recycling of base and subbase layers with the addition of low percentages of cement, emulsion or foamed bitumen is popular in South Africa. See Concrete inlays and overlays can also be considered.
- **Subgrade deformation due to insufficient cover** is rectified by reconstructing the base and subbase combination to form a reworked subbase, followed by the addition of a new base or a thick overlay.
- Measures for **rectifying subgrade swell and collapse** are not aimed at strengthening the pavement. Drainage may be the indirect cause of these problems, which then needs to be rectified. A thick asphalt levelling course can be applied to counteract undulations, after the pavement has settled. For swelling clays, the migration of moisture into and out of the subgrade needs to be limited and moisture equilibrium maintained. In

the case of existing pavements, special treatments, such as construction of cut-off membranes to sufficient depth on both sides of the structure, may be required.

- **Post-construction compaction** actually increases the structural capacity of the pavement. Normally, only a thin levelling course is required.

(iii) Rectifying Cracking

Different remedial actions are considered, depending on the origin of cracking:

- **Reflective cracking** from previously stabilised layers is rectified by ripping or milling and recompacting the treated layer. Granular overlays have also been proven effective in preventing propagation of reflection cracking. Because reflection cracks are often not associated with structural deficiency, some authorities permit their occurrence, but require sealing to prevent ingress of moisture as part of the routine maintenance programme.
- **Fatigue cracking** us rectified with an overlay of sufficient thickness; inlay (asphalt or concrete) to decrease the deflection; or, partial reconstruction, but be careful of reflective cracking. To mitigate potential reflective cracking, a stress absorbing membrane interlayer, typically a bitumen rubber seal, may be constructed before proceeding with the overlay. Where secondary cracking becomes visible, patching or milling of the defected areas are common pre-treatment requirements, before construction of an overlay for strengthening. See also crocodile cracking.
- **Crocodile cracking** caused by an aged surfacing is rectified with a special surface treatment; replacing the aged material with fresh material; or, where the extent is large, complete removal by milling.
- Surfaces with **advanced cracking** should be removed before an overlay is placed.

Note that when deep in situ recycling is used, the surfacing layer can be recycled in with the base materia l to form the new base.

5.3.2.2 Appropriate Rehabilitation Options: Concrete Pavements

(i) Cause and Mechanism of Distress

The mode and type of distress serves as pointers towards the origin and cause of distress in pavements. The types of distress that manifest in rigid pavements are illustrated and discussed in Chapter 14, Section 3.

The distress types in concrete pavements are:

- **Cracking**, including corner and mid-slab cracking, cracking close to a joint and closely spaced cracks.
- **Shattered slabs**
- **Pumping**
- **Faulting**

These distresses are construction, environmental, traffic and load related. Poor or erodible support under concrete pavements leads to pumping, and later faulting and cracking under traffic loading and if water is allowed to enter through cracks and poorly maintained joints.

Cracking of slabs is initiated by shrinkage of the concrete, either through concrete mix properties or low temperatures. Corner and mid-slab cracking is only found in jointed concrete slabs with curling at the corners or between transverse joints, loss of slab support and loading at the corners or joints occurs. These cracks are normally properly sealed to keep surface water from entering the pavement structure. When surface cracking occurs, partial depth repairs may be considered. However, the rate of success of this approach makes it a questionable option.

Cracking close to joints normally develops as a result of poorly designed or constructed joints. Closely spaced cracks develop in CRCP pavements where variations in concrete properties occur. These cracks do not require maintenance, but may eventually lead to punch-outs in CRCP.

If cracks are wide enough, and joints are not maintained properly, surface water enters the pavement and accumulates between the concrete slab and the supporting subbase layer. The deflection of the slab under traffic loading results in erosion of this supporting layer, manifesting as pumping of fines through joints and cracks.

Maintainability

It is unwise to opt for a rehabilitation option that impairs the future maintainability of the facility.

In a jointed concrete pavement, erosion of the supporting layer not only results in pumping, but also causes displacement of subbase fines from the slab downstream of the transverse joint towards the upstream side of the joint. Accumulation of fines below the upstream slab caused it to rise, whereas the downstream slab goes down due to the void. This results in a step at the transverse joint, also called faulting. Usually the void that develops under the downstream side of the transverse joint results in transverse cracking about 2 metres away from the joints. Faulting affects the riding quality of the road, and is rectified by grinding down the step at the joints.

Punch-outs, i.e., loose blocks about 300 mm x 300 mm in size, are only found in CRCP. Punch-outs develop where transverse shrinkage cracks are relatively wide and are closely spaced (load transfer at these cracks is limited), pumping is occurring and longitudinal cracks occur between these transverse cracks.

Once cracking of the slab becomes excessive, resulting in loose blocks, punch-outs and a loss in riding quality, it is defined as a shattered slab, which then requires a full depth repair or even slab replacements. Methods for this are described in the Concrete Road Construction Manual (C & CI, 2008).

Overlays using asphalt or concrete are also considered as a long term rehabilitation option. To avoid reflection cracking, the type of asphalt mix must be carefully selected to resist the relative vertical movement at joints and cracks.

Before overlays are placed, and as routine maintenance, severely cracked or shattered slabs should be replaced, cracks should be cleaned out (routed) and resealed with silicone or bitumen rubber.

5.3.2.3 Appropriate Rehabilitation Options: Block Pavements

Block pavements can exhibit deformation as in flexible pavements. Cracking of the concrete blocks can occur which generally indicates a strength problem of the concrete used for the manufacturing of the blocks. Should the block pavement need to be repaired, the blocks can be removed, the support reinstated and then the same blocks relaid, provided the blocks are undamaged.

5.3.2.4 Appropriate Rehabilitation Options: Composite Pavements

Composite pavements are pavements that are rehabilitated by using concrete and bituminous materials together. For example, concrete overlays can be constructed on flexible pavements. This is often referred to as white-topping. A special case is an Ultra-Thin Continuously Reinforced Concrete Pavement (UTCRCP), which is at least 50 mm thick and constructed on an existing flexible pavement (see Chapter 9, Section 12.2.2). Full or partial depth concrete inlays are constructed in the slow lanes of flexible pavements, where the heavy traffic has increased to such an extent that flexible pavements are no longer cost effective. The fast lanes of heavily trafficked roads can be constructed as a flexible pavement and the middle and slow lanes, or only the slow lane, in concrete, to cater for the heavier loads.

Concrete roads with insufficient texture depth, or where the riding quality is unacceptably low, are repaired by regrinding to restore the skid resistance and riding quality. However, this type of restoration requires expensive specialised equipment and, if it is not required on large scale, is likely to be unfeasible. Pavements which have been heavily repaired, or show severe faulting of joints and cracks, are overlaid with asphalt or concrete. Where new concrete inlays constructed by labour intensive methods have a low riding quality, this is rectified by the construction of an asphalt overlay.

Where the noise levels on concrete pavements are an issue, particularly in urban areas, the surface can be treated to reduce the noise. Re-texturing or asphalt overlays are an appropriate surface treatment in this situation.

Composite pavements have some special construction requirements, for example, the vertical longitudinal joint between a concrete and asphalt pavement. Chapter 12, Section 3.12.7.1 discusses some special requirements.

Decision makers should note that creating a composite pavement by overlaying concrete with asphalt results in a new pavement system, requiring a different set of deterioration models and intervention criteria in pavement management systems. Traditional flexible and concrete models do not necessarily apply.

5.3.2.5 Rehabilitation Design Methods

The structural capacity of uniform sections requiring structural improvement may be increased by an overlay, partial or full reconstruction. Many methods of designing these rehabilitation options are available, and are discussed in Sections 7, 8 and 9.

The most appropriate methods should be selected for the design. The input information required for a structural design should be available if the process outlined in this chapter and Chapter 6, Section 5 is followed, and a uniform section report produced.

5.4 Economic Assessment

The selection of the final pavement design is based on the life-cycle economic assessment of a number of alternative designs. The purpose of the structural design method is therefore not the selection of the final design, but to provide the designer with a number of design alternatives with the required structural capacity. The details of the economic analyses for new and rehabilitation design are slightly different, refer to TRH4 (1996) and TRH12 (1997) for more information. Although TRH4 and TRH12 refer to flexible pavements, the same economic principles apply for concrete or concrete block pavements.

The basic principle of the economic assessment is the same for new and rehabilitation design. The cost comparison of alternative pavement designs, for a specific design case, is based on the Present Worth of Cost (PWOC), Net Present Value (NPV) and Internal Rate of Return (IRR) of the initial construction and anticipated maintenance and rehabilitation costs. The cost comparison allows the selection of the final design on economic considerations, but should not override all other considerations. Financial affordability should also be considered. The availability of funds for the initial construction, and the availability of maintenance funds must be considered, as these could influence the final design decision.

The PWOC of the alternative designs is calculated from Equation (16), and the NPV from Equation (17). The IRR is obtained by setting the NPV in Equation (17) equal to zero, and solving for r, the real discount rate.

$$\mathrm{PWOC} = \mathrm{C} + \left(\sum_{i=1}^{m} \frac{\mathrm{M_i}}{(1+\mathrm{r})^{n_i}}\right) - \frac{\mathrm{S}}{(1+\mathrm{r})^{p}} \tag{16}$$

$$\mathrm{NPV} = -\mathrm{C} - \left(\sum_{i-1}^{m} \frac{\mathrm{M_i}}{(1+\mathrm{r})^{n_i}}\right) + \frac{\mathrm{S}}{(1+\mathrm{r})^{p}} \tag{17}$$

where
PWOC = Present Worth of Cost
NPV = Net Present Value
C = Construction cost in terms of current cost
S = Salvage value in terms of current cost
M_i = Cost of the i^{th} maintenance or rehabilitation action in terms of current cost
m = Total number of maintenance and rehabilitation actions
n_i = Number of years from the present to the i^{th} maintenance or rehabilitation action
p = Analysis period
r = Real discount rate

Items that are assumed to be equal for different design alternatives, such as the salvage value of the pavement at the end of the analysis period and the road user cost, are usually omitted from the economic assessment. An indication of the real discount rate is obtained from the difference between bank lending rates and the inflation rate.

A difference in the economic indicator between two alterative designs of less than 10% is insignificant, and the designs are assumed to be equivalent in economic terms.

The development of a realistic life-cycle strategy is of utmost importance to the validity of the economic assessment. The cost of future maintenance and rehabilitation actions cannot be ignored. Table 25 lists typical ranges of surfacing life for different combinations of base type, surfacing type, road category and traffic class. Table 26 provides typical future maintenance and rehabilitation measures for different pavement types. These values are only meant for planning purposes during the design stage, and may be different under in service conditions. Monitoring and management of in service maintenance and rehabilitation actions form part of the activities of the Pavement Management System (PMS).

Table 25. Typical Ranges of Surfacing Life for Flexible Pavements

Base Type	Surfacing Type (< 50 mm thickness)	Range of Surfacing Life (Years) for Combinations of Road Category and Traffic Class		
		A	B	C, D
		ES3 – ES10	ES3 – ES10	ES1 – ES10
Granular	Bitumen sand or slurry seal	–	–	2 – 8
	Bitumen single surface treatment	6 – 8	6 – 10	8 – 11
	Bitumen double surface treatment	6 – 10	6 – 12	8 – 13
	Cape seal	8 – 10	10 – 12	8 – 18
	Continuously graded asphalt	8 – 11	–	–
	Gap graded asphalt	8 – 13	–	–
Hot mix asphalt	Bitumen sand or slurry seal	–	–	2 – 8
	Bitumen single surface treatment	6 – 8	6 – 10	8 – 11
	Bitumen double surface treatment	6 – 10	6 – 12	8 – 13
	Cape seal	–	8 – 15	8 – 18
	Continuously graded asphalt	8 – 12	8 – 12	–
	Gap graded asphalt	8 – 14	10 – 15	–
	Porous asphalt	8 – 12	10 – 15	–
Cemented	Bitumen sand or slurry seal	Base type not used for category A roads	–	–
	Bitumen single surface treatment		4 – 7	5 – 8
	Bitumen double surface treatment		5 – 8	5 – 9
	Cape seal		5 – 10	5 – 11
	Continuously graded asphalt		5 – 10	–
	Gap graded asphalt		6 – 12	–

Table 26. Typical Future Maintenance Actions for Life-Cycle Cost Analysis of Flexible Pavements

Base Type	Surface Maintenance		Structural Rehabilitation	
	Surface Treatment Original Surfacing	Asphalt Original Surfacing	Moderate Distress	Severe Distress
Granular	S1 @ 9 years S1 @ 18 years	S1 @ 9 years S1 @ 18 years or 40 mm AG @ 11 years 40 mm AG @ 22 years	30 – 40 mm AC/BRAC	> 100 mm BC or Granular/concrete overlay or Recycling
Hot mix asphalt	S1 @ 10 years S1 @ 20 years	S1 @ 11 years S1 @ 21 years or 40 mm AG @ 11 years 40 mm AG @ 22 years	30 – 40 mm AC/BRAC	> 100 mm BC/concrete or Recycling
Cemented	S1 @ 5 years S1 @ 10 years S1 @ 15 years S1 @ 20 years	S1 @ 5 years S1 @ 10 years S1 @ 15 years S1 @ 20 years	Further surface treatments	Thick granular/concrete overlay or Recycling
Concrete	Re-grooving @ 25 years		30 – 40 mm AC/BRAC	Concrete

Notes:
AC is continuously graded hot mix asphalt
BC is hot mix asphalt base
BRAC is bitumen rubber asphalt
S1 is a single surface treatment

(i) TRH12

TRH12 (1997) introduces uncertainty in the economic assessment by using decision trees and Bayesian theory to assess various probable outcomes, within the present worth of cost method. This procedure allows for the incorporation of personal experience in the formal analysis procedure. Refer to TRH12 for more detail.

(ii) Highway Design and Maintenance Standards Models (HDM)

The Highway Design and Maintenance Standards Model (HDM-III), developed by the World Bank, has been used for over two decades to combine technical and economic appraisals of road projects, to prepare road investment programmes and to analyse road network strategies.

The International Study of Highway Development and Management (ISOHDM, 2004) was carried out to extend the scope of the HDM-III model, and to provide a harmonised systems approach to road management, with adaptable and user-friendly software tools. This produced the Highway Development and Management Tool (HDM-4), which is also used for the economic assessment of road projects. The scope of HDM-4 goes beyond traditional project appraisals and provides a powerful system for the analysis of road management and investment alternatives. The HDM-4 analytical framework is based on pavement life cycle analysis, over typically 15 to 40 years, to predict the following over the life cycle of a road pavement:

- Road deterioration
- Road work effects
- Road user effects
- Socio-economic and environmental effects

Once constructed, road pavements deteriorate as a consequence of several factors, most notably:

- Traffic loading
- Environmental weathering
- Effects of inadequate drainage systems

The rate of pavement deterioration, and overall long-term condition, is directly affected by the maintenance applied to repair defects on the pavement surface. When a maintenance standard is defined, it imposes a limit to the level of deterioration that a pavement experiences. Consequently, in addition to the capital costs of road construction, the total costs incurred by road agencies depend on the applied standards of maintenance and improvement.

The impacts of the road condition and design standards on road users are measured in terms of road user costs, and other social and environmental effects. Road user costs comprise vehicle operating costs, costs of travel time and the cost to the economy of road accidents.

Economic benefits from different road investment options are determined by comparing the total cost streams for various road works and construction alternatives against a base case alternative. The base case is not doing the project or doing the minimum, usually representing the minimum standard of routine maintenance. HDM-4 estimates the costs for a large number of alternatives year-by-year for a user-defined analysis period. All future costs are discounted to the specified base year.

To make these comparisons, detailed specifications of investment programmes, design standards, and maintenance alternatives are needed, together with unit costs, projected traffic volumes, and environmental conditions. Although a substantial number of inputs to this system are required, only a few inputs and variables are project specific. The rest remain unchanged for the larger economic environment. For this reason, road authorities requiring HDM analysis usually adopt defaults, or standard inputs, for non-project related variables.

6. STRUCTURAL CAPACITY ESTIMATION

The purpose of structural pavement design methods are to provide a method for the unbiased estimate of the structural capacity of alternative design options, with the aim of selecting the most economical option and that ensures the traffic demand will be met.

Structural design methods vary greatly in the amount of detail required and the level of analysis involved. The methods are generally:

- **Empirical methods,** based on observations of the performance of pavements.
- **Mechanistic-empirical (ME) methods,** which analyse the pavement as a mechanism, and link mechanistic parameters to the structural capacity through empirical observations of performance.
- **Catalogues** with standard designs for general conditions. These are developed either empirically or using ME methods, or a combination of both.

The next three sections, Sections 7, 8 and 9, include the common empirical and mechanistic-empirical design methods for flexible, rigid and block pavements. Section 6.1 gives some general comments on Mechanistic-Empirical Design Methods.

TRH4 (1996) contains catalogues of pavements for different combinations of pavement type, road category and design structural capacity for flexible pavements. The flexible pavement designs are all based on the South African Mechanistic-Empirical Design Method of 1995 (Theyse et al, 1995 and Theyse et al 1996). These catalogues are “tried and tested”, and should be used to benchmark any pavement design. The M10 manual (1995) contains some catalogues for rigid pavements. UTG (1982) contains catalogues for block pavements.

Design Catalogues

These catalogues available in TRH4 are “tried and tested”, and should be used to benchmark any pavement design.

It is important to keep in mind that there is no single structural capacity value associated with a pavement, but rather a range of appropriate values. This is because of the variation in all the variables used to determine the structural capacity, resulting in a distribution in the structural capacity assessment. In addition, all assessment methods are estimates, and no one method provides an absolute prediction. The structural capacity estimate is therefore a single point estimate taken from a possible distribution of values and most often reflects the average value. The use of stochastic simulation techniques to estimate a structural capacity distribution, such as the Monte Carlo simulation technique, is useful. An example of the use of such a method is given in Jooste (1997).

6.1 General Comments on Mechanistic-Empirical Design Methods

Classical mechanistic-empirical (ME) methods have been used for many years. Many aspects of this type of analysis are common to all ME methods and to rigid, flexible and block pavements. Some general comments on these methods are discussed here, with details on specific methods given in Sections 7.1 for flexible pavements, 8.2 for rigid pavements, and 9.4 for block pavements.

Mechanistic-empirical methods analyse the pavement as a mechanism by assuming a material model for the pavement type (see Chapter 2, Section 9) and calculates engineering parameters such as stresses and strains. The engineering parameters are then linked to the structural capacity of the pavement through observations of performance. These methods follow a logical structure and adhere to the laws of solid mechanics, but have absolutely no “intelligence” incorporated in the design method. Therefore, although the design method enables the design engineer to develop a pavement design with sufficient structural capacity, it may not necessarily be the optimal design. The “intelligence” in the system is provided by experience and sound engineering practice. Fortunately, much of the sound engineering practice in South Africa has been captured in national and industry guideline documents.

Figure 29 shows the typical components of a classical mechanistic-empirical design method.

ME Methods

Although ME design methods enable a pavement design with sufficient structural capacity, it may not necessarily be the optimal design. **The “intelligence” in the system is provided by experience and sound engineering practice.**

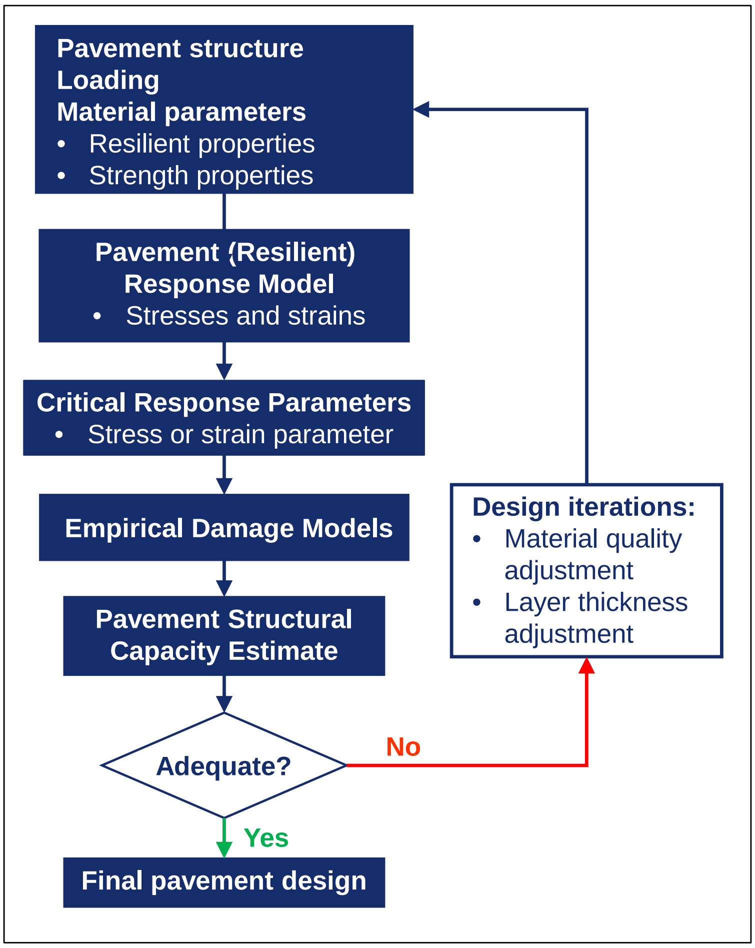


Figure 29. Main Components of a Mechanistic-Empirical Pavement Design Method

The process begins with the characterisation of the design problem in terms of the following inputs:

- **Pavement structure**
  - Number of layers
  - Layer thickness
- **Material characteristics for each pavement layer**
  - Resilient response characteristics: Resilient modulus and Poisson's Ratio (see Chapter 2, Section 10.1 for definitions)
  - Strength characteristics, which depend on the material type
  - Material properties, which also depend on the material type
- **Design loading**
  - Number of loads
  - Coordinates for each individual load
  - Contact stress, load or load radius for each individual load

(i) Loading

Historically, ME design has used the standard axle as the load input. See Section 4.1.3 for a definition of a standard axle and details on how to convert actual loading to the equivalent standard axle. A circular load of uniform contact pressure is used.

(ii) Inputs

The solution type used for the multi-layered pavement system dictates the required material models and inputs, namely Young's modulus and Poisson's ratio (defined in Chapter 2, Section 10.1). Young's modulus applies to perfectly linear-elastic materials, while pavement materials mostly behave non-linearly and inelastically. The resilient modulus (M_r), a linear secant modulus, is used as the Young's modulus input to approximate the non-linear, stress-strain behaviour of pavement materials.

