THE FEDERAL DEMOCRATIC REPUBLIC OF

ETHIOPIA

ETHIOPIAN ROADS AUTHORITY

PAVEMENT REHABILITATION AND ASPHALT OVERLAY DESIGN MANUAL

2013

Foreword

The road network in Ethiopia provides the dominant mode of freight and passenger transport and thus plays a vital role in the economy of the country. The network comprises a huge national asset that requires adherence to appropriate standards for design, construction and maintenance in order to provide a high level of service. As the length of the road network is increasing, appropriate choice of methods to preserve this investment becomes increasingly important.

In 2002, the Ethiopian Roads Authority (ERA) first brought out road design manuals to provide a standardized approach for the design, construction and maintenance of roads in the country. Due to technological development and change, these manuals require periodic updating. This current version of the manual has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the road sector during the last 10 years. Completion of the review and updating of the manuals was undertaken in close consultation with the federal and regional roads authorities and the stakeholders in the road sector including contracting and consulting industry.

This Manual supersedes the Pavement Rehabilitation and Asphalt Overlay Design Manual part of the ERA Design Manuals of 2002. The standards set out shall be adhered to unless otherwise directed by the concerned bodies within ERA. However, I should emphasize that careful consideration to sound engineering practice shall be observed in the use of the manual, and under no circumstances shall the manual waive professional judgment in applied engineering. For simplification in reference this manual may be cited as ERA's *Pavement Rehabilitation and Asphalt Overlay Design Manual - 2013.*

On behalf of the Ethiopian Roads Authority I would like to thank the UK Department for International Development (DFID), Crown Agents and the AFCAP team for their cooperation, contribution and support in the development of this manual. I would also like to extend my gratitude and appreciation to all of the industry stakeholders and participants who contributed their time, knowledge and effort during its development. Special thanks are extended to the members of the various Peer Review Panels, whose active support and involvement guided the authors of the manual and the process.

It is my sincere hope that this manual will provide all users with both a standard reference and a ready source of good practice for the rehabilitation of roads, and will contribute to the cost effective and environmentally sustainable development of our road network. I look forward to the practices contained in this manual being quickly adopted into our operations, thereby making a sustainable contribution to the improved infrastructure of our country.

Comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Road Authority

Preface

The Ethiopian Roads Authority is the custodian of the series of technical manuals, standard specifications and bidding documents that are written for the practicing engineer in Ethiopia. The series describe current and recommended practice and set out the national standards for roads and bridges. They are based on national experience and international practice and are approved by the Director General of the Ethiopian Roads Authority.

This *Pavement Rehabilitation and Asphalt Overlay Manual* forms part of the Ethiopian Roads Authority series of Road and Bridge Design documents.

Companion documents and manuals include the Standard Technical Specifications, Standard Detailed Drawings and Standard Bidding Documents.

The complete series of documents, covering all roads and bridges in Ethiopia, are contained within the series:

1. Route Selection Manual
2. Site Investigation Manual
3. Geotechnical Design Manual
4. Geometric Design Manual
5. Pavement Design Manual Volume I Flexible Pavements
6. Pavement Design Manual Volume II Rigid Pavements
7. Pavement Rehabilitation and Asphalt Overlay Design Manual
8. Drainage Design Manual
9. Bridge Design Manual
10. Low Volume Roads Design Manual
11. Standard Environmental Procedures Manual
12. Standard Technical Specifications.
13. Standard Drawings
14. Best Practice Manual for Thin Bituminous Surfacings
15. Standard Bidding Documents for Road Work Contracts – A series of Bidding Documents covering a full range from large scale projects unlimited in value to minor works with an upper threshold of $300,000. The higher level documents have both Local Competitive Bidding and International Competitive Bidding versions.

These documents are available to registered users through the ERA website: www.era.gov.et

Manual Updates

Significant changes to criteria, procedures or any other relevant issues related to new policies or revised laws of the land or that is mandated by the relevant Federal Government Ministry or Agency should be incorporated into the manual from their date of effectiveness.

Other minor changes that will not significantly affect the whole nature of the manual may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant chapter.