(iii) Pavement Response Models

The pavement response model uses layered elastic analysis to determine the displacements, strains and stresses induced in the pavement by the loading. This is done through the pavement response model, which attempts to model the resilient response of the individual pavement layers and the whole pavement system. The response model therefore includes the material models and the system model:

- The **material model** describes the stress-strain behaviour of the material in each of the pavement layers when viewed in isolation.
- The **system model** combines the material models of the individual pavement layers, the interaction between pavement layers, the external loading and the boundary conditions of the problem to model the response of the complete system.

The continuum mechanics model used in classical ME models for pavement materials is the homogenous, isotropic, linear-elastic model. See Chapter 2, Sections 9 and 10 for a description of this model and further discussion. This solution is available in a number of software packages, the oldest being BISAR, ELSYM5, CHEV15 and WESLEA, some of which are part of Cyrano, ME-Pads and Rubicon Toolbox, which are more commonly used now. See Sections 7.10, 8.4 and 9.6.

The pavement response model provides stress and strain results at any location within the pavement system using the multi-layered linear-elastic system. The damage in pavement layers is determined by the stress or strain induced at specific locations in the pavement structure. The location and stress or strain parameter is determined by the material type and associated expected distress mechanism. These stresses and strains at specific locations in the pavement are referred to as critical parameters and serve as the primary, load related, input to the damage model.

See the green side-box below for a discussion on the complexities of the response models.

(iv) Damage Models

The permanent response of the pavement to loading is captured in the damage models, also referred to as 'transfer functions' or failure criteria. The transfer functions are material specific and are calibrated for the dominant mechanical modes of distress, permanent deformation and fatigue in flexible pavements and shattered slabs, pumping and faulting for rigid pavements. The calibration of the transfer functions is done from observed damage data and is thus the empirical component of the process. The damage models often include some measure of material strength. Although the strength properties are associated with the damage models, they are also part of the material input parameters. Often these are not known explicitly and typical properties associated with the material type and class are used, termed "default" properties. Examples of the types of strength and materials properties included are the shear strength, density and saturation for granular materials and strain-at-break for lightly cemented materials.

The structural capacity of a pavement also depends on the conditions in which the pavement is operating. Wet conditions, for example, reduce the structural capacity of a pavement. Classical ME design methods assume that a consistent set of conditions apply for the duration of the structural life of the pavement.

Most mechanistic-empirical (ME) methods were only calibrated for predefined terminal conditions. These methods estimate the structural capacity (N) from the initial condition to a predefined terminal level of distress. No information is provided on how damage is accumulated and how the terminal condition is reached.

(v) Recursive Analysis Methods

Recent developments in ME design methods involve modelling the incremental damage that occurs within periods of similar conditions in terms of traffic loading, environmental conditions and pavement characteristics. Methods based on this approach are referred to as recursive ME-design methods. The incremental damage that occurs within a period of similar conditions may be modelled using a linear incremental damage model, based on Miner's Law, or a non-linear incremental damage model. For linear recursive methods, the damage models from classical ME design methods may be used, although they were not calibrated for this use. Non-linear recursive methods require more advanced mathematical formulations of the damage models to capture the non-linear accumulation of damage. Research is underway to incorporate recursive methods into the South African Mechanistic-Empirical Method. See Section 4.1.3.3 for a brief discussion on recursive methods.

Discussion of Complexities of Response Models

The continuum mechanics model used for pavement materials is the homogenous, isotropic, linear-elastic model. See Chapter 2, Sections 9 and 10 for a description of this model and further discussion.

The system response for continuum mechanics models is done using integral transformation or finite element techniques.

- **Integral transformation techniques** are based on closed form integral solutions of the displacement, strain and stress of layered systems. These solutions are derived from classic elasticity theory for layered systems of simple geometry and basic load types.
- **Finite element solutions** model the pavement system as a number of separate but interconnected elements. Finite element solutions can accommodate more complex geometry, including pre-defined cracks in the pavement system, as well as non-linear material models.

Both solutions consider conditions of equilibrium and compatibility as well as the boundary conditions of the problem, to solve the internal displacement, strain and stress for a given external load. These system models are also based on static or dynamic response analyses, although static response models are much more common.

Static response analysis assumes that the load is applied to the system for such a long period that the response of the system comes to rest. Although there are internal displacements in the system and therefore displacement at the boundaries of the system, these displacements are constant and the velocity and acceleration of all points within the system are zero. The external load, is therefore, only resisted by the stiffness of the system.

Dynamic response analysis incorporates the effects of load magnitude variation, movement of the point of load application and the dynamic response of the system to the changing load conditions. The load characteristics therefore change continuously, and the system reacts dynamically and has not come to rest. The damping and inertia of the system therefore needs to be included in the response, in addition to the system stiffness.

The best known, and most often used, response model is the integral transformation solution for the static analysis of a homogenous, isotropic multi-layered, linear-elastic system subjected to a circular load of uniform contact pressure. This solution is available in the pavement engineering industry in a number of software packages, the most common being the older BISAR, ELSYM5, CHEV15 and WESLEA and the more recent Cyrano 200, ME-Pads, and Rubicon Toolbox (see Section 7.10).

7. STRUCTURAL CAPACITY ESTIMATION: FLEXIBLE PAVEMENTS

There are many structural capacity estimation methods available for flexible pavements. Each has their own advantages and disadvantages, and may only be used for appropriate situations. The following methods are discussed in this section:

- South African Mechanistic-empirical Design Method (SAMDM)
- Pavement Number (PN)
- Dynamic Cone Penetrometer (DCP)
- AASHTO Structural Number (SN)
- FWD deflection bowl parameter method
- FWD Structural Number (SN)
- TRRL Surface Deflection
- Asphalt Institute Surface Deflection

Mechanics of Materials

The mechanics of materials, including stresses and strains and materials models, are discussed in Chapter 2, Sections 9 and 10. How these concepts are applied to pavement materials is also discussed.

This section briefly discusses each of these methods, and provides a reference for each design method where full details are available. The advantages and disadvantages of the methods are summarised, and the applicability to various pavement situations is mentioned. It is the responsibility of the designer to select the most appropriate method(s) for a particular design situation.

The input parameters required by the selected design method should be determined for the actual materials available for the project. It serves no purpose to select values from published data, because that only results in a general design similar to those already contained in a design catalogue. Mechanistic-empirical methods do, however, provide the scope to study the impact of changes in certain input parameters, such as the loading conditions and pavement composition, on the structural capacity estimate.

7.1 South African Mechanistic-Empirical Design Method (SAMDM)

The 1996 version of the South African Mechanistic-Empirical Design Method (SAMDM) has been widely used in South Africa for many years. The historical development of the method is discussed in Theyse et al (1996). Ranges of typical resilient modulus and material strength input values are published for South African road-building materials. Damage models were calibrated for each of the main material groups used in South African road construction.

The damage models currently provided for by the 1996 version of the SAMDM are:

- **Hot mix asphalt fatigue**
  - Fatigue of thin (< 50 mm thick) surfacing layers: continuously and gap-graded
  - Fatigue of thick (> 75 mm thick) base layers
- **Unbound granular base and subbase layer**
  - Permanent deformation
- **Cemented base and subbase layers**
  - Crushing failure
  - Effective fatigue
  - Permanent deformation
- **Subgrade permanent deformation**

The SAMDM is currently under review. Details on the anticipated characteristics of the revised method can be found in Theyse et al, 2007, with some interim recommendations and models included in Theyse et al, 2011. Where possible, some brief details on the interim SAMDM (SAPDM) are provided. Further updates are likely, with the revised SAPDM likely to be published in 2013.

Substantial research has recently been done on bitumen stabilised materials (BSMs) (TG2, 2002; Long and Theyse, 2003; Jooste et al, 2009), unbound granular materials including crushed stone and natural gravel, and South African subgrades (Theyse, 2001) to improve the mechanistic-empirical models. The improved models for these materials will be incorporated as part of the method revision.

Latest AASHTO Design Guide

In the USA, a mechanistic-empirical design guide has been developed by AASHTO for all typical pavement types. The method is also known as NCHRP 1-37, and was published in 2004. The method is complex, and needs to be calibrated for climatic areas and specific projects.

The method utilises a recursive procedure, to calculate the deterioration over the life of the pavement.

Aspects of this AASHTO method that are applicable and useful to South Africa are being included in the SAMDM revision, which is known as the South African Pavement Design Method, SAPDM.

The correct characterisation of the input required, and the availability of unbiased, properly formulated and calibrated transfer functions for different pavement materials are vitally important to the successful application of the method. The models are developed in a research environment, but the resilient and strength properties of the available materials must be characterised from available data as best as possible for each design project. Detail on material characterisation for the SAMDM 1996 can be found in Theyse et al (1995 and 1996.) The 1996 material resilient response characteristics (Young's modulus and Poisson's ratio) and damage models are currently generally used by practitioners and are provided in the following sections. It is important to always clearly state how the inputs were obtained, and provide justification for their use.

7.1.1 Hot Mix Asphalt Fatigue

(i) 1996 SAMDM

In the SAMDM, asphalt surfacing layers are only analysed for fatigue. It is assumed the cracks start at the bottom of the layer and propagate up to the surface. The structural capacity, or fatigue life, determined represents surface cracking over a defined area of the road. This area depends on the reliability assigned to the road category, for example, 95% reliability for Category A implies 5% of the road area is cracked.

Rutting, or permanent deformation in an asphalt layer has typically been considered a function of the mix properties, and has therefore not been considered in the structural analysis.

The resilient moduli for thin continuously and gap-graded asphalt surfacing layers of are shown in Table 27. These layers are generally less than 50 mm in thickness.

Table 27. Elastic Moduli for Asphalt Materials used in SAMDM 1996

Code	Depth (d) Below Surface (mm)	Modulus (MPa)
AG[1]	≤ 50	3000
BC[2]	≤ 100	4000
	100 < d ≤ 150	5000
	150 < d ≤ 200	6000
	200 < d ≤ 250	7000

Notes
1. Gap graded asphalt surfacing, as defined in TRH14 (1985)
2. Continuously graded hot mix asphalt, as defined in TRH14 (1985)

The Poisson's Ratio is generally assumed between 0.4 and 0.44, with 0.44 the recommended value.

Asphalt layers are modelled as a bound layer, which bends under the load application. This induces cracks at the bottom of the layer, which propagate up to the surface. The damage function uses the horizontal tensile strain at the bottom of the layer, which represents the resistance to the crack formation. This concept is illustrated in Figure 30, where ε_t represents the tensile strain. The transfer functions for both thin (< 50 mm) surfacing layers and thick (> 75 mm) asphalt bases are shown in Equation (18) in Table 28, with the constants applicable to the required reliability level or Road Category. For thick asphalt bases, a shift factor to account for the propagation of cracks from the bottom of the layer to the surface is also used, and is given in Equation (19) in Table 28.

HMA Fatigue Transfer Functions

It is generally understood that the 1996 SAMDM fatigue transfer functions for asphalt are not that reliable.

In South Africa, we generally use asphalt layers that are less than 50 mm thick, and failure of the asphalt layer is not necessarily a terminal condition for the pavement. The pavement can continue to carry traffic with the application of crack sealants to cracks, a seal to waterproof the layer, or patches to correct particularly weak areas. For these reasons, in an analysis of the full pavement system, the structural capacity of the asphalt layer is usually not considered in the critical layer determination.

Both Shell (Huang, 1993) and the Asphalt Institute (Austroads, 1992) have transfer functions for fatigue of asphalt. It is appropriate to use these transfer functions as an additional check for a design.

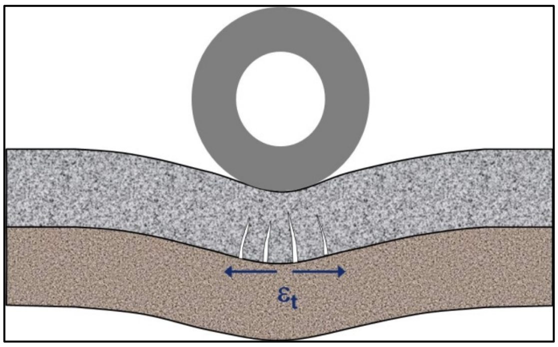


Figure 30. Critical Parameter and Location for Asphalt Layers

Table 28. Transfer Function and Constants for Asphalt Layers

$$N_f = 10^{\alpha\left(1 - \frac{\log \varepsilon_t}{\beta}\right)} \quad (18)$$

Where N_f = Fatigue life

α, β = Constants, values shown below

ε_t = Horizontal tensile strain at bottom of asphalt layer

Thin Surfacing (< 50 mm)

Reliability Level (Category)	Continuously Graded		Gap-graded	
	α	β	α	β
95% (A)	17.40	3.40	15.79	3.705
90% (B)	17.46	3.41	15.85	3.719
80% (C)	17.54	3.42	16.93	3.736
50% (D)	17.71	3.46	16.09	3.774

Thick Asphalt Bases (> 75 mm)

Reliability Level (Category)	Asphalt Stiffness (MPa)									
	1000		2000		3000		5000		8000	
	α	β	α	β	α	β	α	β	α	β
95% (A)	16.44	3.378	16.09	3.357	15.78	3.334	15.52	3.317	15.086	3.227
90% (B)	16.81	3.453	16.43	3.428	16.11	3.403	15.73	3.362	15.296	3.272
80% (C)	17.25	3.543	16.71	3.487	16.26	3.435	15.83	3.383	15.390	3.291
50% (D)	17.87	3.671	17.17	3.583	16.68	3.524	16.10	3.441	15.650	3.346

Shift Factor for Crack Propagation

If thickness of layer < 25 mm SF = 1

If thickness of layer $\geq$ 25 mm SF = 0.0489 • t – 0.2225 (19)

Where t = Layer thickness in mm

(ii) SAPDM

The revised SAMDM, known as SAPDM (South African Pavement Design Method), will include updated models for asphalt layers. These models will include analysis methods for both fatigue and permanent deformation.

At the time of writing this chapter, field testing is being performed, to obtain data with which to calibrate the potential models. Thereafter, the models most suited to South African conditions will be selected. For more on the progress on this research, see www.sapdm.co.za.

7.1.2 Unbound Granular Base and Subbase Layer Permanent Deformation

(i) 1996 SAMDM

In the 1996 SAMDM, unbound granular layers are assumed to accumulate permanent deformation, from shear deformation, in the layer. The resilient properties for unbound granular base and subbase layers are given in Table 29. The suggested ranges are shown, along with the values used in the development of TRH4 in brackets.

The stiffness in a granular layer depends on the strength of the support; the stronger the underlying layer, the stiffer the granular layer. For rehabilitation investigations, it is important to realise that wide ranges can exist for the same material, depending on the in situ state.

Table 29. Elastic Moduli for Granular Materials

Material Code	Material Description	Elastic Modulus Support Condition	
		Over Cemented	Over Granular
G1	High quality crushed stone	250 – 1000 (450)[1]	150 – 600 (300)
G2	Crushed stone	200 – 800 (400)	100 – 400 (250)
G3	Crushed stone	200 – 800 (350)	100 – 350 (250)
G4	Natural gravel (base quality)	100 – 600 (300)	75 – 350 (225)
G5	Natural gravel	50 – 400 (250)	40 – 300 (200)
G6	Natural gravel (subbase quality)	50 – 200 (225)	30 – 200 (150)
EG4	Equivalent granular, G5/G6 parent material	–	200 – 400 (300)
EG5	Equivalent granular, G7/G8 parent material	–	100 – 300 (200)
EG6	Equivalent granular, G9/G10 parent material	–	30 – 200 (140)

Note
1. Values shown in brackets were used for the development of the catalogues in TRH4 (1996).

Granular layers are analysed by determining the shear stress state in the middle of the layer, and comparing this to the shear strength, in terms of the cohesion and friction angle using the Mohr-Coulomb model. This shear strength state is known as the safety factor, and is used in the transfer function to determine the structural capacity of the layer. The damage model (transfer function) is given in Equations (20) and (21) in Table 30, along with the shear strength parameters (cohesion and friction angle) for the applicable materials classes. The transfer function calculates the structural capacity of the granular layer to a terminal condition of 20 mm of rutting in the layer.

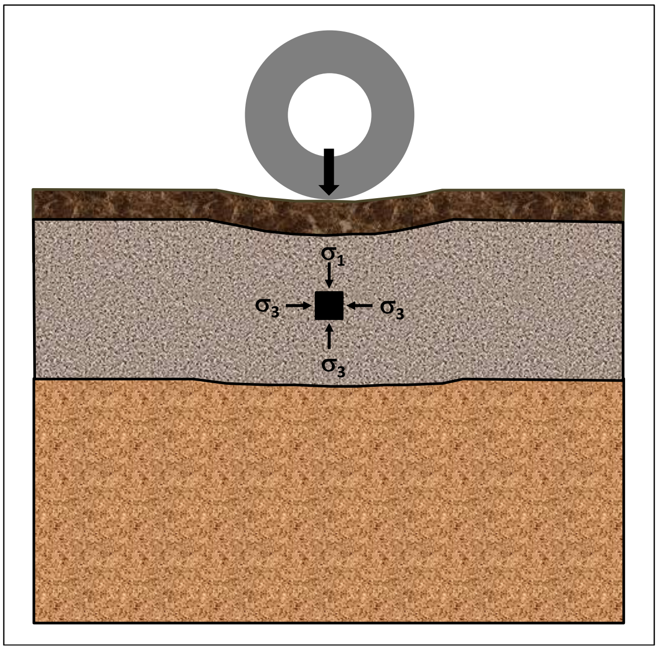


Figure 31. Critical Parameter and Location for Granular Layers

Granular Materials Transfer Functions

It is generally understood that the permanent deformation transfer functions for granular materials are on the conservative side.

The calculation of the safety factor can become quite complicated, primarily because the material is assumed to behave linear elastically. In reality, granular materials are not linear elastic materials, and they cannot take tension. This requires some adjustments to be made to the calculations. These adjustments are detailed in Theyse et al, “Overview of South African Mechanistic Pavement Design Method”, 1996.

Table 30. Transfer Function and Constants for Shear Failure (Granular Materials)

$$N = 10^{(\alpha F + \beta)} \tag{20}$$

Where N = Number of equivalent standard axles to safeguard against shear failure
α, β = Constants, values shown below
F = Stress Ratio, defined in Equation (21)

$$F = \frac{\sigma_3 \left[K\left(\tan^2\left(45 + \frac{\phi}{2}\right) - 1\right)\right] + 2\,K\,C\tan\left(45 + \frac{\phi}{2}\right)}{(\sigma_1 - \sigma_3)} \tag{21}$$

$$F = \frac{\sigma_3 \phi_{term} + C_{term}}{(\sigma_1 - \sigma_3)}$$

Where σ_1, σ_3 = Major and minor principle stresses acting in the middle of the granular layer (compressive stress positive)[1]
C = Cohesion
ϕ = Angle of Internal Friction
C_{term} = Values given below for material codes
ϕ_{term} = Values given below for material codes
K = Constant for moisture
- 0.65 for saturated (wet)
- 0.8 for moderate
- 0.95 for normal

Constants for Equation (20)

Reliability Level	α	β
95% (Category A)	2.605122	3.480098
90% (Category B)	2.605122	3707667
80% (Category C)	2.605122	3.983324
50% (Category D)	2.605122	4.510819

C and Ø-Terms for Equation (21)

Material Code	Moisture Condition					
	Dry		Moderate		Wet	
	Ø-term	C-term	Ø-term	C-term	Ø-term	C-term
G1	8.61	392	7.03	282	5.44	171
G2	7.06	303	5.76	221	4.46	139
G3	6.22	261	5.08	188	3.93	115
G4	5.50	223	4.40	160	3.47	109
G5	3.60	143	3.30	115	3.17	83
G6	2.88	103	2.32	84	1.76	64
EG4[2]	4.02	140	3.50	120	3.12	100
EG5[3]	3.37	120	2.80	100	2.06	80
EG6[4]	1.63	100	1.50	80	1.40	60

Notes
1. If a tensile stress, i.e., a negative σ_3 is calculated, σ_3 is reset to zero and σ_1 is increased by the value of σ_3. The net result is that $(\sigma_1 - \sigma_3)$ remains the same, and $\sigma_3 = 0$.
2. EG4: broken down cemented material, G5/G6 parent material
3. EG5: broken down cemented material, G7/G8 parent material
4. EG6: broken down cemented material, G9/G10 parent material

This 1996 SAMDM granular materials transfer functions are the most commonly used when implementing the SAMDM.

(ii) Wolff Model (1993)

An alternative damage model for permanent deformation in crushed and natural granular layers (G1 to G6) for use in the SAMDM was developed by Wolff (1993, 1994). The model relates the stress state in a granular layer of a specific strength to the permanent deformation caused by repetitions of that stress state. The model was developed from Heavy Vehicle Simulator (HVS) testing over a period of approximately ten years. The HVS tests indicated that granular materials do not behave as a fully elastic material, but rather as an elasto-plastic material. An elastic and plastic deformation component exists for each load application. The cumulative plastic deformation caused by the load repetitions appears as a rut at the top of the granular layer. Therefore, the sum of the principal stresses calculated in the centre of a granular layer using non-linear elastic theory defines the stress state. The model allows for the calculation of the permanent deformation in each granular layer of a specific strength for any number of load repetitions. The structural capacity of a pavement structure can thus be determined by limiting the sum of the permanent deformation calculated in each pavement layer to the maximum permanent deformation allowable on the surface of the pavement (10 or 20 mm depending on the road category). This concept makes the approach very attractive for use in rehabilitation design, where measured rut levels can be effectively used to realistically calibrate the system.

An example of the function relating permanent deformation to stress state and stress repetitions is shown in Figure 32 for a G6 material at 90 percent design reliability.

While this model was developed using sound engineering, it is not widely used in practice. This is primarily because it has only been incorporated into the Cyrano software package, and not into other software packages.

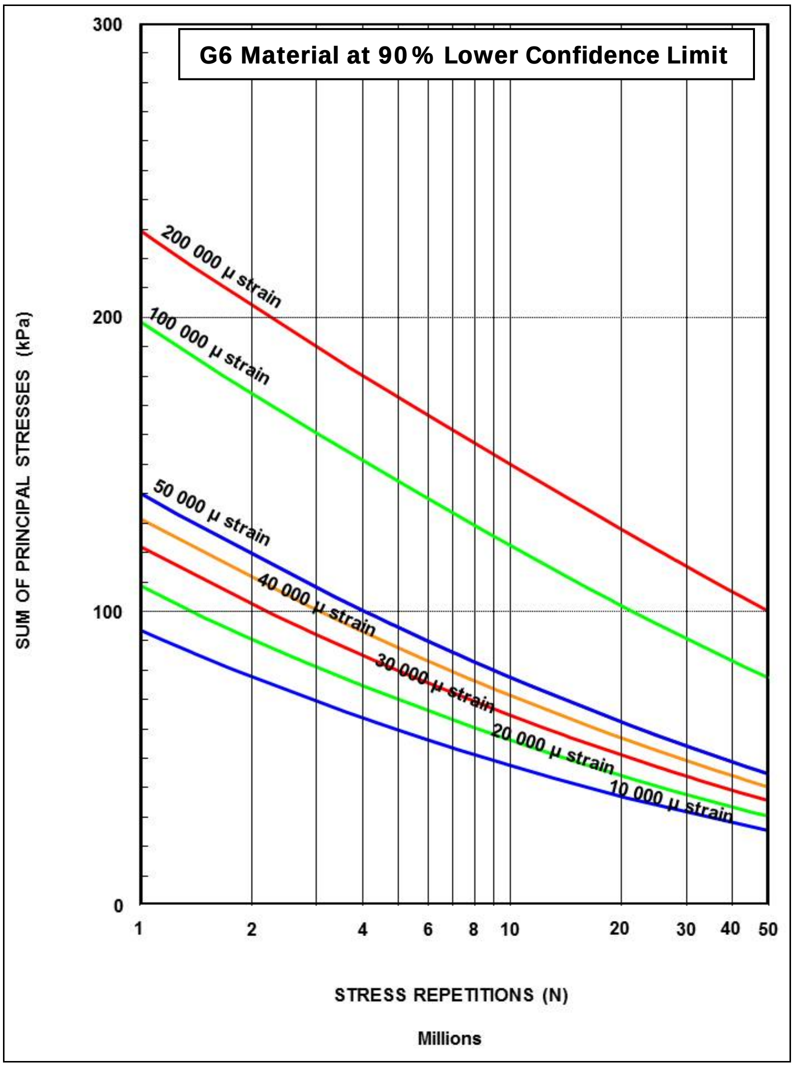


Figure 32. Relationship between Stress, Permanent Deformation (Strain) and Repetitions

(iii) SAPDM

Theyse (2008b) has recommended shear strength properties and transfer functions for unbound granular materials for use in the SAPDM. These properties are listed in Table 31. A notable change is that G5 and G6 subbase quality material are subdivided according to coarse and fine material, and the product of the Bar Linear Shrinkage (BLS) and the percentage passing the 0.425 sieve. Coarse subbase materials showed a significant reduction in friction angle under wet conditions when the product of the BLS and the -0.425 fraction exceeds 170.

The updated transfer function still looks at the shear behaviour of the granular layer, but uses the stress ratio rather than the safety factor. The stress ratio is conceptually the inverse of the safety factor. The model provides for subgrade deformation levels from 1 to 20 mm. Details of the models, and how they are implemented, are provided in Theyse et al, 2011. The formulation of these models may change for the final SAPDM, expected in 2013.

Table 31. Revised Shear Strength Properties for Unbound Granular Materials (2008b)

Application	Material Code	Saturation Level[1]	Shear Strength Parameters	
			Cohesion (kPa)	Friction Angle (°)
Base	**G1**	Dry	90 – 130	53 – 57
		Moderate	75 – 100	51 – 55
		Wet	50 – 75	50 – 53
	G2	Dry	100 – 125	54
		Moderate	50	52
		Wet	45	50
	G3	Dry	75	51
		Moderate	40	51
		Wet	20	50
	G4	Dry	75	51
		Moderate	40	47
		Wet	20	45
Subbase Coarse Materials GM: 1.7–2.3 Max size: 26.5 – 37.5	**G5/6** (BLS425[2]< 170)	Dry	100 – 125	45 – 49
		Moderate	50 – 100	41 – 45
		Wet	10 – 50	39 – 42
	G5/6 (BLS425 > 170)	Dry	225 – 275	45 – 49
		Moderate	50 – 100	41 – 44
		Wet	25 – 35	31 – 33
Subbase Fine material GM: 1.5–1.6 Max size: < 13.2	**G5/6** (BLS425 < 100)	Dry	125 – 250	43 – 45
		Moderate	40 – 50	43 – 45
		Wet	10 – 25	40 – 43
Selected Subgrade	**G7** Ferricrete	Dry	75 – 100	43 – 45
		Moderate	0	41 – 43
		Wet	0	40 – 41
	G7 Sand	Dry	75 – 100	35 – 37
		Moderate	0	37 – 40
		Wet	0	35 – 37
In Situ Subgrade	**G10** Ferricrete	Dry	75	45
		Moderate	0	37
		Wet	–	–
	Silt	Dry	80 – 100	30 – 35
		Moderate	0	30 – 35
		Wet	0	30 – 35
	Clay	Dry	> 250	20 – 30
		Moderate	25 – 50	15 – 25
		Wet	5 – 25	10 – 15

Notes:
1. Dry = 20% Moisture
   Moderate = 50% Moisture
   Wet = 80% Moisture
2. BLS425 = Product of the bar linear shrinkage and percentage passing the 0.425 mm sieve

7.1.3 Cement Stabilised Base and Subbase Layers

(i) 1996 SAMDM

Cement stabilised layers are analysed as bound layers for effective fatigue, and for crushing of the material at the top of the layer. It is assum ed that the cracks start at the bottom of the layer and propagate to the top of the layer, with cracks in thicker layers taking longer to propagate. The terminal condition is when the material has cracked or weakened to an extent that it has a similar effective stiffness to an unbound granular layer. This is known as the equivalent granular state. It is important to note that the "cracked" state does not imply the material is the same consistency as a granular material, or that it has visibly cracked into smaller, granular like pieces. The cracks are generally micro-cracks that are not that visible. The stiffness of the layer is, however, reduced. The term "effective fatigue" is used to suggest that the typical fatigue cracking, such as with asphalt layers, is not expected. Once a cemented material has reached the end of its effective fatigue life, is enters into a new phase wherein it behaves as an equivalent granular layer. The parameter used to calculate the effective fatigue life is the horizontal tensile strain at the bottom of the layer, shown in Figure 33.

Equivalent Granular State

Cemented layers are assumed to crack, starting at the bottom of the layer and propagating to the top of the layer, with cracks in thicker layers taking longer to propagate. The terminal condition is when the material has cracked or weakened to an extent that it has a similar effective stiffness to an unbound granular layer. This is known as the **equivalent granular state**. It is important to note that the "cracked" state does not imply the material is the same consistency as a granular material, or that it has visibly cracked into smaller, granular like pieces. The cracks are generally micro-cracks that are not that visible, but results in a loss of stiffness.

Inverted Pavements

An inverted pavement is when the base layer is a high quality granular layer, and the subbase a cement stabilised layer. A thin asphalt layer or seal provides the surfacing. The term "inverted" is used because the strength of the pavement does not decrease with pavement depth, because of the stiff cemented layer.

The idea behind an inverted pavement is that the cemented layer provides an anvil against which the granular base can be well compacted. This achieves a high quality, dense base.

Inverted or "upside-down" pavement structures are commonly used in South Africa, and are included in the TRH4 catalogues.

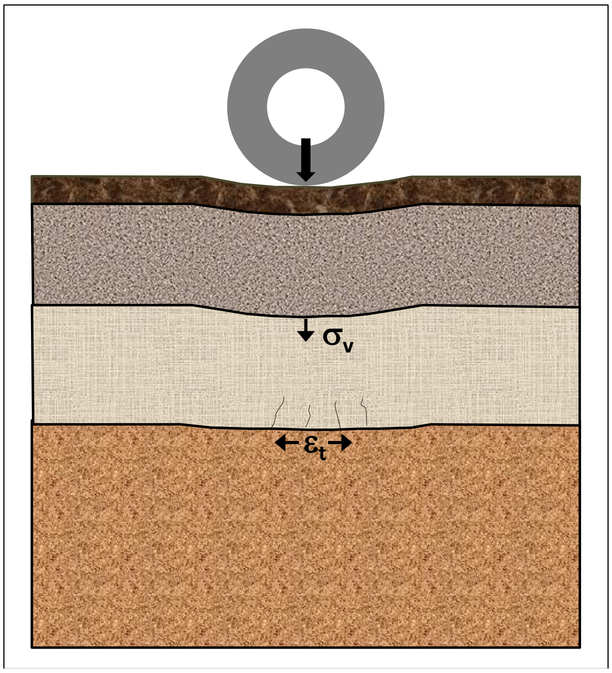


Figure 33. Critical Parameters and Locations for Lightly Cemented Layers

When the cemented layer has reached the equivalent granular state, if is analysed in a second phase using the granular materials transfer function, with the materials properties for the equivalent granular state shown in Table 30. The combined life of the layer is calculated from the life of both phases, as discussed in Section 7.1.6.2. Theyse et al (1996) provides complete details.

Cemented layers are also analysed for crushing at the top of the layer, using the vertical compressive stress as shown in Figure 33. This is particularly relevant for base layers. Crushing is not considered a terminal condition, and hence it is not used in the critical layer calculation. However, it is an important check, as any crushing has a significant impact on the surfacing.

The resilient properties (elastic moduli) for cemented materials are shown in Table 32, along with recommended material properties for the damage models. The damage models for crush initiation, advanced crushing and effective fatigue are given in Equations (22) and (23) in Table 33. The relationship to account for layer thickness and the time for crack propagation, used with the effective fatigue transfer function, is also given in Equation (24) in Table 33. Note that C1 and C2 cemented layers are not commonly used anymore, as the high cement content results in significant cracking and consequent reflection cracking.

Table 32. Elastic Moduli and Material Properties for Cemented Materials in 1996 SAMDM

Initial Class	Modulus (MPa)	Strain-at-Break (ε_b)	UCS (kPa)	Equivalent Granular Class	Modulus (MPa)
C3	2 000	125	2250	EG4	300
C4	1 500	145	1125	EG5	200

Table 33. Transfer Functions for Cemented Materials

Crush Initiation and Advanced Crushing

$$N_{ci/ca} = 10^{a\left(1-\frac{\sigma_v}{b\,UCS}\right)} \qquad (22)$$

where
$N_{ci/ca}$ = Standard axles to crack initiation or advanced crushing
σ_v = Vertical compressive stress at top of layer
UCS = Unconfined compressive strength (kPa), recommended values in Table 32
a, b = Constants, given below

Effective Fatigue

$$N_{eff} = SF \bullet 10^{c\left(1-\frac{\varepsilon}{d\,\varepsilon_b}\right)} \qquad (23)$$

where
N_{eff} = Effective fatigue life
ε = Horizontal tensile strain at bottom of layer (microstrain)
ε_b = Strain-at-break, recommended values in in Table 32
c, d = Constants, given below
SF = Shift Factor for crack propagation (see below)
t = Layer thickness

Shift Factor (SF) to Account for Layer Thickness (t) (24)

Thickness	Shift Factor
< 102 mm	1
102 mm to 319 mm	$10^{(0.00285t - 0.293)}$
> 419 mm	8

Constants

Reliability Level	Crush Initiation		Advanced Crushing		Effective Fatigue	
	a	b	a	b	c	d
50% (Category D)	8.216	1.21	8.894	1.31	7.06	7.86
80% (Category C)	7.706	1.31	8.384	1.23	6.87	7.66
90% (Category B)	7.506	1.10	8.184	1.20	6.84	7.63
95% (Category A)	7.386	1.09	8.064	1.19	6.72	7.49

(ii) SAPDM

Recent research by Theyse has suggested new input parameters for cement stabilised materials. These input parameters are used with the existing transfer functions (Equation (20) in Table 30 and Equation (23) in Table 33). Refer to Theyse (2008b), for further information. These new inputs are not yet commonly used.

7.1.4 Subgrade Permanent Deformation

(i) 1996 SAMDM

The subgrade, and lower selected layers are analysed for permanent deformation in the layer, which manifests as permanent deformation, or wide rutting, on the pavement surface. The rutting is calculated using the vertical compressive strain at the top of the layer, as shown in Figure 34. Transfer functions are available for both 10 and 20 mm of rutting in the layer. The 20 mm transfer function is conservative, in that if the subgrade or selected layer has 20 mm of rutting, then the rutting at the surface of the pavement is likely to be considerable higher than the typically 20 mm terminal rut depth. Therefore, the 10 mm transfer function is more commonly used, particularly for Category A and B roads. The recommended resilient moduli are shown in Table 34, and the damage model in Equation (25) in Table 35.

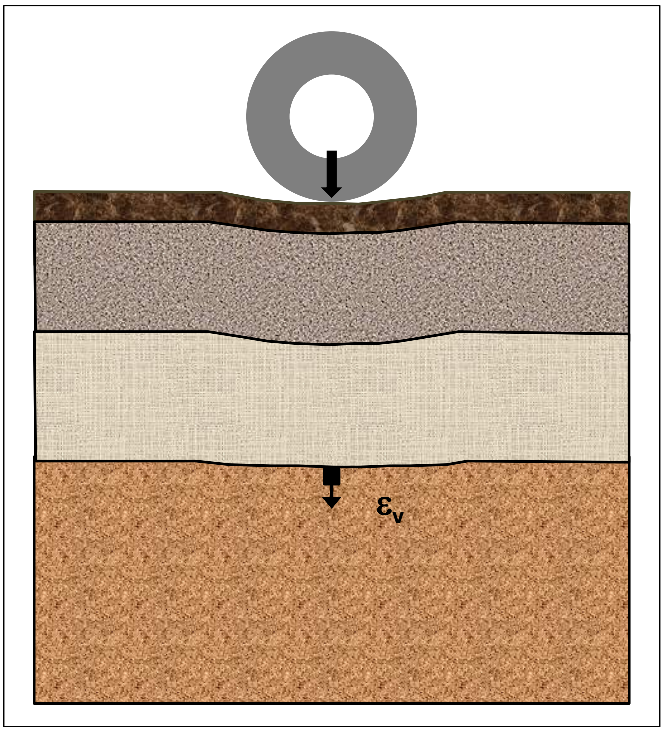


Figure 34. Critical Parameter and Location for Selected and Subgrade Layers

Table 34. Elastic Moduli (MPa) of Subgrade Materials for the 1996 SAMDM

Selected Layers and Subgrade Material Classes	Elastic Moduli (MPa)
G7	120
G8	90
G9	70
G10	45

(ii) SAPDM

SAPDM uses new models for subgrade permanent deformation. These are described in a 2011 CAPSA paper by Theyse et al, "Interim Revision of the South African Mechanistic-Empirical Pavement Design Method for Flexible Pavements" and Theyse, 2008a. These models use the deflection at the top of the subgrade layer to predict the permanent deformation, as this was found to be a better predictor than vertical compressive strain. New transfer functions using deflection have been developed. These new models have not yet been widely implemented.

Table 35. Transfer Functions for Subgrade and Selected Layer Permanent Deformation

Permanent Deformation

$$N_{PD} = 10^{(a - 10\log \varepsilon_v)} \quad (25)$$

where N_{PD} = Standard axles to set level of permanent deformation
ε_v = Vertical compressive strain at top of layer
a = Constant, given below

Constants

Reliability Level	A	
	10 mm Terminal Rut Depth	**20 mm Terminal Rut Depth**
95% (Category A)	33.70	36.70
90% (Category B)	33.47	36.47
80% (Category C)	33.38	36.38
50% (Category D)	33.30	36.30

7.1.5 Analysis Positions for Critical Stress/Strain Parameters

The location in the pavement structure at which the critical parameters are calculated depends on the material type and the assumed mode of failure. Figure 35 summarises the critical parameters and locations used for a typical South African pavement structure, as discussed in the previous sections. Note that generally a half-axle load is assumed, because the other side of the axle is out of the zone of influence. The critical parameters are calculated both underneath and in-between the loads. This is because the deeper into the pavement, the more the two zones of loading influence, represented by the "cones" Figure 35, interact. Standard practice is to calculate the critical parameter under and between the wheels, and then use the value which produces the shortest structural capacity.

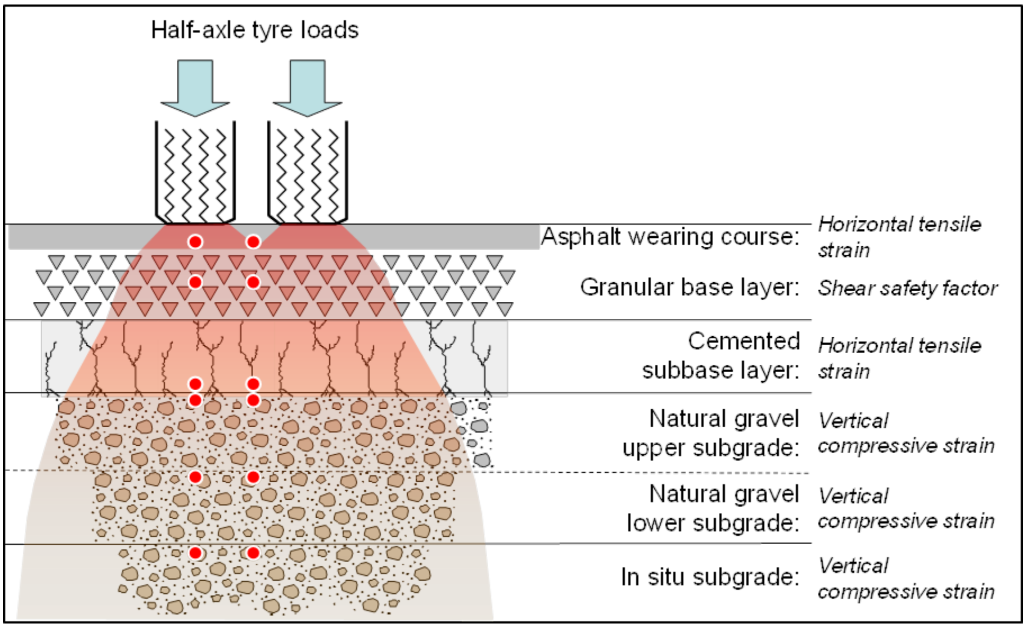


Figure 35. Analysis Positions for Critical Parameters

7.1.6 General Comments on SAMDM 1996

The SAMDM is a powerful design tool. However, it does have some weaknesses, which must be considered when using the method. There are also some important tips and checks useful for using the method. This section attempts to discuss some of these aspects.

7.1.6.1 Standard Axle and Tyre Pressure

The SAMDM is generally used with standard axle loading. See Sections 3.5, 3.5.2 and 4.1.3 for definitions and discussion on standard axles. The catalogues in TRH4 were developed using standard axle loading, with a 520 kPa tyre pressure. However, since those analyses in 1995, tyre pressures in South Africa have risen, and 520 kPa is no longer representative of current loading. Theyse (2011) recommends a 650 kPa tyre pressure. A pressure of 750 kPa is widely used to account for current loading. These higher values affect the results, which should be noted when comparing designs to the TRH4 catalogues.

7.1.6.2 Critical Layer and Structural Capacity Determination

The SAMDM works by determining the structural capacity of each layer in the pavement. The structural capacity of the pavement is determined by the layer with the shortest life, termed the critical layer. The exception is when thin asphalt layers are not considered in the determination of the critical layer, as discussed in Section 7.1.1.

For pavements that include cement stabilised layers, the analysis is done in two phases. The first phase is the effective fatigue phase, and in this the cemented layer is analysed as a cemented layer, with a high resilient modulus used as input. The structural capacity of this phase is determined by the effective fatigue of the cemented layer. At the end of the first phase, the cemented layer is assumed to be in an equivalent granular state. The layer is not in a terminal condition. In the second phase, the cemented layer is now analysed as an equivalent granular material. The structural capacity of the pavement is a combination of the first and second phase. Should two cemented layers be present, the pavement is likely to need analysis in three phases. Details on how to calculate the combined structural capacity from the phases are included in Theyse, 1996.

7.1.6.3 Method Sensitivity

The 1996 SAMDM is very sensitive to the input values, and large differences in the structural capacity estimates are found with small changes in the inputs. To prevent inaccurate and inappropriate results, it is important for the following issues to be taken note of:

- The **input values** used should be realistic for the available materials. It is highly recommended that the input values used are similar to those that are suggested for the applicable materials (see Table 27, Table 29, Table 32 and Table 34). In addition, it is important that, especially for rehabilitation design, the available materials information is used to verify the design inputs. The material classification system for design discussed in Chapter 9, Section 15 is useful for deciding appropriate material classes for existing pavement layers.
- The SAMDM can give **inappropriate answers**. It is important to check the results with some common sense "*does it pass the test of reasonableness?*", and against other methods including the catalogues in TRH4. A CAPSA 2004 paper by Jooste, illustrates some weaknesses of the method.
- The method must be used in the way it was **developed and calibrated**. For example, the method specifies that that granular shear transfer function calculates the safety factor in the middle of the layer. The middle of the layer may not be the point at which the safety factor is the highest, but the transfer functions were validated for the calculation in the middle of the layer, and the answers are inappropriate if another position is used.

Advantages of the SAMDM

- Suited to new and rehabilitation design
- Evaluates the adequacy of individual layers and the pavement system
- Calibrated for South African conditions and materials
- Accommodates different pavement types and pavement compositions
- Accommodates changes in operating conditions, such as axle loads

Disadvantages of the SAMDM

- Developed for new pavement design, and adapted to rehabilitation design with difficulty
- Perceived to be biased towards certain pavement types
- Overly sensitive to small variation in input
- Input parameters not well related to routine engineering parameters
- Damage models outdated
- Relatively complex, only suited to computer application
- Inconsistent results, stronger and thicker layers (especially subbase) do not always lead to an increase in structural capacity

7.2 Pavement Number (PN) Design Method

The Pavement Number (PN) design method was developed as part of TG2 (2009) for the design of bituminous stabilised layers in pavements. The method is, however, also applicable to granular and cemented materials. The method is described in detail in Appendix C of TG2 and the background development by Jooste (2009). Although the method is relative new, it is already being widely used in South Africa.