All suggestions to improve the draft Pavement Rehabilitation and Asphalt Overlay Manual should be made in accordance with the following procedures:

1. Users of the manual must register on the ERA website: www.era.gov.et
2. Proposed changes should be outlined on the Manual Change Form and forwarded with a covering letter of its need and purpose to the Director General of the Ethiopian Roads Authority.
3. Agreed changes will be approved by the Director General of the Ethiopian Roads Authority on recommendation from the Deputy Director General (Engineering Operations).
4. All changes will be made prior to release of a new version of the manual.
5. The release date will be notified to all registered users and authorities.

ETHIOPIAN ROADS AUTHORITY
CHANGE CONTROL DESIGN MANUAL

MANUAL CHANGE		This area to be completed by the ERA Director of Quality Assurance
Manual Title:_____ _____		CHANGE NO._____ (SECTION NO. CHANGE NO. _____
Section Table Figure Page	**Explanation**	**Suggested Modification**

Submitted by: Name:_____Designation:_____

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Manual Change Action

Authority	Date	Signature	Recommended Action	Approval
Registration				
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Approval / Provisional Approval / Rejection of Change:

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The Ethiopian Roads Authority (ERA) wishes to thank the UK Government's Department for International Development (DFID) through the Africa Community Access Programme (AFCAP) for their support in developing this Pavement Rehabilitation and Asphalt Overlay Manual. The manual will be used by all authorities and organisations responsible for the provision roads in Ethiopia.

From the outset, the approach to the development of the manual was to include all sectors and stakeholders in Ethiopia. The input from the international team of experts was supplemented by our own extensive local experience and expertise. Local knowledge and experience was shared through a series of meetings of Peer Review Groups comprising specialists drawn from within the local industry which were established to provide advice and comments in their respective areas of expertise. The contribution of the Peer Group participants is gratefully acknowledged.

The final review and acceptance of the document was undertaken by an Executive Review Group. Special thanks are given to this group for their assistance in reviewing the final draft of the document. Finally, ERA would like to thank Crown Agents for their overall management of the project

This manual is based on a review of the methods used in several countries but primarily on the Transport Research Laboratory's Overseas Road Notes No 18, *Pavement Evaluation and Maintenance for Bitumen-surfaced Roads,* Overseas Road Note 19, *A Guide to the Design of Hot Mix Asphalt in Tropical and Sub-tropical Countries* and Overseas Road Note 31, *A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries.* These reference documents and companion TRL documents have drawn on the experience of TRL and collaborating organizations in many tropical and sub-tropical countries, including Ethiopia, over many years.

Other major reference sources include the AASHTO *Guide for Design of Pavement Structures* as revised in 1993 and the Asphalt Institute's publications for asphalt concrete and other hot- and cold-mix types.

As with the other manuals of this series, the intent was, where possible and in the interests of uniformity, to use those tests and specifications included in the AASHTO and/or ASTM Materials references. Where no such reference exists for tests and specifications mentioned in this document, other references are used.

Addis Ababa, 2013

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Glossary of Terms

Adhesion failure	Joint sealant rendered ineffective through loss of adhesion between the sealant and the vertical faces of the concrete in the joint groove.
Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt	In American literature asphalt is another term for bitumen. The term is also commonly used in this way in Ethiopia. In other countries, asphalt is commonly used as shorthand for asphaltic concrete or, indeed, any design of high quality bitumen/aggregate mixture.
Asphalt Concrete	A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mixed and usually placed by means of a paving machine.
Asphalt Surfacing	The layer or layers of asphalt concrete constructed on top of the roadbase, and, in some cases, the shoulders.
Average Annual Daily Traffic (AADT)	The total yearly traffic volume in both directions divided by the number of days in the year.
Average Daily Traffic (ADT)	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
Base Course	The main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Bituminous base courses may also be used (for higher classes of traffic). Materials stabilised with cement or lime may also be contemplated.
Bay	The concrete slab between two joints.
Binder Course	The lower course of an asphalt surfacing laid in more than one course.
Bitumen	The most common form of bitumen is the residue from the refining of crude oil after the more volatile material has been distilled off. It is a very viscous liquid comprising many long-chain organic molecules. For use in roads it is practically solid at ambient temperatures but can be heated sufficiently to be poured and sprayed. Some natural bitumens can be found worldwide that are not distilled from crude oil but the amounts are very small.
Borrow Area	An area within designated boundaries, approved for the purpose of obtaining borrow material. A borrow pit is the excavated pit in a borrow area.
Borrow	Any gravel, sand, soil, rock or ash obtained from borrow areas, dumps or sources other than cut within the road prism and which is used in