The PN method is applicable to Category A and B roads where the design traffic is between 1 and 30 MESA. The method can be used in the initial and detailed assessment phase in the design of new and rehabilitated pavements.

The PN design method is a knowledge-based approach. It is based on the Structural Number concept, as used in the AASHTO method (Section 7.4). However, some of the shortcomings of the Structural Number have been overcome in the PN method. The PN method has the following advantages:

- Data from **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.

> **PN Method**
>
> The PN Method is described in detail in TG2 (2009) in Appendix C. The method is relatively new and although it has been well validated, it may be necessary from time to time to make changes to improve the method. Any such changes will be published on www.asphaltacademy.co.za/bitstab, and registered users will be notified.

This method relies on basic rules-of-thumb, which reflect well-established principles of pavement behaviour and performance, and ensure an appropriate pavement design solution in most situations. The concepts in the rules-of-thumb are quantified into specific rules with constants or functions associated with each rule. The rules-of-thumb are briefly described in the following sections.

The constants shown for the PN method included here and in Appendix C of TG2 were the values used at the time of publication of TG2. Although these values are well validated it may be necessary from time to time to make changes to improve the system. If changes are made, the modified values will be reflected on www.asphaltacademy.co.za/bitstab. It is therefore recommended that before commencing a Pavement Number calculation, the website is checked for any changes in values or tests.

The PN method is designed to be used in conjunction with the material classification system described in Chapter 9, Section 15 and in TG2, Chapter 3 and Appendix A. Software is also available on www.asphaltacademy.co.za/bitstab. However, it is simple to do the calculations in a spreadsheet.

7.2.1 Applicability of Pavement Number Method

Before the Pavement Number method is used, the designer must check that the following situations do not apply:

- **Design traffic greater than 30 MESA:** The method was calibrated using a knowledge base which was limited to pavements that had accommodated less than 30 MESA. Thus, if the design traffic exceeds 30 MESA, the PN method is inappropriate as it has not been validated at these high traffic levels.
- **Presence of thin, weak lenses:** If thin, weak lenses of material exist below the surfacing, or between stabilised layers, especially within the upper 400 mm, then zones of high slip and shear develop, and the PN calculations do not apply. In such situations, the structural capacity assessment of the PN method is not appropriate, and special treatment of the affected weak lens must be undertaken.
- **Subgrade CBR less than 3%:** The knowledge base on which the PN method was calibrated did not include any pavements that had a subgrade CBR less than 3%. The PN method should therefore not be used in cases where the subgrade CBR is less than 3% at a depth 600 mm below the surface.

7.2.2 Rules of Thumb / Departure Points

The rules-of-thumb underlying the method reflect well established principles of pavement behaviour and performance. The following rules-of-thumb are used:

- **Pavement system in general**
  - The structural capacity of a pavement is a function of the combined long term load spreading potential of the pavement layers and the relative quality of the subgrade.
  - The relative quality and stiffness of the subgrade is the departure point for design, as the subgrade is a key determinant in the overall pavement behaviour and performance.

  - For pavements with thin surfacings, the base layer is the most critical component, and failure in this layer effectively constitutes pavement failure.
- **Specific pavement layers**
  - The load spreading potential of an individual layer is a product of its thickness and its effective long term stiffness under loading.
  - The Effective Long Term Stiffness (ELTS) of a layer depends on the material type and class, and on its placement in the pavement system.
  - 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 generally soften with decreased cover thickness.
  - Coarse-grained, unbound layers 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 increases with increasing support stiffness, by means of the modular ratio limit, up to a maximum stiffness, determined mainly by the material quality.
  - BSMs are assumed to act in a similar way to coarse granular materials, but with a higher cohesive strength. However, owing to the higher cohesive strength in bituminous stabilised materials, these layers are less sensitive to the support stiffness than unbound granular materials and can therefore sustain higher modular ratio limits. If the cement content of a BSM mix exceeds 1 percent, then the material is assumed to behave as a cemented material.

These rules-of-thumb introduce several concepts such as the ELTS, modular ratio limit, maximum stiffness and stress-stiffening behaviour, which are briefly described in the following sections.

(i) The Effective Long Term Stiffness (ELTS)

The ELTS is a model parameter which serves as a relative indicator of the average long term in situ stiffness of a pavement layer. As such, the ELTS averages out the effects of decreasing stiffness owing to traffic related deterioration, as well as seasonal variations in stiffness. Thus, the ELTS does not represent the stiffness of a material at any specific time.

The ELTS is also not a stiffness value determined by means of a laboratory or field test. It is a model parameter, calibrated for use in the PN design method. It may therefore differ from stiffness values typically associated with material classes.

(ii) Modelling of Subgrade Materials

Characterization of the support is critical to the pavement design of all pavements. For new construction, the TRH4 procedure for delineation of the in situ subgrade and for importing selected subgrade material, if necessary when the structural strength of the in situ subgrade is insufficient, applies to the PN method. For rehabilitation projects, the guidelines in TRH12 for evaluating and designing for changing support conditions should be followed.

The first step in the calculation of the PN-value is the determination of the subgrade material class using the Material Classification System for Design described in Chapter 9, Section 15 and Appendix C of TG2. Once the subgrade class has been determined, the ELTS for the subgrade is calculated. This involves the following steps:

- Assignment of an **ELTS** based on the materials class. The values are given in Table 36.
- Adjustment of the ELTS for different **climatic regions** (wet, dry or moderate), according to Table 37.
- Adjustment of the stiffness determined to take account of depth of **subgrade cover** using the chart or equation shown in Figure 36.

Table 36. Stiffness Determination for the Subgrade

Design Equivalent Material Class for Subgrade	Stiffness Value (MPa)
G6 or better	180
G7	140
G8	100
G9	90
G10	70

Note:
Subgrade stiffness value should be adjusted for climate (Table 37) and cover depth (Figure 36).

Table 37. Climate Adjustment Factors

Climate and Weinert N Values (after TRH4, 1996)	Adjustment Factor
Wet (Weinert N < 2)	0.6
Moderate (Weinert N = 2 to 5)	0.9
Dry (Weinert N > 5)	1.0

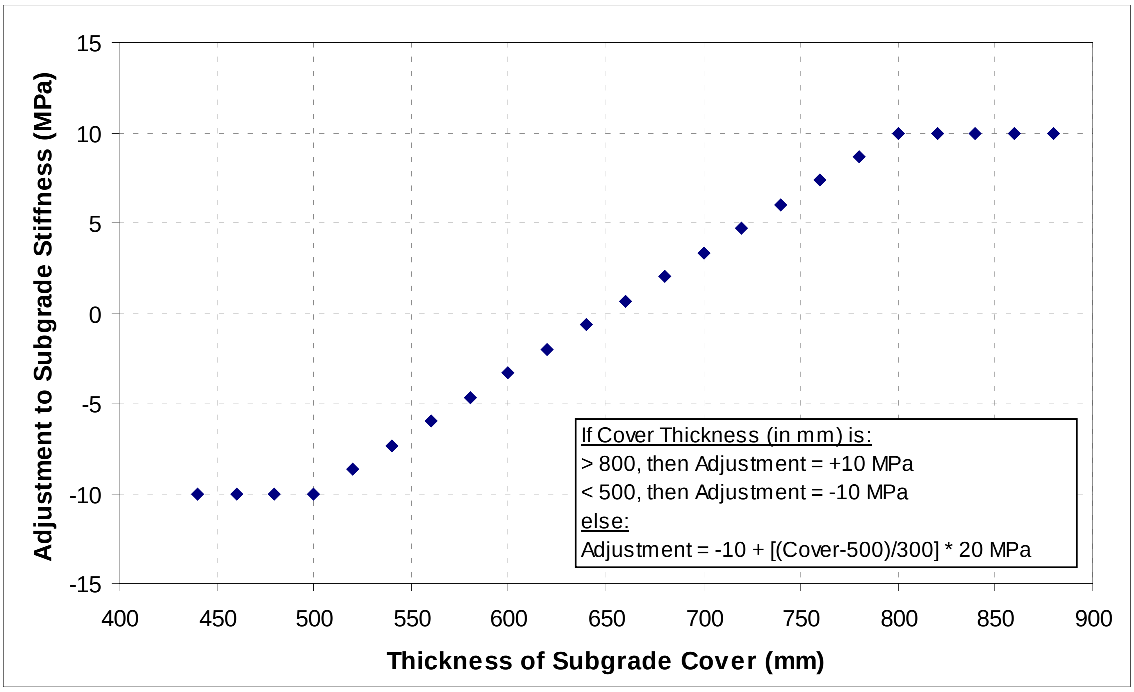


Figure 36. Adjustment of Subgrade Stiffness Based on Cover Thickness

(iii) The Modular Ratio Limit and Maximum Stiffness

The modular ratio is defined as the ratio of a layer's stiffness relative to the stiffness of the layer below it. Thus, if the stiffness of a base layer is 300 MPa, and the stiffness of the support below it is 200 MPa, then the modular ratio of the base layer is 1.5.

The modular ratio accounts for the stress-sensitive stiffness of granular materials, and, albeit to a lesser extent, BSM materials. The stress-sensitivity causes the stiffness of the material to decrease when the material is placed over a weaker support. This decrease in stiffness occurs where the support layer is soft, causing a tendency for the overlying layers to bend into the support, thereby increasing the likelihood of higher shear and tensile forces in the overlying layers. This effect limits the stiffness obtained in the layer. By placing a limit on the modular ratio that can be sustained for a specific material, it is ensured that the stiffness value assumed for that layer is realistic, given the material quality and stiffness of the support. In essence, the concept of a limiting modular ratio for materials ensures that stress-sensitive stiffness behaviour is implicitly taken into account.

The modular ratio that a material can sustain varies over the life of a pavement, and in the PN method it pertains to the overall long term stiffness that a material can maintain. Modular ratio values for use in the PN method are given in Table 38.

Modular Ratio

The modular ratio is the ratio of a layer's stiffness relative to the stiffness of the layer below it. Thus, if the stiffness of a base layer is 300 MPa, and the stiffness of the support below it is 200 MPa, then the modular ratio of the base layer is 1.5.

Table 38. Modular Ratio Limit, Maximum Allowed Stiffness and Base Confident Factor for Pavement Layers

General Material Description	Material Class[1]	Modular Ratio Limit	Maximum Allowed Stiffness (MPa)	Base Confidence Factor
Hot mix asphalt (HMA) surfacing and base material	AG, AC, AS, AO	5.0	2 500	1.0
Surface seals	S1, S2, S3, S4, S5, S6	2.0	800	N/A
High strength bitumen stabilised material, normally using crushed stone or reclaimed asphalt (RA) source material	BSM1	3.0	600	1.0
Medium strength bitumen stabilised material, normally using natural gravel or RA source material	BSM2	2.0	450	0.7
Crushed stone material	G1	2.0	700	1.1
	G2	1.9	500	0.8
	G3	1.8	400	0.7
Natural Gravel	G4	1.8	375	0.2
	G5	1.8	320	0.1
	G6	1.8	180	-2.0
Gravel-soil blend	G7	1.7	140	-2.5
	G8	1.6	100	-3.0
	G9	1.4	90	-4.0
	G10	1.2	70	-5.0
Cement stabilised crushed stone	C1 and C2	9	1500	0.8
Cement stabilised natural gravel	C3	4	550	0.6
	C4	3	400	0.4

Note:
1. Design equivalent material class for rehabilitation projects (see Chapter 9, Section 15).

(iv) Maximum Stiffness

Under the action of loading, there is a maximum stiffness that materials achieve. As with the modular ratio, the maximum stiffness depends on the quality of the material. Less dense and angular materials do not develop very high stiffnesses under loading, regardless of the stiffness of the support. The maximum allowed stiffnesses for use in the PN are given in Table 38.

In the PN model, the modular ratio limit and the maximum allowed stiffness are used to determine ELTS values. These parameters are used in the following way:

- The **stiffness of the supporting** layer is first determined. Thus the PN calculation process starts from the subgrade and proceeds upward towards the surfacing.
- The **modular ratio limit** and **maximum allowed stiffness** for each layer are determined based on the material type and class, as determined by the material classification system for design.
- The **ELTS** for a layer is determined as the minimum of the support stiffness multiplied by the modular ratio limit and the maximum allowed layer stiffness.

(v) The Base Confidence Factor

The type of material in the base layer is an important determinant of the performance of the pavement because the base is the main load bearing element and failure of the base effectively constitutes pavement failure. Experience has shown that there is a limit on the types of base materials considered for any given traffic situation. In particular, suitable design options are significantly limited as the design traffic increases.

In the PN method, the appropriateness of the base material is controlled by the Base Confidence Factor (BCF), which is used to adjust the layer contribution to the PN. Table 38 contains BCF values.

(vi) Thickness Adjustment for Cemented Layers

For cemented layers, the layer contribution to the PN is adjusted for thickness to account for crack propagation.

7.2.3 Pavement Number Calculation

Appendix C of TG2 contains the full details of the PN calculation along with a worked example. The main steps are summarized below. In a pavement design situation, the steps described are applied for each uniform design section. For rehabilitation design situations, it is thus presumed that the designer has detailed information on the existing pavement layer properties for each uniform section.

Step 1: Check to ensure that the **design method is applicable** for the design situation.

Step 2: Determine the **layer thicknesses**, and available **material properties** for each layer. Determine the design equivalent material class (DEMAC) using the guidelines in Chapter 9, Section 15 or Chapter 3 and Appendix A of TG2. To prevent the use of unrealistic layer thicknesses, maximum and minimum limits are given. Values outside these limits have not been validated.

Step 3: Combine layers with similar properties to obtain a **five layer pavement system**, including the subgrade (i.e., four layers plus the subgrade). Check that the layer thicknesses do not exceed the maximum for design purposes. Guidelines are given in TG2 on increasing or decreasing to 5 layers.

Step 4: Determine the **ELTS of the subgrade** by means of the given values. Adjust the stiffness for the climatic region and depth of subgrade cover.

Step 5: For each layer above the subgrade, determine the **modular ratio limit** and **maximum allowed stiffness**.

Step 6: Use the modular ratio limit and maximum allowed stiffness to determine the **ELTS for each layer** by working up from the subgrade.

Step 7: For the base layer, determine the **Base Confidence Factor** (BCF).

Step 8: For each layer, calculate the **layer contribution to the PN** using the ELTS, layer thickness and BCF (for base layers).

Step 9: **Add the layer contributions** for each layer to get the PN.

Inputs in the PN Method

The values used for the ELTS, modular ratio, layer thickness limits and BCF are specific to the PN method and **should not** be adjusted by the designer.

The values used for the ELTS, modular ratio, layer thickness limits and BCF are specific to the PN method and should not be adjusted by the designer.

7.2.4 Pavement Capacity Calculation

The calculation of the pavement capacity depends on the Pavement Number and the Road Category. The relationship in Equation (26) is used, in conjunction with the constants in Table 39. The relationship between the PN and the pavement capacity does not give a pavement life prediction, but rather provides a lower limit for which the pavement should carry the desired traffic sufficiently. Figure 37 shows the criteria in a graphical format.

The criteria are only applicable to Category A and B roads and for design capacities between 1 and 30 MESA. For Category C and D roads, a catalogue of design is recommended, see TG2.

7.2.5 Comment on the PN Method

The PN method was developed from data from in service pavements and from the TRH4 catalogues. None of the pavement structures available had asphalt surfacing layers thicker than 50 mm. Experience with the PN number is showing that when an asphalt surfacing thicker than 50 mm is used, the asphalt layer contribution to the PN seems unreasonably high, resulting in an unconservative pavement life. Until data from pavements with asphalt layers thicker than 50 mm can be obtained, it is not advised to use the PN method for such thick asphalt layers. In the interim, to obtain reasonable answers, limit the asphalt thickness to 50 mm or less. With thicknesses less than 50 mm, the layer contributions from all the layers seem to balance out to provide reasonable answers.

Table 39. Pavement Capacity Calculation

$$N_{allow} = N_1 + (PN - PN_1) \cdot slope \quad (26)$$

where
N_{allow} = Allowed pavement capacity (MESA)
N_1 = Lower limit for the capacity range
PN = Calculated pavement number
PN_1 = Lower limit for the PN range
Slope = Slope for the PN range

PN Range	N1	PN1	Slope
Category A			
PN < 15	Less than 3 MESA, not suited for Category A roads		
15 < PN ≤ 23	3	15	0.00
23 < PN ≤ 25	3	23	3.50
25 < PN ≤ 32	10	25	0.00
32 < PN ≤ 35	10	32	6.67
PN > 35	30	35	0.00
Category B			
PN < 3	Less than 1 MESA, use Design Catalogue		
3 < PN ≤ 8	1	3	0.00
8 < PN ≤ 11	1	8	0.67
11 < PN ≤ 15	3	11	0.00
15 < PN ≤ 25	3	15	0.70
PN > 25	Use Category A Criteria		

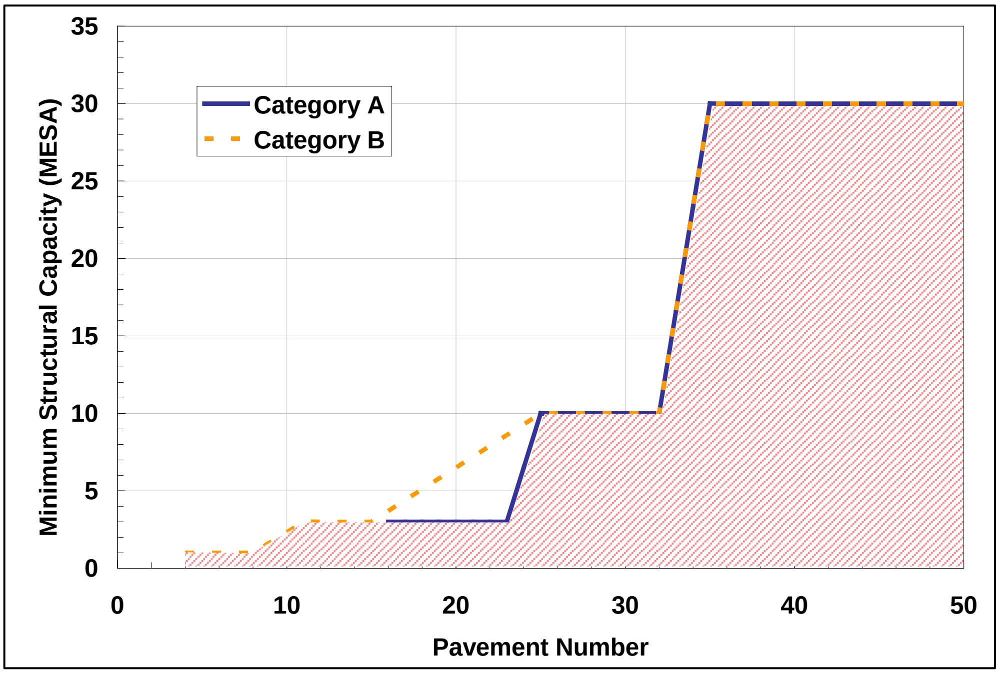


Figure 37. Criteria for Determining Allowed Capacity from PN

Advantages of the PN Method

- Relatively easy to apply and robust
- For new and rehabilitation design
- For intial and detailed assessments
- Evaluates the adequacy of individual layers in the pavement system
- Calibrated for southern African conditions using performance data from existing pavements
- One of the few methods calibrated for BSM layers
- Easy to implement in a spreadsheet

Disadvantages of the PN Method

- Limited to Category A and B roads
- Limited to between 1 and 30 million E80s
- Cannot be used where subgrade of CBR < 3% within 600 mm pavement depth
- Currently should not be used for asphalt surfacing thicker than 50 mm

7.3 Dynamic Cone Penetrometer (DCP) Method

The DCP design method is for new and rehabilitation pavement design. It is an empirical method that incorporates concepts such as pavement balance (see Section 3.4). The Dynamic Cone Penetrometer (DCP), shown in Figure 38, is a fairly basic evaluation instrument. However, the analysis and interpretation of DCP data have evolved to the extent where it may be used as a design tool in South Africa (Kleyn, 1984; Kleyn et al, 1989; Sampson, 1984; Jordaan, 1989a; and De Beer, 1991). The DCP device and measurements are described in Chapter 6, Section 5.4.5.

Figure 38. Dynamic Cone Penetrometer (DCP)

DCP References

A good reference for the DCP design method is:

- P Paige-Green, P. and Du Plessis, J.L. 2009. **The Use and Interpretation of the Dynamic Cone Penetrometer (DCP) Test.** CSIR Built Environment. Pretoria.
- Paige-Green, P. and Pinard, M.I. 2012. *Optimum Design of Sustainable Sealed Low Volume Roads using the Dynamic Cone Penetrometer (DCP).* **25th ARRB Conference**. Perth, Australia.

The DCP design method was developed and calibrated for new and rehabilitation design of granular and lightly cemented pavements. This design method has been verified with a number of HVS tests and by use in the industry for many years. The method is suitable to light pavement structures with mostly unbound granular or lightly cemented layers.

The important concepts involved in the DCP design approach and the input parameters required are:

- DCP **penetration rate**, DN (mm/blow).

- **Number of blows** DSN_d required to penetrate to depth "d". Typically, a depth of 800 mm is used, denoted DSN_{800}.
- Relative contribution of the individual pavement layers to the strength of the total pavement system, also referred to as the **pavement strength balance**.

A DCP penetration rate is required for any material to be used in the DCP design method. Correlations to convert the penetration rate (DN) to a CBR, UCS and effective stiffness are available, with the best known correlations reported by Kleyn et al (1989) and de Beer (1991). Based on the correlations and minimum strength requirements for unbound and cement stabilised material, recommended typical DN values are given in Table 40.

Table 40. Typical DCP Penetration Rates (DN-values) for Road-Building Materials

Material Code or Description		DN-values
Surfacings	Dense and hard asphalt	< 0.6
	Open and/or cracked asphalt	< 0.8
	Surface treatment, good condition	< 1.0
Unbound granular	G1	1.25 (1.1 – 1.4)
	G2	1.6 (1.4 – 1.8)
	G3	< 2.0
	G4	< 3.7
	G5	< 5.7
	G6	< 9.1
	G7	< 14
	G8	< 19
	G9	< 25
	G10	< 48
Lightly cemented	C3	1.2 (0.6 – 1.8)
	C4	2.6 (1.8 – 3.4)

The number of blows required to penetrate a certain material depth is referred to as the DSN_d-value with the subscript "d" indicating the depth of material penetrated. Empirical correlations have been developed that relate the structural capacity to the DSN_{800} for full-depth granular pavements, and to the DN_{50} (weighted average DN in the top 50 mm of the cemented layer) and DSN_{200} for lightly cemented (C3 and C4) pavements.

The DCP design method also explicitly handles pavement balance (see Section 3.4.1). To do this, a pavement balance curve is plotted, using the pavement depth and the contribution of that depth to the total pavement strength. The balance curve is compared to a perfectly balanced structure, which has a constant contribution to the total pavement strength with depth. A curve that deviates significantly from the perfectly balanced structure is not in balance. Guidelines for determining where the pavement is out of balance are provided.

Jordaan (1989) developed a series of standard layer-strength diagrams for different levels of structural capacity, as illustrated in Figure 39. These diagrams are often used for rehabilitation design by plotting the actual DCP layer-strength diagram with the design curves. Any portion of the actual DCP layer-strength diagram that plots to the right of the required design curve indicates inadequate material quality. Shifting the required design curve upwards until the total DCP layer-strength diagram plots to the left of the design curve indicates the additional cover required.

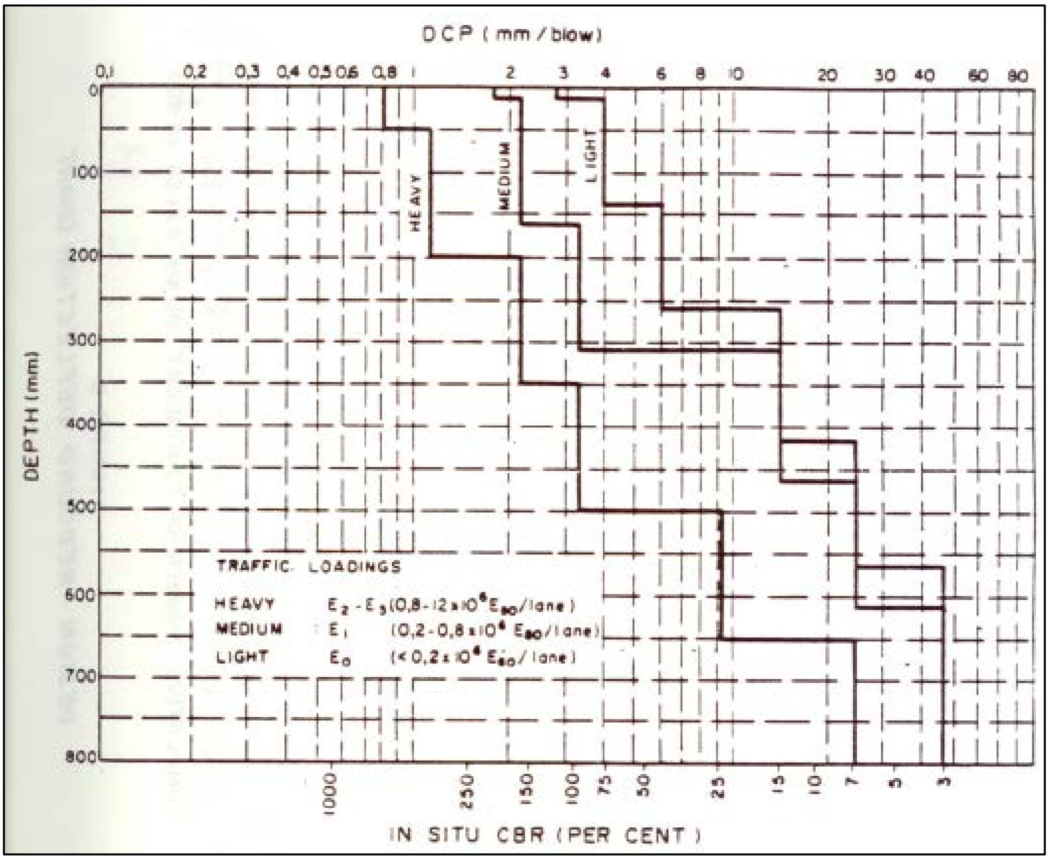


Figure 39. Standard Layer-Strength Diagrams for DCP Design

The DCP method, as with other methods like the SAMDM, does not use the current functional condition, i.e., the rut depth, in the estimation of the structural capacity. To incorporate the current condition via the rut depth, a simple linear relationship can be used, as illustrated in Figure 40.

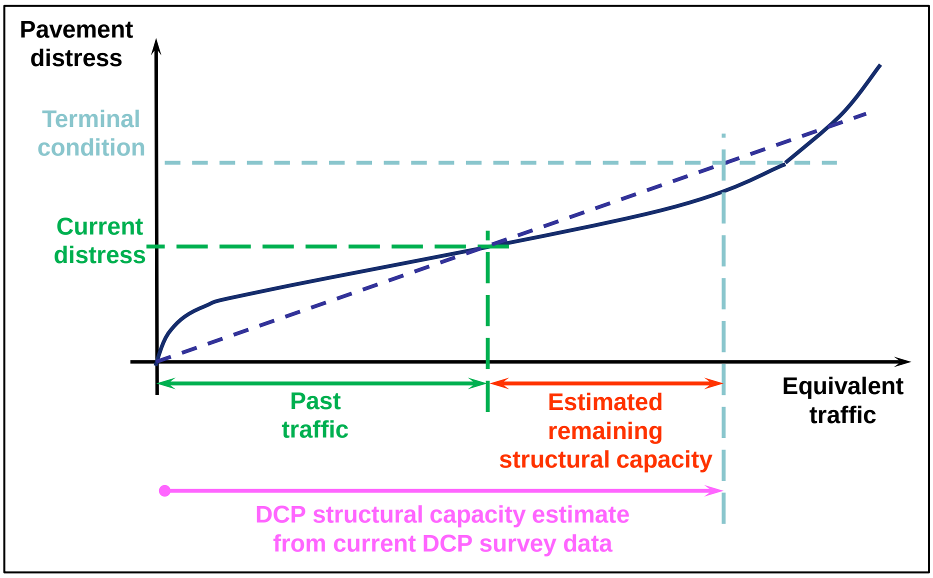


Figure 40. Estimating Remaining Life Considering the Current Condition

Advantages of the DCP Method

- Suited to new and rehabilitation design
- Relatively simple and easy to apply
- Tested for South African conditions and materials

Disadvantages of the DCP Method

- Need DCP Data
- Empirical: derived from CBR cover design
- Mostly applicable to unbound and lightly cemented pavements
- Variable results (need many repeats)
- Dependent on in situ moisture conditions (seasonal)
- Influenced by large aggregates in the pavement structure

7.4 The AASHTO Structural Number (SN) Method

The AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) gives a full description of the structural number (SN) method. The method can be used for new and rehabilitation pavement design. The method is based on the results of the AASHO road test done in Ottawa, Illinois during the late 1950s to early 1960s. This method must be applied with caution for a number of reasons:

- The method is an **empirical** method, based on performance data collected almost 50 years ago.
- The **subgrade and pavement materials,** as well as the pavement structures, used in the AASHO road test are foreign to South Africa.
- The method is in **imperial units** and conversion to metric units must be done correctly.

AASHO Road Test

The AASHO Road Test is described in Chapter 2, Section 2.2, "History of Pavement Design". The test was an enormous effort to systematically quantify the complex interaction between road deterioration, traffic and composition of the pavement structure on a closed loop test track with trucks.

The structural number method is, however, important because it is the only structural design method used in South Africa based on a reduction in the functional level of service of the pavement. The structural capacity estimation is based on a reduction in the Present Serviceability Index (PSI), a measure of riding quality. The pavement deterioration models of the Highway Development and Management system, HDM-4 (ISOHDM, 2004) also uses the structural number approach and these models will be incorporated in the revision of the South African flexible pavement design method. The method also provides a good check of designs completed using more complex methods.

The basic formula to estimate the structural capacity of a pavement is given by Equation (27). Each layer in the pavement contributes to the structural number according to a layer coefficient depending on material type, the thickness of the layer and a drainage coefficient for the layer, calculated using Equation (28). Note these equations use imperial units.

Refer to Table 2 for the initial and terminal serviceability indices for the road categories used in South Africa. Variability is accounted for by using the standard normal deviate and overall standard deviation, for which recommended values related to the design reliability are given in the references for the method.

The first step in the process of estimating the structural capacity of a pavement with the SN method is to determine the effective roadbed resilient modulus, which is an average subgrade resilient modulus adjusted for seasonal changes. The AASHTO design guide makes provision for dividing a year into half-month periods to accommodate seasonal moisture content variation in the subgrade. This division is believed to be too fine for subgrade moisture content variation in South Africa and a monthly or quarterly division is recommended, if such data are available.

Each layer is assigned a layer coefficient, representing the strength of the material. The value of the layer coefficients increases with increasing material quality. Typical ranges for the layer coefficients of the main South African material groups are provided in Table 41, and are based on the AASHTO 1993 design guide and local research.

$$\log \mathrm{SC} = Z_R \times S_o + 9.36 \times \log(SN+1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log(M_{ER}) - 8.07 \qquad (27)$$

where SC = Structural capacity of the pavement (Standard Axles)
Z_R = Standard normal deviate
S_0 = Combined standard error of the traffic and performance predictions
SN = Structural number of the total pavement thickness
ΔPSI = Difference between the initial (PSI_0) and terminal (PSI_t) serviceability indices
M_{ER} = Effective roadbed resilient modulus adjusted for seasonal variation (psi)

$$\mathrm{SN} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (28)$$

where SN = Structural number of the total pavement thickness
a_i = i^{th} layer coefficient (per inch)
D_i = i^{th} layer thickness (inches)
m_i = i^{th} layer drainage coefficient

Table 41. Layer Coefficients

Material	Ranges for South African Materials
Asphalt concrete	0.20 – 0.44
Crushed stone	0.06 – 0.14
Cemented-treated material	0.10 – 0.28
Bituminous-treated material	0.10 – 0.30

The AASHTO design guide of 1993 provides recommendations and predictive equations relating the layer coefficients of different material groups to other engineering parameters. The method also requires that drainage coefficients are applied to account for the quality of the drainage and the time the materials are exposed to near saturation conditions.

When using the AASHTO SN method to estimate the initial structural capacity in the initial assessment phase, the subgrade modulus derived from FWD backcalculation, or from the DCP penetration rate, may be substituted for the effective roadbed modulus. However, care should be exercised when using these values in a rehabilitation design situation since South African subgrade stiffnesses are often much higher than those the method caters for, which may result in unrealistically high structural capacity outputs. The change in riding quality is calculated from the appropriate percentile value for the current riding quality (PSI) and the terminal riding quality (PSI).

Advantages of the AASHTO Method

- Models riding quality deterioration
- Models available for flexible and rigid pavement design
- Relatively simple to apply
- Applicable to new and rehabilitation design
- Principles used in the HDM IV economic analysis software
- Relatively quick and easy to use, and provides a good check of pavement designs done by other methods

Disadvantages of the AASHTO Method

- Empirical: derived from data collected at one site in the USA
- Not sensitive to quality of base
- Outdated: derived from data collected almost 50 years ago
- Developed for foreign conditions and materials
- Uses imperial units

7.5 Deflection Bowl Parameter Analysis

Although not listed as an independent rehabilitation design method in its own right in TRH12, the interpretation of Falling Weight Deflectometer (FWD) deflection results has evolved to the point where it is quite a widely used method for evaluating pavements and for crude estimates of the remaining life. The method is described in more detail in TRH12, Maree (1990) and Horak (2008). Much of the discussion in this section is taken from Horak (2008).

When a flexible pavement deflects under the load of a heavy vehicle, the influence of the load extends to about 1 to 2 metres in three dimensions. The deflection area is typically a circular deflected indentation known as a deflection bowl. See Section 3.5.1 for a brief discussion on deflection bowls and FWD measurements. Deflection bowls are often used to backcalculate stiffness moduli for the pavement layers. To do this however, requires good knowledge of the materials in the layers and the layer thicknesses. The deflection bowl can be used to identify weak areas in the depth of a pavement structure and over the length of a uniform section, without detailed knowledge of the pavement structure and without backcalculation.

Deflection bowl parameters have been developed for analysing the deflection bowl. These are the base layer index (BLI), middle layer index (MLI) and lower layer index (LLI). The formulae for calculating these parameters are shown in Table 42.

Table 42. Deflection Bowl Parameters

Parameter	Formula			
Base layer index (BLI)[1]	BLI	=	$D_0 - D_{300}$	(29)
Middle layer index (MLI)[2]	MLI	=	$D_{300} - D_{600}$	(30)
Lower layer index (LLI)[3]	LLI	=	$D_{600} - D_{900}$	(31)
	where D_0	=	Maximum (peak) deflection, measured under the load[4]	
	D_{300}	=	Deflection at 300 mm sensor	
	D_{600}	=	Deflection at 600 mm sensor	
	D_{900}	=	Deflection at 900 mm sensor	

Notes
1. Previously referred to as surface curvature index (SCI)
2. Previously referred to as base curvature index (BCI)
3. Previously referred to as base damage index (BDI)
4. Also known as Y-max.

A deflection bowl measured under a load can be divided into three zones, as reflected in Figure 41 (from Horak, 2008):

- **Zone 1** is the closest to the load, and generally lies within 300 mm from the load. In this zone, the curvature of the bowl is positive. This zone is mainly surface and base layers and correlates well with the base layer index (BLI).
- **Zone 2** is typically between 300 mm to 600 mm, although the exact positions depend on the pavement structure. In this zone, the curvature switches from a positive to reverse curvature. This zone is mostly the subbase layers, and correlates well with the middle layer index (MLI).
- **Zone 3** lies furthest away from the load, and is generally from 600 mm to up to 2 000 mm from the load. The curvature is reverse and the deflection eventually reduces to zero. The extent of the deflection bowl depends on the pavement structure. Zone 3 is mostly selected and subgrade layers, and correlates well with the lower layer index (LLI).

Condition classification criteria have been developed for a number of FWD deflection bowl parameters. Limiting criteria, relating the cumulative number of E80s to a number of deflection bowl parameters are available. The criteria given in TRH12 are shown in Table 43. Criteria for different behaviour states are given, including a crude estimate of remaining life for pavements with deflection bowl parameters within those ranges. These correlations to remaining life must be used with great care, as they can lead to an over simplification and inaccuracies.

Horak has also suggested criteria for assessing pavements in terms of sound, warning and severe, shown in Table 44. By using these assessment criteria, deficiencies in the structural layers are identified. By assessing a length of road, the possible cause of structural deficiencies can be deduced. An example is given in the stripmap in Figure 28.

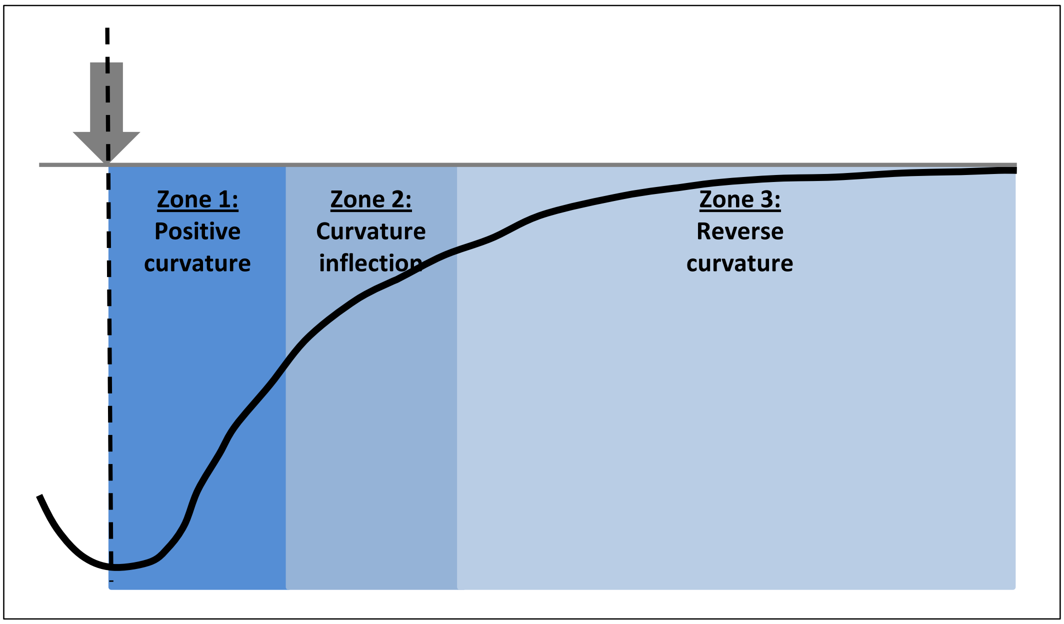


Figure 41. Zones of a Deflection Bowl (Horak, 2008)

Table 43. Behaviour States for Granular Base Pavements

Behaviour State	Traffic Range (MESA)	Maximum Deflection (mm)	BLI (mm)	MLI (mm)	LLI (mm)
Very stiff	12 – 50	< 0.3	< 0.08	< 0.05	< 0.04
Stiff	3 – 8	0.3 – 0.5	0.08 – 0.25	0.05 – 0.15	0.04 – 0.08
Flexible	0.8 – 3	0.5 – 0.75	0.25 – 0.5	0.15 – 0.2	0.08 – 0.1
Very flexible	< 0.8	> 0.75	> 0.5	> 0.2	> 0.1

Table 44. Deflection Bowl Parameter Structural Condition Rating Criteria

Pavement Base Type	Structural Condition Rating	Deflection Bowl Parameters				
		Ymax	RoC	BLI (mm)	MLI (mm)	LLI (mm)
Granular base	Sound	< 500	> 100	< 200	< 100	< 50
	Warning	500 – 750	50 – 100	200 – 400	100 – 200	50 – 100
	Severe	> 750	< 50	> 400	> 200	> 100
Cementitious base	Sound	< 200	> 150	< 100	< 50	< 40
	Warning	200 – 400	80 – 150	100 – 300	50 – 100	40 – 80
	Severe	> 400	< 80	> 300	> 100	> 80
Bituminous base	Sound	< 400	> 250	< 200	< 100	< 50
	Warning	400 – 600	100 – 250	200 – 400	100 – 150	50 – 80
	Severe	> 600	< 100	> 400	> 150	> 80

The following comments apply to the use of the deflection bowl analysis method:

- The method is an empirical method that was **calibrated for South Africa pavements and conditions**.
- The method allows for **different pavement types**.
- The method should therefore be used with caution as a **rough indication of the remaining structural capacity** of pavements.
- The use of **deflection bowl parameters** allows for the assessment of the adequacy of at least three zones within the pavement, from the upper layers to the subgrade.
- The method is **based on the FWD,** which is currently the preferred deflection test instrument at network and project level.

Advantages of the Deflection Bowl Parameter Method

- Very simple and easy to apply
- Calibrated for South African conditions, pavements and materials
- Allows for different pavement types
- Allows adequate assessment of different zones in the pavement
- Current: uses the preferred deflection device for network and project level testing

Disadvantages of the Deflection Bowl Parameter Method

- Crude indication of remaining life.
- Conservative: little remaining life is often calculated when the visual condition of the pavement is still good

7.6 Falling Weight Deflectometer (FWD) SN design method

The method was developed by Rohde (1994) for use in network level analyses, and uses FWD deflection measurements to determine the AASHTO Structural Number. The method is rapid, does not need mechanistic analysis tools and is generally used for characterising pavement strength in pavement management systems (PMS). As such, it can be used in the initial assessment phase of rehabilitation projects. In South Africa, the method is not frequently used, and when it is used it is generally only in PMSs, such as the Western Cape PMS.

The method uses the assumption that the surface deflection measured at an offset of 1.5 times the pavement thickness originates entirely in the subgrade. By comparing this deflection with the peak deflection, an index associated with the magnitude of deformation occurring within the pavement structure is calculated using Equation (32).

$$SIP = D_0 - D_{1.5Hp} \tag{32}$$

where
- SIP = Structural index of pavement (µm)
- D_0 = Peak deflection measured under a standard 40 kN FWD load (µm)
- $D_{1.5Hp}$ = Surface deflection measured at offset of 1.5 times Hp, under standard 40 kN FWD impulse load (µm)
- Hp = Total pavement thickness (mm), which includes all imported pavement layers. Where selected layers are used, they form part of the pavement thickness.

A relationship between FWD measured surface deflections and a pavement's structural number was developed by analysing a large number of pavements with layered-elastic theory, and shown in Equation (33). The structural capacity is calculated using the AASHTO method.

$$SN = k_1 \, SIP^{\,k2} \, Hp^{\,k3} \tag{33}$$

where
- SN = structural number (inches) as used in HDM-III
- SIP = structural index of pavement (µm)
- Hp = total pavement thickness (mm)
- k_1, k_2, k_3 = coefficients

Surface Type	k_1	k_2	k_3
Surface Seals	0.1165	-0.3248	0.8241
Asphalt Concrete	0.4728	-0.4810	0.7581

Using Surface Deflection Methods

Surface deflection methods are applicable to pavements where the subgrade is a problem, or where the subgrade cover is insufficient.

If Benkelmen beam data are the only deflection data available, the TRRL or Asphalt Institute surface deflection methods are appropriate, however the conditions for their use still apply.

When using the Benkelman beam, be careful of the loading used, as there are a few "standard" loads.

Advantages of the FWD SN Method

- Models riding quality deterioration
- Models available for flexible and rigid pavement design
- Relatively simple to apply
- Principles used in the HDM IV economic analysis software
- Uses FWD data, which is quick and relatively inexpensive to obtain

Disadvantages of the FWD SN Method

- Empirical: derived from data collected at one site in the USA
- Outdated: derived from data collected almost 50 years ago
- Developed for foreign conditions and materials
- For rehabilitation design only
- For initial assessment purposes only

7.7 The TRRL Surface Deflection Design Method

This discussion of the TRRL surface deflection method is taken largely from Jordaan (1989). The TRRL method was developed by the Transport and Roads Research Laboratory in England and is applicable to rehabilitation design. The primary pavement input to the method is the peak Benkelman beam surface deflection under a dual wheel load of 31 kN (single axle load of 62 kN) moving at creep speed, at a temperature of 20 °C 40 mm below the pavement surface. The design standard in South Africa is, however, an 80 kN axle load. Jordaan adjusted the design charts linearly to accommodate deflections at 80 kN. Deflections should be recorded at intervals of less than 12 metres.