Material	the construction of the specified work for a project. It does not include crushed stone or sand obtained from commercial sources.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 0.30 m or more.
Bound Pavement Materials	Pavement materials held together by an adhesive bond between the materials and a binding material such as bitumen.
Camber	The convexity given to the curved cross-section of a roadway.
Capping Layer	(Selected or improved subgrade). The top of embankment or bottom of excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better quality subgrade material imported from elsewhere or subgrade material improved by stabilisation (usually mechanical), and may also be considered as a lower quality sub-base.
Carriageway	That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.
Cohesion failure	Joint sealant within which cracks have occurred at right-angles or parallel to the joint groove.
Compression ('blow up') failure	Crushing failure of a slab at a joint or crack caused by excessive compressive stress resulting from thermal expansion of a pavement containing joints that have locked up and/or have filled with incompressible detritus.
Construction joint	A joint made in a concrete pavement at the end of a working day. Similar joints may have to be introduced in an emergency when plant breaks down or paving is stopped by bad weather.
Contraction Joint	A joint normally placed at recurrent intervals in a rigid slab to control transverse cracking.
Cross-Section	A vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centreline.
Crossfall	The difference in level measured transversely across the surface of the roadway.
Culvert	A structure, other than a bridge, which provides an opening under the carriageway or median for drainage or other purposes.
Cutting	Cutting shall mean all excavations from the road prism including side drains, and excavations for intersecting roads including, where classified as cut, excavations for open drains.
Chippings	Stones used for surface dressing (treatment).

CRCP	Continuously Reinforced Concrete Pavement. A concrete running surface that does not include transverse joints but in which the slab contains sufficient reinforcing steel to control transverse thermal contraction/shrinkage cracks.
Cracks	Structural fractures in the pavement categorised as, Wide (unspalled width exceeding 1.5 mm) Medium (unspalled width between 0.5-1.5 mm) Narrow (unspalled width less than 0.5 mm) or hairline (present, but detectable only with difficulty).
Crack-and-seat	Process in which a distressed concrete pavement is broken into small slabs and rolled before overlaying in order to inhibit reflective cracking of the subsequent overlay.
Crack inducer	An insert put in the concrete to create a plane of weakness where subsequent thermal contraction/shrinkage cracking will occur in a controlled manner.
Deformed Bar	A reinforcing bar for rigid slabs conforming to "Requirements for Deformations" in AASHTO Designations M 31M.
Deep spalling at joints	Multiple cracking and breaking away of concrete adjacent to a joint, often semi-circular in plan and extending down below the bottom of the joint groove.
Design Period	The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.
Diverted Traffic	Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
Dowel	A load transfer device in a rigid slab, usually consisting of a plain round steel bar.
Equivalent Standard Axles (ESAs)	A measure of the potential damage to a pavement caused by a vehicle axle load expressed as the number of 8.16 metric tonnes single axle loads that would cause the same amount of damage. The ESA values of all the traffic are combined to determine the total design traffic for the design period.
Equivalency Factors	Used to convert traffic volumes into cumulative standard axle loads.
Equivalent Single Axle Load (ESA)	Summation of equivalent 8.16 ton single axle loads used to combine mixed traffic to design traffic for the design period.
Escarpment	Escarpments are geological features that are very steep and extend laterally for considerable distances, making it difficult or impossible

	to construct a road to avoid them. They are characterised by more than 50 five-metre contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 50%.
Expansion Joint	A joint located to provide for expansion of a rigid slab without damage to itself, adjacent slabs, or structures.
Faulting	An abrupt change of level in the running surface at a joint between two bays, also known as stepping.
Fill	Material of which a man-made raised structure or deposit such as an embankment is composed, including soil, soil-aggregate or rock. Material imported to replace unsuitable roadbed material is also classified as fill.
Flexible Pavements	Includes primarily those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to differentiate between asphalt and Portland cement concrete pavements.
Formation Level	Level at top of subgrade.
Foundation	All materials up to the top of sub-base.
Generated Traffic	Additional traffic which occurs in response to the provision of improvement of the road.
Grading Modulus (GM)	The cumulative percentages by mass of material in a representative sample of aggregate, gravel or soil retained on the 2.00 mm, 0.425 mm and 0.075 mm sieves, divided by 100.
Grinding	Mechanical removal of out-of-tolerance surface concrete projecting above the required finished level.
Heavy Vehicles	Those having an unloaded weight of 3,000 kg or more.
High-early-strength concrete	Pavement-quality concrete designed to achieve 25 MPa at ages from three days to less than six hours, depending on the urgency of opening the pavement to traffic
Hot mix asphalt (HMA)	A generic name for all high quality mixtures of aggregates and bitumen that use the grades of bitumen that must be heated in order to flow sufficiently to coat the aggregates. Includes Asphaltic Concrete, Dense Bitumen Macadam and Hot Rolled Asphalt.
Induced crack	A full-depth crack intentionally induced in a concrete slab by providing a joint groove in the top surface and (sometimes) a crack-inducing insert in the underside.
Joint groove	The groove provided at the top of a joint to receive the sealant.