The TRRL surface deflection method provides design charts to estimate remaining life and determine overlay thickness. Although the method is based on Benkelman beam deflections, it also allows for the use of Deflectograph deflections by converting using a single conversion relationship. FWD deflections also need to be converted to a Benkelman beam deflections before the method can be used. It should also be noted that the conversion of deflections measured from one deflection device to another depends on the pavement structure, materials and moisture conditions, and therefore conversions should always be used with great care.

Charts for adjusting the deflection for variation in the pavement temperature at the time of the deflection survey to the reference temperature of 20 °C for the "standard deflection" are provided for different pavement types. Jordaan warns that the temperature corrections are inaccurate for pavements with weak or strong subgrades at the extremes of the conversion ranges.

(i) Estimate Remaining Life

Charts to estimate the remaining life of an existing pavement to reach a critical (terminal) condition are available for the following bases types:

- Granular bases
- Granular bases exhibiting self-cementing
- Bituminous bases (HMA)
- Cement-bound bases

Figure 42 shows an example of the charts, this one for a self-cementing granular base.

Converting Deflections from Different Devices

Converting deflections measured from one deflection device to another depends on the pavement structure, materials and moisture conditions. If it generally advised not to convert deflections. However, if conversion is done, it must be done with great care.

The charts for estimating remaining life should be used with caution on pavements with extensive surface distress. These pavements may have low deflections, but the surface distress leads to rapid deterioration and early failure.

The following cautions apply to the use of the chart for pavements with cement-bound bases:

- The performance of pavements with cemented layers depends on **deterioration associated with shrinkage cracking** of the cemented layer.
- The chart for estimating the remaining life of cement-bound pavements is only **valid above 10 million E80s** for bituminous cover 175 mm or thicker.
- The **surface deflection** does not give a good indication of the remaining life when structural weakness is present in and upper cemented layer, supported by a sound lower cemented layer.
- Surface deflection early during the life of pavements with cemented layers does not give a good indication of the remaining life, if the grading of the aggregate for the cemented layers is such that the material has little

mechanical stability. Cracking of the layer results in disintegration, with an associated rapid reduction in remaining life.

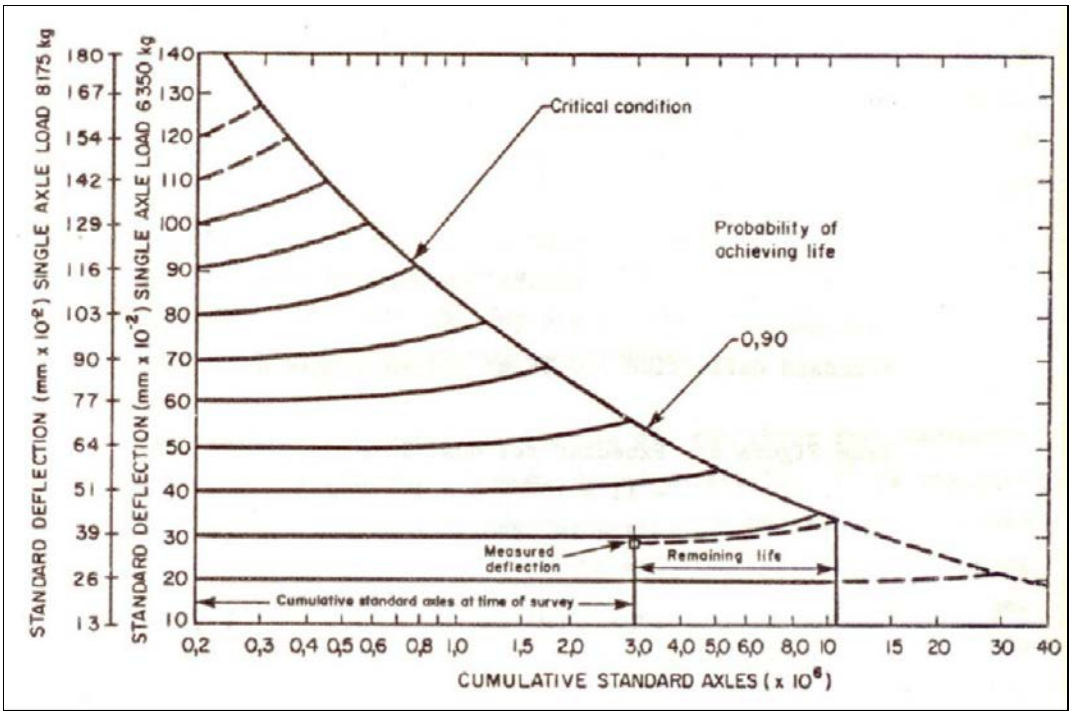


Figure 42. Using the TRRL Chart to Determine Remaining Life for a Self-Cementing Granular Base Pavement

(ii) Overlay Design

Overlay design charts are provided for the same pavement types as for remaining life. The thickness of the overlay depends on the pavement type, the standard deflection before rehabilitation and the expected future traffic. The design charts are based on a 90% probability of achieving the design life. An asphalt overlay thickness of 40 mm is considered the minimum. Figure 43 shows an example for pavements with granular bases.

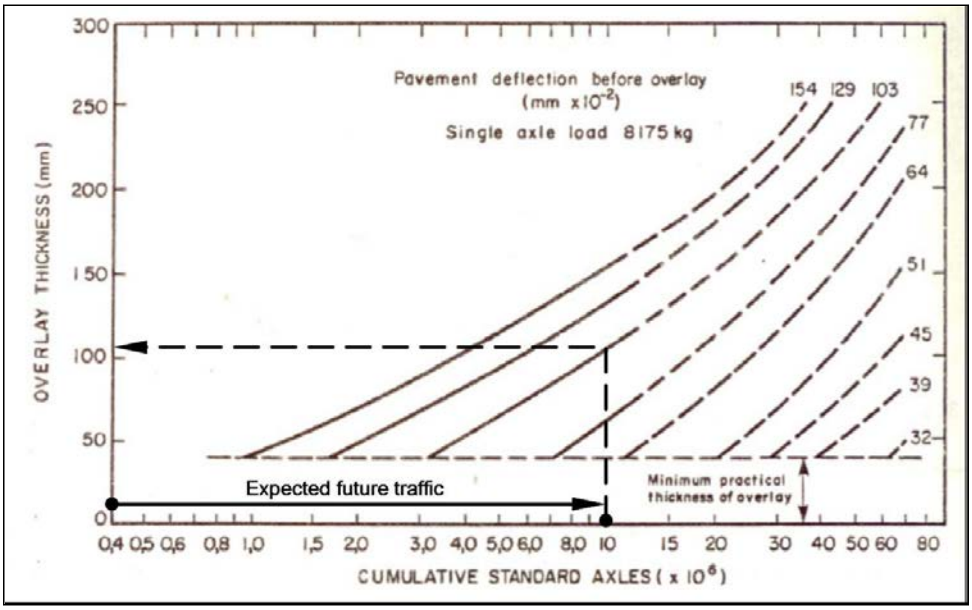


Figure 43. Example of the TRRL Overlay Design Chart for Granular Base Pavements

The following comments apply to the use of the TRRL surface deflection method:

- The method is an **empirical method** developed from data collected only in England.
- The method is based on **Benkelman beam deflection**. Although a conversion from Deflectograph to Benkelman beam deflection is provided, it is based on a single correlation between these two deflection types, which probably does not hold for all pavement types.
- The method provides for different deflection-performance relationships depending on **pavement type.**
- The method was developed for a **62 kN axle load** while the design standard in South Africa is an 80 kN axle load. Although Jordaan (1989b) adjusted the charts to accommodate the deflection at an 80 kN axle load, the adjustment is linear. It is known from field observations that the relationship between axle load and pavement deflection is not linear.
- Although deflections are adjusted for **temperature,** no adjustment is recommended for **seasonal (moisture)** variation.
- The method was developed using data from sections that all had **inadequate subgrade cover**, designed to fail within a reasonable time. The method is there suited for pavements that have inadequate cover, which needs to be provided.
- The **deflection–capacity relationship** is only valid for the types of pavements tested. The trend that the deflection increases with accumulated traffic and deterioration is because the pavements did not have adequate subgrade cover.
- The method does not allow for the evaluation of the **adequacy of the individual pavement layers**.
- The **mathematical formulation** for the design charts is not available and the application of the method relies on plotting the deflection and past traffic on the charts by hand and visually reading off the remaining life and required overlay thickness. The method is therefore not well suited for modern software systems for pavement rehabilitation.
- The **Benkelman beam** and **Deflectograph** are becoming obsolete and the current preferred instrument for deflection surveys is the Falling Weight Deflectometer.

Advantages of the TRRL Method

- Provides for different pavement types
- Accommodates Benkelman beam and Deflectograph deflection
- Corrects deflection for temperature variation
- Relatively simple and easy to apply

Disadvantages of the TRRL Method

- Empirical: derived from data collected in England
- Outdated: based on Benkelman beam deflection
- Developed for 62 kN axle loads
- Relies on correlations for converting Deflectograph and FWD deflections to equivalent Benkelman beam deflections
- Not convenient for software implementation
- Applies to rehabilitation design only

7.8 The Asphalt Institute Surface Deflection Design Method

The Asphalt Institute (AI) method for pavement rehabilitation design uses either a pavement layer analysis or a deflection analysis procedure. Only the deflection procedure is discussed in this section, and the discussion is taken largely from Jordaan (1989c). This method is not routinely used in South Africa.

The primary pavement input to the method is Representative Rebound Deflection (RRD). The RDD is calculated from the mean Benkelman beam surface deflection plus two standard deviations for the deflection data sample. A minimum of 10 deflections at maximum intervals of 80 metres are recommended per road section. The deflection is adjusted to a reference temperature of 21 °C at the most critical period, usually just after the rainy season. A chart is provided for temperature adjustment of the deflection, but no guidelines are provided on the adjustment for the critical period.

The AI surface deflection method provides design charts to estimate the remaining life of an existing pavement to reach a terminal condition, and to determine the asphalt overlay thickness required to extend the life of the pavement to carry the future expected traffic.

(i) Remaining Life Estimation

A single chart, shown in Figure 44, is provided for estimating the remaining life, regardless of the pavement type. The method was, however, developed for pavements with granular base layers and thin surfacings, and therefore only applies to these pavements. This relationship was developed by combining the data from a number of sources (Jordaan, 1989c). The RRD (adjusted for deflection variation, temperature and season) is used to determine the total traffic that can be accommodated from the AI design line. The past traffic estimate is subtracted from the total traffic estimate to obtain the remaining life.

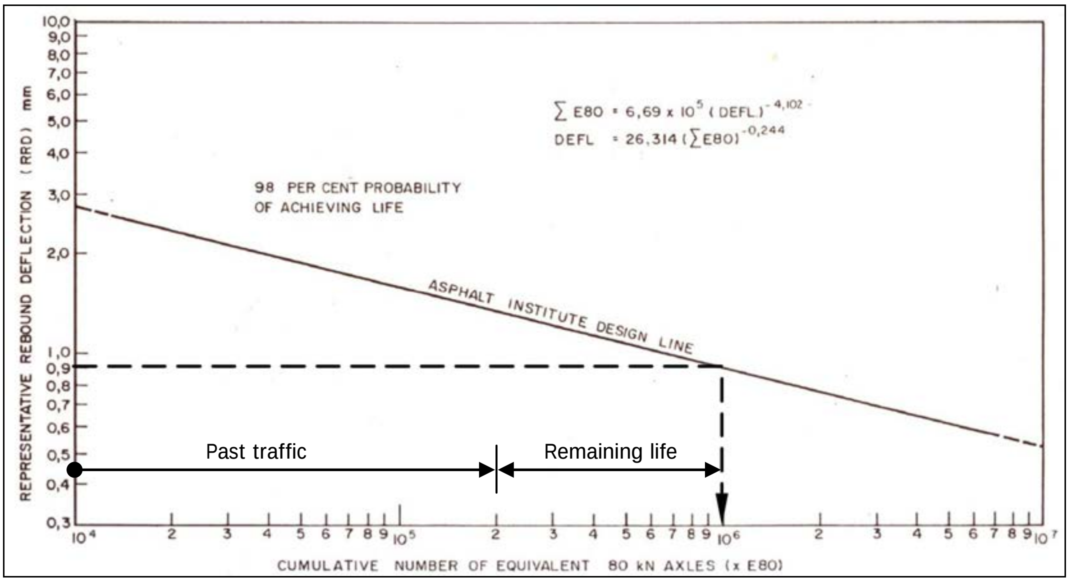


Figure 44. Example of Using of the AI Chart to Determine Remaining Life for a Pavement

(ii) Overlay Design

The AI overlay design chart is shown in Figure 45. The overlay design chart was developed using linear-elastic, multi-layer theory assuming an overlay resilient modulus of 3 400 MPa.

The following comments apply to the use of this method:

- The method was developed with data from test sections with **granular bases and thin surfacings**.
- The **Representative Rebound Deflection (RRD) and pavement life relationship** is only applicable to traffic loadings between 0.014 and 7.3 million equivalent standard axles, and deflections between 0.76 and 3.56 mm.
- The **overlay design chart** was developed for asphalt with a resilient modulus of 3 400 MPa at 21 °C, and is only applicable to design overlays with similar characteristics.
- Although an **adjustment for temperature variation** is provided, no adjustment guideline is provided for seasonal variation.
- The method was developed to provide adequate cover to the subgrade and does not allow for the evaluation of the adequacy of the **individual pavement layers.**
- The thickness design chart is not sensitive to a **RRD below 1 mm**.
- The original overlay design chart was only developed for **traffic loadings** up to 7.3 million equivalent standard axles and was only later extended to 50 million.
- The **mathematical formulation** of the RRD and pavement life relationship is provided, but not for the overlay design chart. Only part of the method is therefore well suited to application in modern software systems.
- The **Benkelman beam** is becoming obsolete and the current preferred instrument for deflection surveys is the Falling Weight Deflectometer.

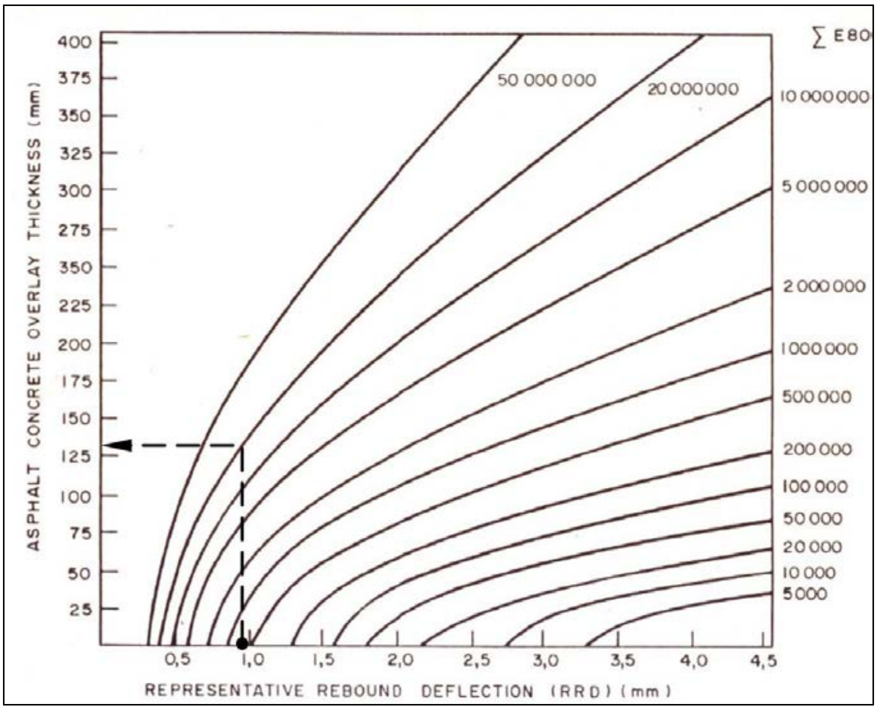


Figure 45. Asphalt Institute Overlay Design Chart for Granular Base Pavements

Advantages of the AI Method

- Very simple and easy to apply
- Corrects deflection for temperature variation
- Incorporates spatial variation of deflection data

Disadvantages of the AI Method

- Empirical: derived from data collected in North America
- Only for granular pavements with thin surfacings
- Only developed for traffic up to 7 million E80
- Insensitive to deflections less than 1 mm, which is where many South African pavements operates
- Outdated: based on Benkelman beam deflection

7.9 Summary of Suitability of Structural Design Methods for Flexible Pavements

Table 45 provides a summary of the potential application of the structural design methods discussed in the preceding sections.

7.9.1 Comment on Determining Remaining Life

One of the key aspects in the analysis of a pavement for rehabilitation is determining the remaining life. All of the structural design methods discussed, except the AASHTO SN method, actually determine the structural capacity from the initial conditions, and do not consider the observed distress, i.e., rut depth. There is no widely accepted method used for adjusting the structural capacity estimate from the initial conditions using the current condition to obtain the remaining life. Jordaan (1989a) provides some indications for incorporating the observed distress, or the simple method discussed for the DCP in Section 7.3 is an approach used. The analysis method suggested by Wolff (1993, 1994) for granular materials (Section 7.1.2(ii)) is an example of an approach that directly uses the current condition in estimating the remaining life.

Table 45. Potential Application of Structural Design Methods

Method	Potential Application		
	New Design	Rehabilitation Design	
		Initial Assessment – Remaining Life	Rehabilitation Design – Future Structural Capacity
SAMDM for flexible pavements	✓	Not practical	✓
Pavement Number method	✓	✓	✓
DCP design method	✓	✓	✓
AASHTO SN method	✓	✓ (with riding quality results)	✓
FWD deflection bowl parameter method	✗	✓	✗
FWD SN method	✗	✓	✓[1]
TRRL surface deflection method	✗	✓	✓
AI surface deflection method	✗	✓	✓

Notes
1. Suitable for rehabilitation design if used in conjunction with AASHTO Method.

7.10 Software Packages for Flexible Pavement Design

Software packages are available to assist with various tasks of the pavement design process, including design traffic estimation, design investigation, pavement modelling and analysis as well as structural capacity estimation. The purpose of these packages is to facilitate capturing, storing, processing and analysing of large volumes of data in a fraction of the time required to do a single analysis by hand. The intention is to improve the design engineer's efficiency and to create a better understanding of the design problem, by including more data in the analysis and interpretation than is possible without the software. These software packages are not intended as "black-box" solutions, and should never be regarded as substitutes for engineering knowledge and expertise, but rather as supplements to engineering knowledge, expertise and the design process.

This section presents a brief overview of the software packages available for flexible design. Rigid and block pavements are dealt with in Sections 8.4 and 9.6. These packages were not critically evaluated and it is not the intention to compare them or to endorse a specific package. Only the salient features of the software are described, refer to the software vendors for detailed information on the functionality of the software.

7.10.1 CYRANO 105

Cyrano was developed by Dr Pieter Strauss and Dr Martin Slavik, and is obtained by contacting martins@bks.co.za. Cyrano is largely based on the South African Mechanistic-Empirical Design Method (SAMDM) with the following modifications and additions:

- The factor of safety model for **unbound granular base layers** (Maree, 1978; Section 7.1.2) is replaced with the permanent deformation model developed by Wolff (1993) (Section 7.1.2(ii)).
- The principle that deformation is dependent on stress ratio is applied to calculate the **deformation of the subbase**.
- **Crushing of the surface of the base** and the associated loss of bond between the surface and the top of the base is based on equations developed from finite element analysis.
- The **change in stiffness** of the surface, base and subbase with increased load cycles or time is done through a recursive simulation scheme.
- The outcome of the analysis, namely the **percentage of the area with unacceptable cracking and deformation**, is translated into area that has to be repaired and the calculation of present worth of pavement life cost.
- The **increase in road roughness with an increase in load cycles** is based on work done at the AASHTO road test and by Ullidtz (1998).

Table 46. Functionality of the CYRANO 200 Software Package

Application Area	Aspect/Functionality	Comment
Design traffic estimation	Traffic data options	Weigh-in motion (WIM) axle load spectra are used. Typical default spectra are included if data are not available.
	Pavement load sensitivity	From calculated pavement response under load spectra and empirical performance prediction.
	Design traffic estimation	Calculated from the traffic input data.
Design investigation	Input data	Pavement layer characteristics and costs, traffic loading data and environmental condition
	Data analysis options	Sensitivity analysis using the "What if" function
	Data import, export and reporting	Summary report containing input and output
Pavement analysis and modelling	Analysis engine	ELSYM 5
	Analysis options	Analyze the effect of variation in input data on the outcome by using the "What if" facility.
	Material data and damage models	User defined material properties and calibration coefficients
	Structural capacity estimation	Progressive distress through a recursive simulation scheme
Simulation schemes	Monte Carlo variability simulation	Used to arrive at the predicted percentage area failed with loading or time
	Recursive damage model	Available as part of Monte Carlo simulation procedure
	Riding quality deterioration	Based on AASHTO and calibrated for SA conditions

7.10.2 PADS Software Series

The Pavement Analysis and Design Software (PADS) series is developed and distributed by CSIR Built Environment. It is available directly from CSIR, www.csir.co.za. The software consists of a number of independent components that are integrated into a single package for project level design investigation and analysis. The series consists of the following packages:

- *Traf*-PADS®: design traffic estimation
- *Back*-PADS®: deflection bowl analysis and backcalculation
- *Me*-PADS®: mechanistic-empirical structural capacity estimation
- *Pro*-PADS®: project level design investigation and analysis

Table 47 summarises the functionality of the PADS series.