Joint seal	Flexible sealant material that adheres to the vertical faces of the joint groove to exclude water and detritus while accommodating opening and closing of the joint.
Load transfer	The distribution of load to an unloaded slab that occurs when the slab on the other side of a joint is loaded.
Load transfer efficiency	The ratio (expressed as a percentage) of vertical deflection of an unloaded slab (adjacent to a joint) to the deflection of an abutting loaded slab.
Longitudinal Joint	A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.
Maintenance	Routine work performed to keep a pavement, under normal conditions of traffic and forces of nature, as nearly as possible in its as-constructed condition.
Mountainous (Terrain)	Terrain that is rugged and very hilly with substantial restrictions in both (terrain) horizontal and vertical alignment. Defined as having 26 to 50 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.
Normal Traffic	Traffic which would pass along the existing road or track even if no new pavement were provided.
Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a levelling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.
Partial depth repair	Repair to a spalled joint at which the depth of spalling exceeds the depth of the joint groove but does not exceed one-third of the slab depth; repair at a location remote from joints where the depth of repair exceeds 40 mm but does not exceed one-third of the slab depth. The repair is fully bonded to, and becomes monolithic with, the original concrete slab.
Partial reconstruction	Pavement rehabilitation that re-uses some of the existing layers.
Pavement Layers	The layers of different materials which comprise the pavement structure.
Plastic cracking	Surface cracks that form before the concrete hardens in a pattern of short cracks usually approximately parallel to each other, oriented diagonally to the bay sides and not extending to the edges of the slab.
Polymer mortar	A cementitious mortar modified by adding a polymer such as styrene butadiene rubber.
Project Specifications	The specifications relating to a specific project, which form part of the contract documents for such project, and which contain supplementary

	and/or amending specifications to the standard specifications.
Pumping	The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.
Quarry	An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.
Reconstruction	The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.
Recycling	The reuse, usually after some processing, of a material that has already served its first-intended purpose
Reflective cracks	Cracks in a concrete slab or asphalt overlay induced by movement in joints or cracks in an underlying layer.
Rehabilitation	Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and pre overlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.
Reinforcement	Steel embedded in a rigid concrete slab to resist tensile stresses and detrimental opening of cracks.
Rigid Pavement	A pavement structure which distributes loads to the subgrade having, as the main load bearing course, a Portland cement concrete slab of relatively high-bending resistance.
Roadbase	A layer of material of defined thickness and width constructed on top of the sub-base, or in the absence thereof, the subgrade. A roadbase may extend to outside the carriageway.
Road Bed	The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.
Road Bed Material	The material below the subgrade extending to such depth as affects the support of the pavement structure.
Road Prism	That portion of the road construction included between the original ground level and the outer lines of the slopes of cuts, fills, side fills and side drains. Not including sub-base, roadbase, surfacing, shoulders, or existing original ground.
Roadway	The area normally travelled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.
Rolling (Terrain)	Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment. Defined as terrain with 11 to 25 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 3 and 25%.

Rubbilise	Process by which a distressed concrete pavement is broken up to such an extent that there is no significant aggregate interlock between adjacent pieces (see also crack-and-seat).
Side Fill	That portion of the imported material within the road prism which lies outside the fills, shoulders, roadbase and sub-base and is contained within such surface slopes as shown on the Drawings or as directed by the Engineer. A distinction between fills and side fill is only to be made if specified.
Side Drain	Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.
Shallow spalling at joints	Cracking, breaking away or erosion of concrete alongside the joint and extending no deeper than the depth of the joint groove.
Stabilisation	The treatment of materials used in the construction of the road bed material, fill or pavement layers by the addition of a cementitious binder such as lime or Portland Cement or the mechanical modification of the material through the addition of a soil binder or a bituminous binder. Concrete and asphalt shall not be considered as materials that have been stabilised.
Stepping	See faulting.
Sub-base	The layer of material of specified dimensions on top of the subgrade and below the roadbase. It is the secondary load-spreading layer underlying the base course. It usually consists of a material of lower quality than that used in the base course and particularly of lower bearing strength. Materials may be unprocessed natural gravel, gravel-sand, or gravel-sand-clay, with controlled gradation and plasticity characteristics. The sub-base also serves as a separating layer preventing contamination of the base course by the subgrade material and may play a role in the internal drainage of the pavement.
Subgrade	The surface upon which the pavement structure and shoulders are constructed. It is the top portion of the natural soil, either undisturbed (but re-compacted) local material in cut sections, or soil excavated in cut or borrow areas and placed as compacted embankment.
Subsurface Drain	Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.
Surface Treatment	The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and crushed stone chippings.
Surfacing	This comprises the top layer(s) of the flexible pavement and consists of a bituminous surface dressing or one or two layers of premixed bituminous material (generally asphalt concrete). Where premixed

	materials are laid in two layers, these are known as the wearing course and the binder course.
Sympathetic crack	A crack induced in a slab by movement at a joint or crack in an abutting slab.
Thin bonded repair	A shallow cementitious patch - usually to a joint groove and not more than 40 mm deep - that is fully bonded to, and becomes monolithic with, the original concrete slab
Tie Bar	A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.
Traffic Lane	Part of a travelled way intended for a single stream of traffic in one direction, which has normally been demarcated as such by road markings.
Traffic Volume	Volume of traffic usually expressed in terms of average annual daily traffic (AADT).
Typical Cross-Section	A cross-section of a road showing standard dimensional details and features of construction.
Unbound Pavement Materials	Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.
Wearing Course	The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.
Welded Wire Fabric	Welded steel wire fabric for concrete reinforcement

Abbreviations

AADT	Average Annual Daily Traffic
AASHO	American Association of State Highway Officials (previous designation)
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt concrete
ACV	Aggregate Crushing Value – a measure of aggregate strength
ASTM	American Society for Testing Materials
BS	British Standard
CBR	California Bearing Ratio (as described in AASHTO T 193)
CRCP	Continuously Reinforced Concrete Pavement
DCP	Dynamic Cone Penetrometer
m_2, m_3	Drainage coefficients. Factors used to modify *layer coefficients* in flexible pavements to take account of climate, the effectiveness of internal pavement drainage and moisture sensitivity.
ERA	Ethiopian Road Authority
ESA	Equivalent standard axles. A measure of the damaging effect of vehicle axles (*see ERA Pavement Design Manual Volume I*).
FWD	Falling Weight Deflectometer
GM	Grading Modulus
HMA	Hot Mixed Asphalt
ICL	Initial Consumption of Lime test
IRI	International Roughness Index
LAA	Los Angeles Abrasion Value – a measure of aggregate strength
MDD	Maximum Dry Density
NDT	Non destructive test
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
a_1, a_2, a_3	Strength coefficients. The empirical strength coefficients used for weighting the contribution of each layer of the pavement to the overall structural number (SN). They are modified by the drainage coefficients (see above).
NDT	Non destructive testing
PCC	Portland Cement concrete
PMS	Pavement management system

RRD	Representative rebound deflection
S1 to S6	Subgrade strength classes used to characterize the subgrade in pavement design (see *ERA Pavement Design Manual Volume I Flexible Pavements*).
SN and MSN	Structural Number and Modified Structural Number. An index of overall pavement strength based on the thicknesses and strengths of each pavement layer.
SN_{eff} and MSN_{eff}	Effective Structural Number of an existing pavement.
T1 to T8	Traffic classes used to characterize the anticipated traffic in terms of ESA for flexible pavement design purposes.
h_1, h_2, h_3	Thicknesses of pavement surface, base and sub-base layers (existing or required)
TRL	Transport Research Laboratory, UK (formerly TRRL)
TRRL	Transport and Road Research Laboratory, UK
VOC	Vehicle Operating Costs
VFB	Voids Filled with Bitumen
VIM	Voids in the Mix
VMA	Voids in the Mineral Aggregate

1. INTRODUCTION

1.1 Purpose and scope of the manual

As the network of paved roads in Ethiopia grows, a gradual shift in emphasis will occur from new design and construction to maintenance and rehabilitation of existing roads. The design of new pavements is covered in the *ERA Pavement Design Manual Volumes I and II*. This manual provides guidance on maintenance and rehabilitation of existing bituminous-surfaced roads and also of rigid (concrete) pavements. It describes methods of pavement evaluation designed to establish the nature, severity and extent of the road deterioration, gives guidance on the use of non-destructive and destructive pavement tests and describes how the results of these tests can be interpreted, both to identify the causes of deterioration and to assess the strength of the existing road. Following this evaluation process, the manual provides guidance on the selection and design of repair and rehabilitation procedures and methods.