Table 47. Functionality of the PADS series

Application Area	Aspect/Functionality	Comment
Design traffic estimation	Traffic data options	Weigh-in-motion (WIM) data are read directly from the .RSA file format to provide vehicle loading and traffic volume inputs. WIM axle load spectra (vehicle loading) may also be used in conjunction with traffic counts (traffic volume) data. Axle load spectra may also be generated from traffic count data in combination with the load capacity utilisation of different vehicle classes.
	Pavement load sensitivity	AASHTO power damage law or mechanistic-empirically derived load sensitivity.
	Design traffic estimation	Estimated from the traffic input data described above as well as traffic and vehicle loading growth rates.
Design investigation	Input data	Accommodates the following data: • Lane configuration • As-built structure information • Test-pit and core data • Visual condition assessment • Rut • Riding quality • Deflection (FWD and Deflectograph) • Dynamic Cone Penetrometer (DCP) • Digital images
	Data analysis options	Multiple data views are provided. The following data analysis options are provided: • Deflection bowl parameter analysis • Cumulative sum of difference analysis for analysis and uniform section identification • Automated backcalculation per user-defined uniform section • DCP single point and average point analysis per user-defined uniform section
	Data import, export and reporting	All input data are imported through Microsoft Excel spreadsheet templates or directly from the instrument files e.g., .F25 or .FWD file format for the FWD. Analysis results are exported to Microsoft Excel for further manipulation. Lane summary reports are generated showing the visual condition assessment input data. Lane summary reports are generated showing all the available processed data on a stripmap for each lane aggregated according to analysis section or uniform section. A one-page summary report is generated per uniform section.
Pavement analysis and modelling	Analysis engine	GAMES (Maina, 2004) with unrestricted number of layers and non-uniform contact stress distribution.
	Analysis options	Stress and strain results at 100 user-defined analysis points Stress and strain profile plots and contour plots.
	Material data and damage models	User defined material properties. Model formulation based on the 1996 SAMDM. User definable calibration coefficients.
	Structural capacity estimation	Allows for 3 distinct phases.
Simulation schemes	Monte Carlo variability simulation	Possible but requires manual user-intervention.
	Recursive damage modelling	Not available
	Riding quality deterioration	Not available

7.10.3 Rubicon Toolbox

Rubicon Toolbox is developed and distributed by Rubicon Solutions (www.rubiconsolutions.co.za). The software consists of a number of independent components that may be combined according to user needs. Rubicon Toolbox consists of the following components:

- Automated backcalculation tool
- Deflection bowl analysis tool
- Data viewer for drawing stripmaps of many different data types
- DCP analyser
- Standard axle tool
- Stress and strain calculator
- Monte Carlo simulation tool
- Structure comparer tool
- Trial pit reporter
- Photo logger
- Grading analysis tool
- Axle spectrum tool
- Finite element tool

Table 48 summarises the functionality of Rubicon Toolbox.

Table 48. Functionality of Rubicon Toolbox

Application Area	Aspect/Functionality	Comment
Design traffic estimation	Traffic data options	Loading is user defined, with many loads allowed. Loads defined by co-ordinates, load and pressure.
	Pavement load sensitivity	Incorporated in Axle Spectrum Tool by including different load groups and allocating daily repetitions to each group.
	Design traffic estimation	Estimated from load definitions, and traffic and vehicle loading growth rates.
Design investigation	Input data	Accommodates the following data: • Test-pit data • Gradings • Visual condition assessment • Rut • Riding quality • Deflection (FWD and Deflectograph) • Dynamic Cone Penetrometer (DCP) • Digital images
	Data analysis options	Multiple, user-defined data views are possible using the data viewer. The following data analysis options are provided: • Deflection bowl parameter analysis • Aggregation of data with summary statistics for user-defined sections • Cumulative sum of difference analysis for analysis and uniform section identification • Automated backcalculation • DCP single and multiple point analysis
	Data import, export and reporting	Input data are imported through Microsoft Excel spreadsheet templates and Microsoft Access databases, or directly from the instrument files, e.g., .F25 or .FWD file format for the FWD. Analysis results are exported to Microsoft Excel for further manipulation Concise summary reports are generated for each analysis option Customized uniform summary reports using the Data Viewer.
Pavement analysis and modelling	Analysis engine	WESLEA
	Analysis options	Comparison of stress and strain results at critical locations in alternative pavement structures. Stress and strain results at user-defined analysis points. Stress and strain profile plots and contour plots using the Finite Element Tool.
	Material data and damage models	User defined material properties Wide selection of damage model formulations User defined calibration coefficients Database with pre-defined published damage models
	Structural capacity estimation	Allows for multiple distinct analysis phases.
Simulation schemes	Monte Carlo variability simulation	Variability allowed on material properties, layer thickness and traffic wander.
	Recursive damage modelling	Based on cumulative damage using Miner's Law.
	Riding quality deterioration	AASHTO SN Method Tool available

8. STRUCTURAL CAPACITY ESTIMATION: CONCRETE PAVEMENTS

The first formal design method for concrete pavements in South Africa was included in Manual M10: Concrete Pavement Design and Construction (1995). It became apparent from overseas research, performance of local pavements and some instrumented sections of a concrete inlay on the N3 that the method was inherently conservative, and thus resulted in expensive pavements. A new design method to design cost effective pavements was needed. It also needed to be mechanistically based, to facilitate interaction with the mechanistic flexible pavement design methods. The mechanistic-empirical design method for concrete pavements was developed, based on the same principles as in the Manual M10. This method is implemented in the software package cncPAVE.

Brief overviews of the following methods for concrete pavements are discussed in this section:

- Manual M10
- Mechanistic-empirical design method (cncPAVE)

8.1 Manual M10

Manual M10 was developed from the AASHTO method for concrete pavements (M10, 1995). The AASHTO method essentially follows a recipe type approach to design and uses a series of nomograms, but it does contain some aspects of mechanistic design (AASHTO, 1993). For Manual M10, the AASHTO method was refined, validated and simplified for South African conditions. It is discussed here in some detail, as it provides a good idea of the basic elements of structural capacity estimation for concrete pavements.

Manual M10 essentially follows this sequence for designing concrete pavements:

- Determine **axle group loading**
- Select **stiffness moduli** used for slab support layers
- Use nomograph to get **equivalent support stiffness**
- Use nomograph to determine **relative vertical movement at joint/crack**, i.e., load transfer
- Use nomograph to obtain **slab thickness**

(i) Determine Axle Group Loading

The graph in Figure 46 is used to convert the axle loads and types to an equivalent single axle load.

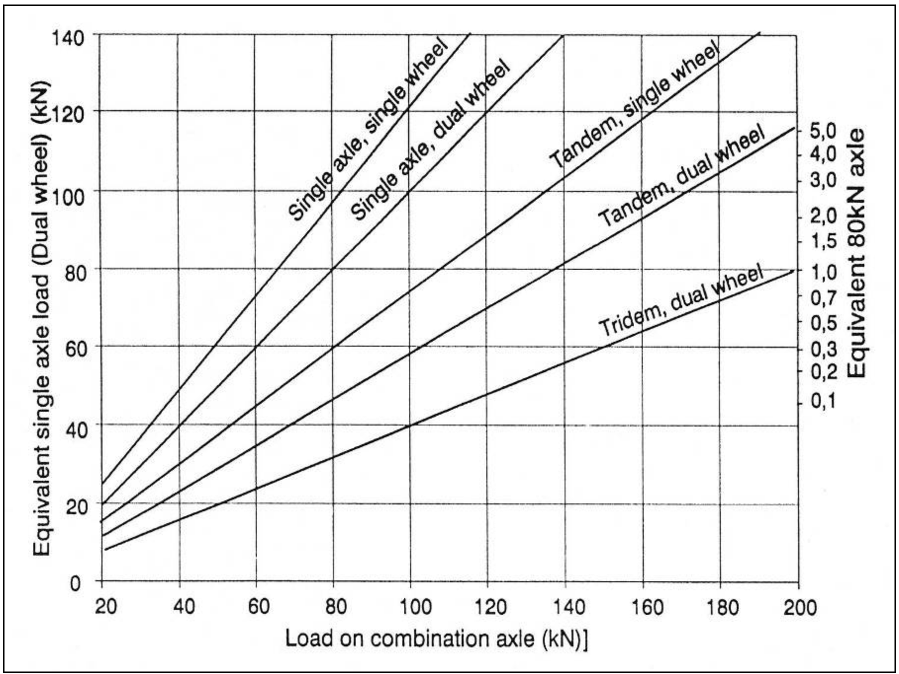


Figure 46. M10 Manual: Determine Axle Loading

(ii) Support Stiffness

The first step in determining the stiffness of the support layers is to identify the type of subbase and subgrade, and the subbase thickness. Recommended stiffness values for the materials types are given in Table 49. To determine the equivalent support stiffness from subgrade and subbase, the nomagraph in Figure 47 is used.

Table 49. Stiffness Moduli for Support Layers

Material and Material Class		Stiffness Value (MPa)
PCC		30 000 to 50 000
Asphalt		1 500 to 5 000
Cemented		1 500 to 15 000
Granular	G1	250 to 750 (400)[1]
	G3	150 to 400 (200)
	G5	100 to 250 (130 for subgrades)
	G7	50 to 160 (90 for subgrades)
	G9	30 to 80 (50 for subgrades)

<u>Note</u>
1. Recommended value in parenthesis

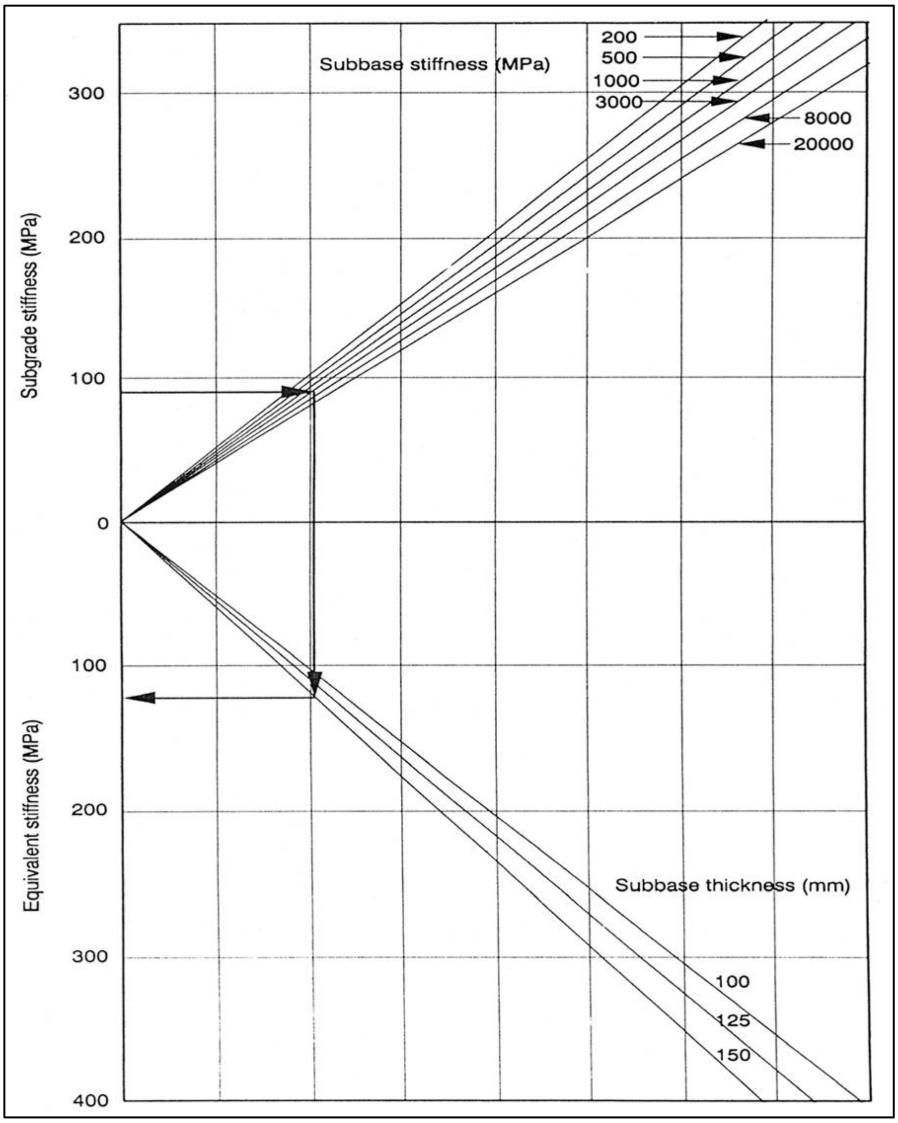


Figure 47. Nomagraph for Determining Equivalent Stiffness

(iii) Determine Relative Vertical Movement at Joints or Cracks

The next step is to determine the relative joint or crack movement, using the nomagraph in Figure 48. This nomagraph requires the spacing of dowel bars and the dowel bar diameter, and/or the crack/joint spacing and maximum aggregate size in the concrete mix. The minimum movement from the two nomagraphs in Figure 48 is used.

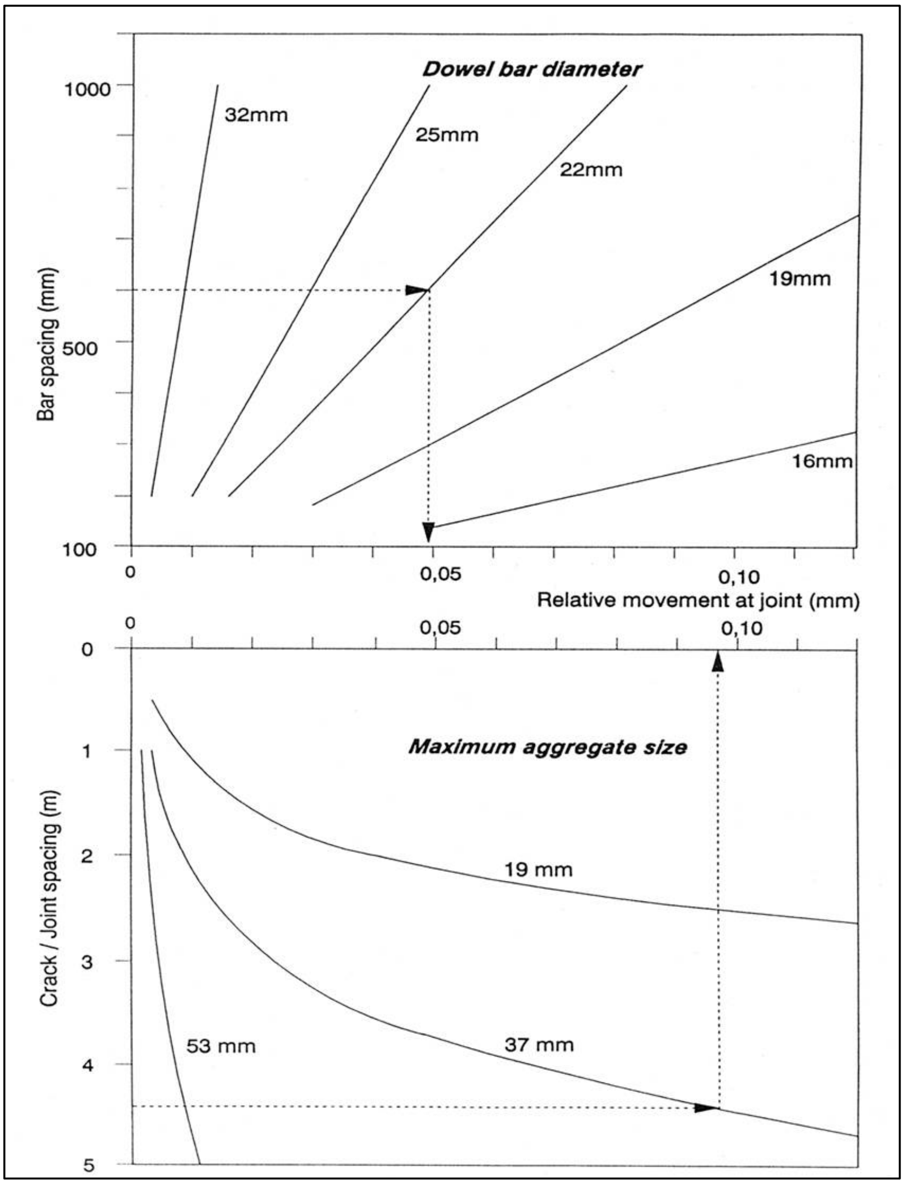


Figure 48. Relative Movement at Joint

(iv) Determine Slab Thickness

The final step in the concrete slab design is to determine the slab thickness required. This is done using the nomagraph in Figure 49, which uses the:

- **Equivalent single 80 kN axle loads** from Figure 46 and the number of each load group
- **Equivalent support stiffness** from Figure 47.
- **Concrete flexural strength**
- Relative **joint movement**
- **Type of concrete pavement** is shown to give an indication of the expected movement

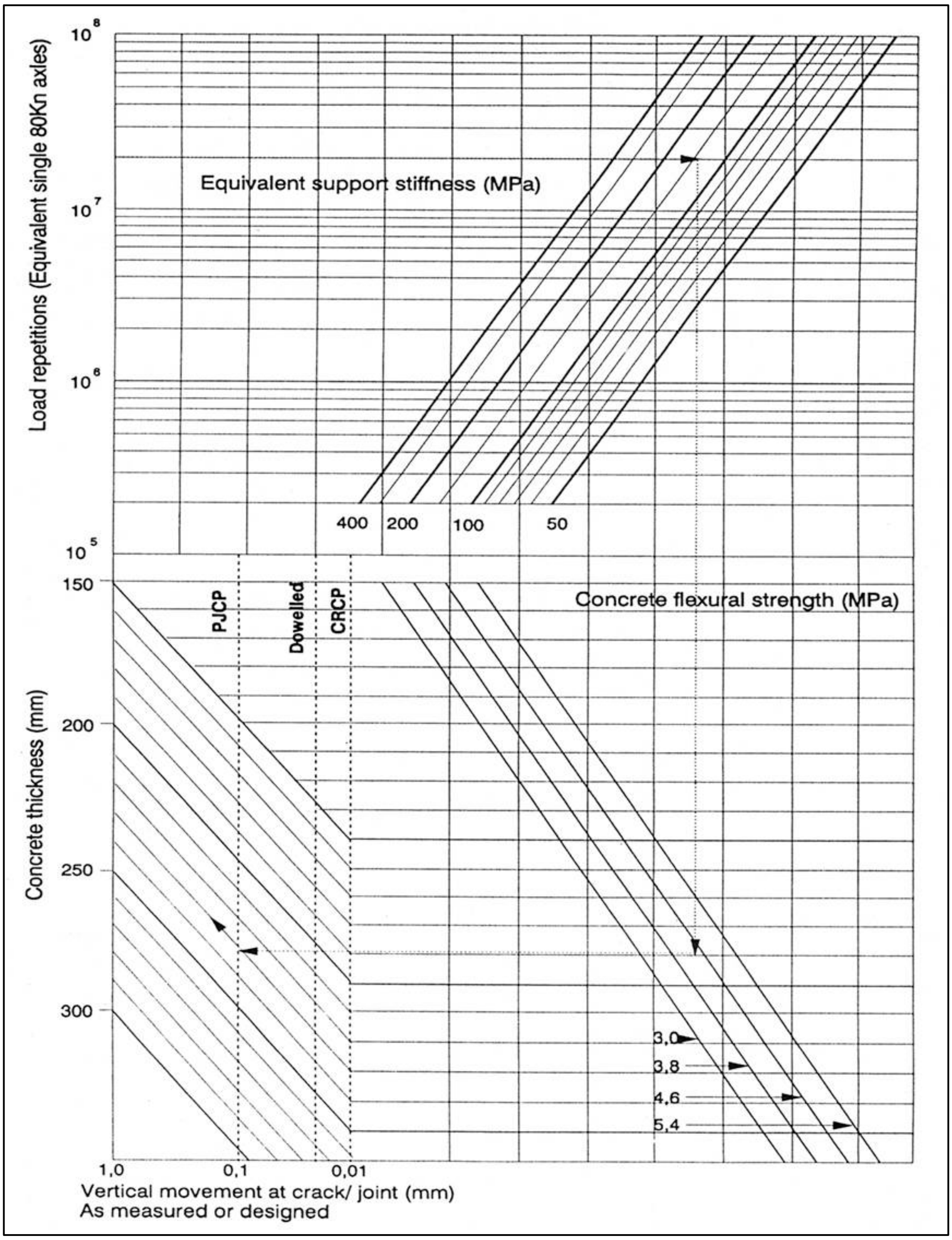


Figure 49. Nomagraph for Determining Concrete Slab Thickness

8.2 Mechanistic-Empirical Design Method for Concrete Pavements

Concrete pavement design in South Africa is now generally done using the Mechanistic-Empirical Design Method, as implemented in cncPAVE. The principles are the same as from Manual M10, with more details and analysis options. The flowchart in Figure 50 illustrates the design process using this method.