The manual is intended primarily for highway engineers who are responsible for maintaining roads, but the techniques and principles will be of interest to others with an interest in road infrastructure.

The repair and upgrading of earth and gravel roads to paved road standards is dealt with in ERA's *Low Volume Roads Design Manual.*

1.2 Principles

All roads deteriorate with time as a result of traffic and environmental effects. The deterioration may be relatively easy to correct or may require major works, depending on the causes and extent of deterioration. The works processes for keeping roads in good condition are often subdivided into the following categories:

(i). Routine maintenance – maintenance that needs to be done at relatively short intervals such as cutting grass and cleaning drainage ditches.

(ii). Emergency maintenance - maintenance that has to be done immediately as a result of an unexpected problem e.g. clearing a rock fall.

(iii). Periodic maintenance – maintenance that needs to be done at longer intervals of, say, 5-10 years e.g. surface dressing. Usually this category excludes structural strengthening.

(iv). Rehabilitation – this term is commonly used when structural strengthening is required and includes overlaying.

(v). Reconstruction – this usually means that at least one layer of the pavement needs to be reprocessed.

(vi). Upgrading – this usually means that strengthening and some realignment are required.

The main distinguishing feature of the categories of maintenance is that the costs increase steadily from 'routine maintenance' up to 'upgrading'. As a result the responsibility for each operation may differ. However, it can be seen that there is considerable overlap between the categories and in the process of keeping the road in good condition it is not helpful to be concerned about precise definitions. Roads can deteriorate (and fail) in many different ways and the repairs that they require depend on the *causes* of deterioration. This is the guiding principle of pavement rehabilitation namely that '*the repairs are determined*

by the cause or causes of the deterioration and the degree to which the deterioration has progressed'. Identifying these causes is therefore of paramount importance. Applying the wrong remedial treatment could be a waste of time and money. Thus the basic principle is to evaluate or assess the road to diagnose the cause (or causes) of deterioration and the severity of the deterioration so that the correct remedial treatment can be applied. The appropriate remedial treatment can fall into any of the above categories except possibly the first (routine maintenance).

Until the diagnosis procedure is complete, the size and scale of the required works and therefore the appropriate budget cannot be determined.

Most methods of rehabilitation design are based on the assumption that repairs are carried out in good time, namely before deterioration has progressed too far; in other words, when a road has reached a '*critical*' rather than a '*failed*' condition. Under these circumstances relatively routine methods of strengthening can be used and the risk element is low. When road pavements have deteriorated beyond this critical state, rehabilitation strategies tend to be very conservative, often requiring complete reconstruction. Methods that attempt to make better use of the existing 'strength' of the existing pavement when it has deteriorated beyond the critical state rely very largely on engineering judgement and therefore are seen to represent a higher degree of risk. Given the nature of the problem, this is perhaps inevitable. To quote the AASHTO Design Guide for Pavement Structures (Chapter III Section 2.3):

> "*Pavement rehabilitation is as much an art as a science. With the exception of certain overlay models presented elsewhere, there are no definitive equations, guides or step-by-step procedures that one can use to 'cook book' a proper rehabilitation design. Therefore a considerable amount of both analysis and engineering judgement must be applied to each project".*

In other words a great deal of thinking and engineering judgement and skill is required if rehabilitation designs are to be optimised to provide the most cost effective solutions.

For the design of rehabilitation or strengthening, the condition of the existing layers of the pavement plays a vital part. It is essential that the assessment or evaluation procedure provides sufficient data for the engineer to identify the modes of deterioration and their causes. In this way the appropriate rehabilitation treatments can be identified and designed.

1.3 Reliability and risk

The design process is based on assumptions about:

a) The design of adequate drainage.
b) Climatic factors, particularly the temperature and rainfall regime of the road.
c) Adequate quality control during construction.
d) Adequate maintenance thereafter.

However, even with a high level of knowledge and control of all these factors, the performance of road pavements is very variable. For example, the range of performance of similar road structures at the AASHO Road Test (now AASHTO) was such that the road sections that performed best carried 4 to 5 times more traffic than those that performed worst.

It must also be borne in mind that for most types of structural deterioration, only a very small percentage of the overall surface area of a road needs to display any noticeable deterioration for the road to be considered to be in poor condition. We are dealing not with average values of pavement parameters but with the extremes of the performance histogram, in other words the behaviour of the 'weakest' few percent of the road.

With such uncertainties, it is necessary to design roads based on statistical ideas of reliability and risk. The essence of reliability analysis is to choose an appropriate level of reliability based on the pavement evaluation data available and the class of road under consideration. [It is a corollary of this that although the average performance of a road network can be predicted accurately, it is much more difficult to predict how a particular length of road will behave].

1.4 Reconstruction or strengthening

An important element in the selection of rehabilitation treatments is the decision to either strengthen by overlaying or to choose some form of partial or full reconstruction. The evaluation procedure is designed to help with this decision, but there are several over-arching principles that apply to these judgements. These are described briefly below.

1.4.1 *Soil and slope stability – geotechnical problems*

One of the most common reasons for pavement reconstruction in Ethiopia is pavement failure caused by underlying geotechnical problems. Pavements are primarily designed to protect the underlying and weaker layers from failure caused by traffic induced stresses. Although the structural design of pavements can cope with structurally weak subgrades, it is necessary to assume that the subgrade soils are inherently stable and not moving en masse through land-sliding processes or through large scale differential settlement. Problems of soil instability cannot be solved by the thickness design of pavement layers alone; solving such problems will inevitably require some form of geotechnical solution.

1.4.2 *Treatment of deterioration and failure within the pavement structure*

The types of deterioration that are occurring within the pavement must be addressed adequately. For example, one of the most common problems afflicting flexible pavements is cracking of the asphalt surface. The rapid reappearance (or reflection) of these cracks through an asphalt surface that is laid directly over them is a well known phenomenon that must be solved as part of the rehabilitation process. If this is not done, the long-run consequences will be considerably more expensive. Another example concerns structural failures that are occurring (or have already occurred) deeper within the pavement. Ideally, rehabilitation should be carried out at the first signs of such failures in order to prevent their more widespread occurrence, but the point is that they must be treated; simply covering them up merely delays reconstruction that will inevitably become necessary. Deep patching is usually required and if too much deep patching is needed, then reconstruction is usually indicated.

1.4.3 *Correcting drainage problems*

If the drainage problems are serious they will normally have resulted in failures somewhere within the pavement. Such drainage problems must be addressed as part of rehabilitation. Sometimes this may require reconstruction to provide access to susceptible

layers, even though this may not be strictly necessary to solve the particular problem resulting from the poor drainage itself.

1.4.4 *Non-standard construction techniques, design and sub-standard materials*

Quantifiable risks can only be based on options that have been tried before and for which there are precedents and an element of predictability. For example, if the properties of a road base or sub-base are found to lie outside any recognised specifications (or reliable research results that have not yet been incorporated into national standards and specifications) it is not possible to predict performance and to guarantee adequate pavement life. Under these circumstances, partial or full reconstruction to replace or modify the materials to meet specifications is the only option that can be considered. This category includes lack of adequate shoulders or other suitable edge support, which leads to shear failures along the outer wheel paths.

1.4.5 *Thickness of overlays required to cope with basic weakness in the structure*

Even if no serious failures have been experienced and overlaying is the logical method of strengthening, the required thickness can sometimes be excessive and it may be more cost effective to strengthen one or more of the existing layers of the pavement or to reconstruct completely.

1.4.6 *High surface roughness, deformations and general unevenness*

The high level of roughness observed on many roads is the result of many processes including settlement, poor maintenance work, poor re-instatement of utility trenches and poor initial construction where roads have literally simply ‘evolved’ over many years. Although some forms of deformation are indicative of soil instability and other forms of failure, there are also circumstances where the underlying structure is now stable. However, the nominal thickness of overlay required to correct these shape problems and the construction difficulties associated with so doing often make full or partial reconstruction a more economic option.