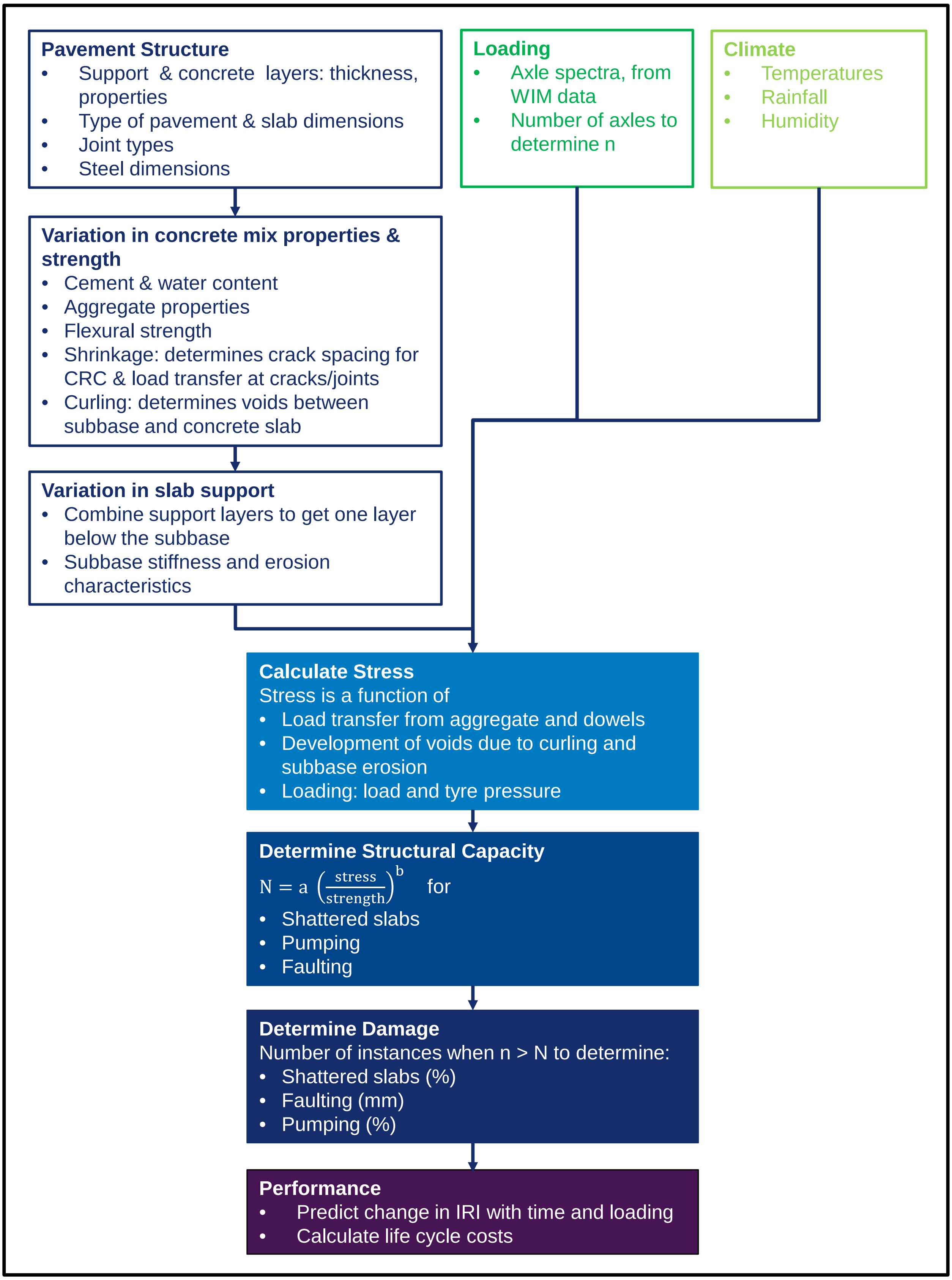


Figure 50. Flowchart for Mechanistic-Empirical Design of Concrete Pavements

The design process begins with determining the inputs, which fall into these categories:

- **Pavement structure**
- **Concrete** mix properties and strength
- **Loading:** number of axles and the axle load spectra
- **Climate**: temperature, rainfall and humidity

In the cncPAVE implementation, many of the input parameters are input as distributions, to account for variability and to enable Monte Carlo type statistical analyses.

Suitable design models were developed from finite element and multilayer evaluations. These models are used to determine:

- **Shrinkage** and **curling** of the slab
- Effective **subbase support** by combining all support layers into one layer
- **Erosion** characteristics of the subbase
- **Load transfer** across joints from aggregate and dowels
- **Development of voids** under the slabs
- **Structural capacity** for the following three mechanisms:
  - Shattered slabs
  - Faulting
  - Pumping

The models were calibrated by performance data from different concrete paved sections, including roads, streets and hardstandings, some of which were accurately instrumented.

The damage is determined as the ratio of the number of axles n, and the structural capacity, N, for each distress mechanism. The number of instances in the statistical simulation, where n>N determines:

- Percentage **shattered** concrete surface
- Percentage **pumping** concrete surface
- **Faulting** in the concrete pavement (in the case of plain and dowel concrete)

The performance of the concrete pavement is determined by predicting the change in IRI with time and loading.

The final step is to calculate the life-cycle costs to enable selection of an appropriate and cost-effective pavement design. The decision criteria for plain, dowel and continuously reinforced concrete are shown in Table 50.

Table 50. Typical Decision Criteria for Plain and Dowel Jointed and Continuously Reinforced Concrete

Plain and Dowel Jointed Concrete			
Decision Variable	**Good**	**Acceptable**	**Excessive**
% shattered concrete	< 2%	2% to 5%	> 5%
% pumping	< 2%	2% to 5%	> 5%
% faulting	< 2%	2% to 5%	> 5%
Continuously Reinforced Concrete			
Decision Variable	**Good**	**Acceptable**	**Excessive**
% shattered concrete	< 0.2%	0.2% to 0.5%	> 0.5%
From % pumping	< 2%	2% to 5%	> 5%
Crack spacing	From 1.5 to 2.0 m	1.0 m to 1.5 m 2.0 m to 2.5 m	Below 1.0 m or more than 2.5 m

Desirable Aspects of Mechanistic Design of Concrete Pavements

- Easy to use
- Updated regularly
- Calibrated for South African conditions
- Allows for different slab support layers
- Suited to new and rehabilitation design
- Can do "what-if" analyses to study the outcome of different input parameters on the design
- Calculates life cycle costs

Cautionary Aspects for Mechanistic Design of Concrete Pavements

- Default values provided with software need to be adjusted for specific designs

8.3 Summary of Suitability of Structural Design Methods for Concrete Pavements

Table 51 provides a summary of the potential application of the structural design methods discussed in the preceding sections.

Table 51. Potential Application of Structural Design Methods

Method	Potential Application		
	New Design	Rehabilitation Design	
		Initial Assessment – Remaining Life	Rehabilitation Design – Future Structural Capacity
M10	✓	✓	✓
C & CI rigid pavement design method	✓	✓	✓

8.4 Software Package for Concrete Pavements: cncPAVE502

The development of cncPAVE is sponsored by the Cement and Concrete Institute (C&CI) and is distributed through their website, www.cnci.org.za. cncPAVE models the structural deterioration of plain, dowel jointed, continuously reinforced concrete and UTCRCP. The following damage mechanisms are provided for:

- Percentage of **shattered** concrete surface
- Percentage of **pumping** concrete surface
- Percentage **faulting** in plain and dowel jointed concrete
- **Crack spacing** in continuously reinforced concrete

cncPAVE also models the life-cycle cost of concrete pavements. Table 52 summarises the functionality of the software. The current version at the time of publishing this chapter was cncPAVE502.

Table 52. Functionality of the cncPAVE502 Software Package

Application Area	Aspect/Functionality	Comment
Design traffic estimation	Traffic data options	Weigh-in-motion (WIM) axle load spectra are used. Typical default spectra are included if data are not available.
	Pavement load sensitivity	From calculated pavement response under load spectra and empirical performance prediction.
	Design traffic estimation	Calculated from the traffic input data.
Design investigation	Input data	Pavement layer characteristics and costs, traffic loading data and environmental condition.
	Data analysis options	Sensitivity analysis using the "What if" facility.
	Data import, export and reporting	Summary report containing input and output generated and exported via the "Write report" facility.
Pavement analysis and modelling	Analysis engine	Equations developed from FE analysis, Westergaard theory, AASHTO and South African experience.
	Analysis options	Analyze the effect of variation in input data on the outcome by using the "What if" facility.
	Material data and damage models	User defined material properties and calibration coefficients to generate output distributions for different damage types.
	Structural capacity estimation	Progressive distress, through a recursive simulation scheme.
Simulation schemes	Monte Carlo variability simulation	Variability in material properties used to arrive at the predicted percentage area failed with loading or time.
	Recursive damage model	Available as part of the Monte Carlo simulation procedure.
	Riding quality deterioration	Available, predicts IRI change over time.

9. STRUCTURAL CAPACITY ESTIMATION: CONCRETE BLOCK PAVEMENTS

The methods that are available for designing concrete block pavements are described in detail in Concrete Block Paving Book 2: Design Aspects published by the Concrete Manufacturers Association (CMA, 2007). The methods are divided into the following four categories:

- Equivalent thickness concept
- Catalogue design method
- Research based design methods
- Mechanistic design methods (Lockpave and blokPAVE)

9.1 Equivalent Thickness Concept

The equivalent thickness concept assumes that the pavement can be designed with established flexible pavement design procedures, and that the blocks and bedding sand substitute an equivalent part of the conventional design. Table 53 gives a summary of the various values of equivalent substitution used in Argentina, Australia, UK and USA. Using the equivalency factor, the block pavement is designed with well-established flexible pavement design procedures, incorporating a measure of subgrade strength, such as the Californian Bearing Ratio (CBR).

Table 53. Summary of Various Factors of Equivalent Substitution

Country	Concrete Block Paving is Equivalent to…
Argentina	2.5 times thickness of granular subbase
Australia	2.1 – 2.9 times thickness of crushed rock base 1.1 – 1.5 times dense graded asphaltic concrete
USA Corps of Engineers	165 mm cover 2 – 2.85 times thickness of granular base
United Kingdom	225 mm of soil cement 160 mm of rolled asphalt

This design approach assumes that block paving responds to traffic in a similar manner to conventional flexible pavements and that, consequently, there is no impediment to the use of established design procedures. However, this is not strictly correct. The advantages particular to block paving, such as the development of progressive stiffening and lockup, the ability to tolerate large transient deflections, and the ability to spread the load, thus reducing the stress below the bedding sand, are not recognized.

The equivalent thickness concept is not generally used in South Africa for designing block pavements.

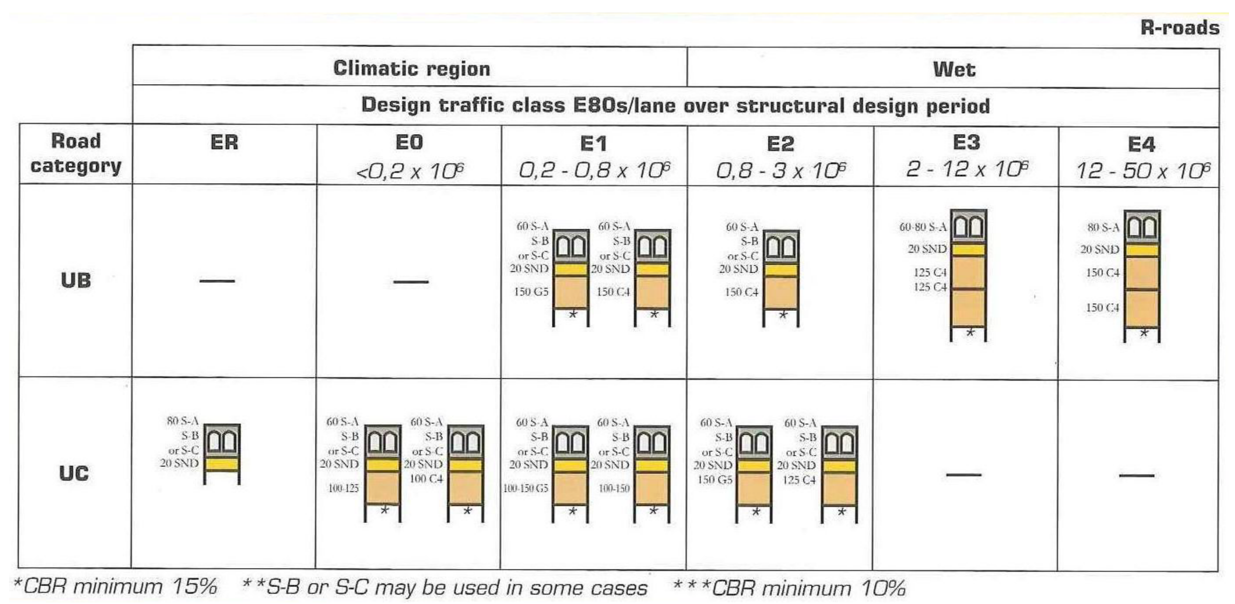
9.2 Catalogue Design Method

With the catalogue design method, the blocks and base thickness are selected based on experience of road construction on subgrades similar to that under consideration. Where experience is extensive, as in Europe, this simple approach can yield satisfactory results. The design procedures are often presented as a design catalogue, which encapsulates local knowledge, but tends to make little distinction between different subgrade conditions or wheel loads.

In South Africa, these design manuals are based on catalogue designs:

- **Draft UTG2** (1987): Structural Design of Segmental Block Pavements for Southern Africa
- Guidelines for the **Provision of Engineering Services** in Residential Townships. (Community Development, 1983.)

In all cases, the road is classified in terms of traffic volume (cumulative E80s), traffic type (residential or industrial) and climatic conditions. Once the road has been classified, the catalogue can be used to select the pavement design. Figure 51 is a typical design taken from UTG2. The material classes specified for the pavement design are as per TRH14. The catalogue method lacks flexibility in that only subgrade strength of CBR = 10 or 15 is accommodated, and often yields a less than optimal pavement design. Experience with the catalogue in Figure 51 is that the pavement structures are thinner than required.



Note:

1. The pre-1995 TRH4 traffic classes are used
2. E3 traffic range is 3 – 12 million E80s and not 2 – 12 as indicated

Figure 51. Typical Catalogue Design of Concrete Block Pavements

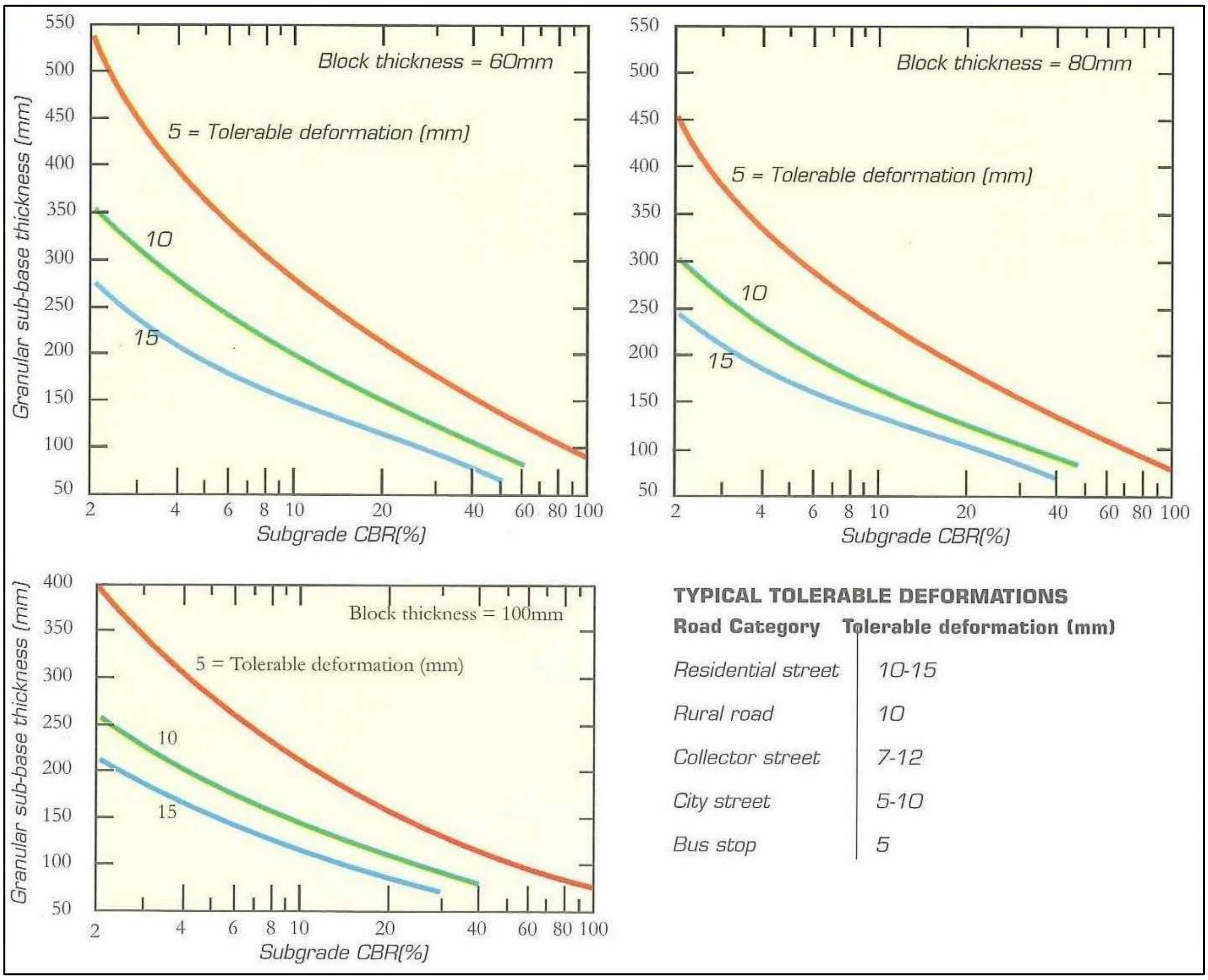
9.3 Research Based Design Methods

Although many engineers have used tests of prototype interlocking concrete pavements to obtain materials equivalencies or substitution ratios, only one design method appears to be wholly based on accelerated trafficking tests. This is the method developed by Shakel at the University of New South Wales for the Cement and Concrete Association of Australia, first published in 1979. Subsequently, following trafficking tests in South Africa, designed in part to verify the procedure, the method was slightly revised (Shakel, 1979 and 1980). The method is restricted to block pavements subjected to highway loadings, and which incorporate unbound granular bases.

It is possible to use accelerated trafficking tests of full-scale prototype block pavements to develop statistically-based models to relate, for a given subgrade strength, the block and base thickness to measures of performance, such as rut depth. These models have been extended to cover the full range of subgrade conditions using mechanistic analyses. Typical design curves are shown in Figure 52. This method has been used successfully in a variety of climates since the late 1970's, but has been replaced by mechanistic procedures similar to those described in Section 9.4.

Reference for Block Pavements

A good reference for block pavements is: Brian Shakel, **"Design and Construction of Interlocking Concrete Block Pavements",** 1990.



TYPICAL TOLERABLE DEFORMATIONS

Road Category	Tolerable deformation (mm)
Residential street	10-15
Rural road	10
Collector street	7-12
City street	5-10
Bus stop	5

Figure 52. Empirical Design Curves for Concrete Block Pavements

9.4 Mechanistic Design Methods for Block Pavements

Several mechanistic procedures for the design of block pavements have been developed, the first in South Africa. The block pavements were analysed as homogenous isotropic flexible mats overlying a flexible subgrade, with a CBR defined strength. The second mechanistic analysis was reported in Britain in 1979. This method was based on a three-layer linear elastic analysis of the pavement and assumed that conventional criteria for relating subgrade strain to the expected life of an asphaltic pavement could be applied to a block pavement.

Recently, a variety of mechanistic procedures utilising the methodology of conventional flexible pavement design have been developed. Usually these analyses either compute the tensile strains in a bound subbase and relate these to a fatigue life, or, determine the vertical compressive strains in the subgrade or granular subbase to relate to the rutting that develops under traffic. By iterating, the thickness of the various pavement layers may be chosen to achieve both an adequate fatigue life and tolerable levels of rut deformation.

Initially, the most effective application of mechanistic methods was in the design of block pavements incorporating bound subbases, such as lean concrete or cement stabilised granular materials. However, in 1988 Shackel published a comprehensive mechanistic design methodology suited to both bound and unbound subbases. This procedure was designed to be run, in an interactive mode, on a computer. This design method is now available as a computer programme called Lockpave. The method is believed to be an advance on earlier mechanistic procedures in that it completely avoids the need to use concepts of axle load equivalency, but rather analyses and designs each pavement in terms of an appropriate spectrum of axle loads. This is of particular importance in the designing of industrial pavements, which often have to accept a very wide range of wheel loads, vehicle configurations and differing load repetitions for each vehicle type. The positions at which the stresses and strains are calculated in Lockpave are similar to those described for flexible pavements (shown in Section 7.1.5). Examples of design curves for both road pavements and industrial hardstands are given in Figure 53.

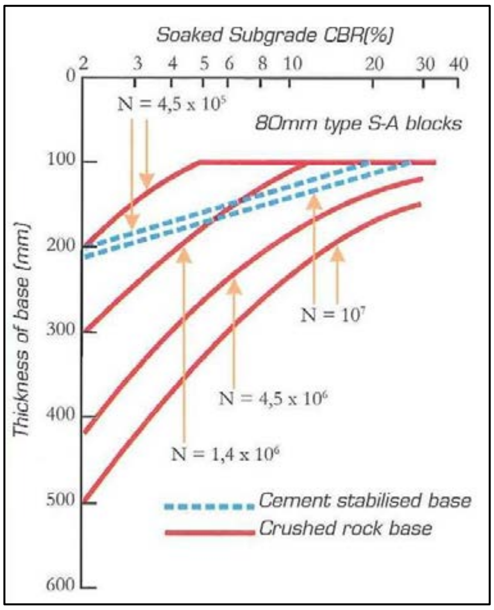


Figure 53. Concrete Block Pavement Design Curves for Road Pavements

9.5 General Aspects for Design of Block Paving

The success of block paving depends on:

- The use of a **stabilised subbase** to provide adequate support to the blocks and the sand blanket.
- A **sand blanket** of 15 to 30 mm in which the blocks embed.
- **Confinement** on the sides of the paved area to hold the blocks in place.

Desirable Aspects of Mechanistic Design for Block Pavements

- Range of methods from simple to more complicated computerised methods
- South African input to design models

Cautionary Aspects for Mechanistic Design for Block Pavements

- Rehabilitation design procedures not included
- Computer based models allow for limited pavement material types

9.6 Software Packages for Concrete Block Pavement Design

9.6.1 Lockpave

Lockpave is distributed together with Permpave, a design method for permeable concrete paving, by the Concrete Manufacturers Association (CMA). The program calculates the critical stresses and strains occurring either on or near the vertical load axis at the bottom of all bound (brittle) layers and at the top of the subgrade. The service life of the pavement is predicted. Conversely, for a designated service life, it is possible to calculate what values of stress and strain can be tolerated and, by trial and error, to determine the combination of layer thicknesses required to not exceed these stresses and strains. Table 54 summarises the functionality of Lockpave.

Table 54. Functionality of the Lockpave Software Package

Application Area	Aspect / Functionality	Comment
Design traffic estimation	Traffic data options	Not modelled separately. Pre-defined axle loads are adjusted by load safety factors to cater for braking, acceleration and cornering
	Pavement load sensitivity	
	Design traffic estimation	
Design investigation	Input data	Pavement layer characteristics and traffic loading
	Data analysis options	Trial and error
	Data import, export and reporting	Outcome reported
Pavement analysis and modelling	Analysis engine	Multi-layer linear elastic model
	Analysis options	No additional options
	Material data and damage models	User defined material properties, environmentally adjusted
	Structural capacity estimation	Adjust base and subbase layer thickness until distress criteria are satisfied
Simulation schemes	Monte Carlo variability simulation	Not included
	Recursive damage model	Not included

9.6.2 blokPAVE

A software program, blokPAVE, for the design of block paving is being developed as part of SAPDM. This program will include new models for prediction of block paving performance. The functionality of the software and analysis options will be the same as cncPAVE, as described in Table 52. The software should be available for distribution in 2013.

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