1.4.7 *Utility trenches, particularly water pipes*

Trenches for water pipes are often not backfilled and compacted properly leading to deformation and poor surface shape – in effect, very sub-standard bases and sub-bases have been introduced within a thin strip down the road. To correct this defect the water pipes need to be re-laid properly. This can be done without complete reconstruction, except if two trenches occur, one on either side of the road. In this case reconstruction may be the best option, especially if the water pipes have regularly leaked and created additional damage to the pavement structure.

1.4.8 *Buildability*

A pavement survey and diagnostic analysis will identify where different treatments are required, but these may not be easy to build. For example, there may be too many changes of rehabilitation type within a short length of road e.g. several lengths of deep reconstruction adjacent to sections requiring thin overlays. This results in very difficult site logistics. Issues of buildability can only be addressed after the rehabilitation treatments appropriate for each section of road have been defined.

The cost of the works is often closely connected with buildability because the contractor's price for some forms of rehabilitation or pre-treatment will reflect his view of this aspect. When options are costed, the need for large areas of deep patching within one section inevitably means that it is better to reconstruct the complete section.

Whist there may be occasions when none of the reasons given above will seem sufficient in themselves to justify reconstruction, the defects often occur simultaneously. The combination of defects may increase the risks to such an extent that reconstruction is the only logical option. This decision is based on the engineering judgement of those responsible for the pavement evaluation.

1.5 General approach

Diagnostic procedures follow a logical pattern. The first stage requires simple and therefore inexpensive measures. These are essentially a desk study of existing information about the road followed by a visual survey (this may also require the use of some inexpensive hand-operated equipment). If these prove sufficient to determine the causes of deterioration and the appropriate remedy (or remedies), only a limited amount of more expensive 'checking' is required.

The Ethiopian Pavement Management System provides the first stage of the process. Regular routine pavement surveys are carried out that provide data that are sufficient for the initial screening of the roads or for feasibility studies at project level; but these surveys are rarely sufficient for detailed design. Indeed, when the PMS is fully implemented, the non-destructive testing required for the second stage (primarily deflection surveys and DCP surveys) may also be routinely carried out.

For design purposes it is usually necessary to carry out additional non-destructive testing to be certain of any diagnosis and to ensure that any other potential problems are identified at an early stage. Finally, destructive testing (test pits and samples collected for laboratory testing) may be necessary, if only for confirmation of the interpretation of the data collected non-destructively.

One of the most important aspects of the process is identifying the cause or causes of deterioration from the survey data. This manual deals with this stage in detail by means of a series of 'process' charts that link the symptoms of the deterioration to the causes and hence to the possible remedial treatments. The method is that described in TRL's *Overseas Road Note 18*.

All of the information is used to identify alternative maintenance or rehabilitation strategies which can be considered in a subsequent project appraisal. For relatively straightforward maintenance this will usually be very simple, but major rehabilitation projects will normally require an extensive economic appraisal process that also considers the social and environmental impact of each alternative. The economic viability is normally assessed using a road transport investment model such as HDM-4 (see references) or a simpler alternative. The engineering input for such a process is also provided by the pavement evaluation surveys described in this manual.

The data and their analysis are primarily directed towards a rehabilitation design on an individual project basis, but are also meant to be used, on a more general basis, as inputs to ERA's Pavement Management System (PMS).

1.6 Organization of the manual

The manual is divided into two parts. The first part (Part A) is for flexible pavements and the second part (Part B) is for rigid (concrete pavements). However, many aspects are common to both and, to avoid repetition, these are all contained in Part A. Part B should be considered a supplement.

In Chapter 2, the user of the manual is taken through the steps required to collect an array of data relevant to the rehabilitation project. These data include elements from different sources (e.g. past road inventories, as-built pavement structures, unit prices, etc.). They are gathered as part of a desk study, and preferably prior to initiating the collection of additional data in the field. An important aspect of this task consists of collecting relevant traffic data.

Details of the data collection that may need to be carried out in the field are presented in Chapter 3. Of primary importance among these are a detailed pavement condition survey and a deflection survey. Other tasks are field testing and sampling of materials, drainage survey, etc. The methods presented cover both flexible and rigid pavements.

Chapter 3 also outlines the method used to verify or confirm the need for rehabilitation (as opposed to maintenance) for a particular project. The method uses newly collected data, which are normally more complete, recent and reliable than the data available at the desk study stage.

In Chapter 4 the treatment of localised surface defects is dealt with. Once the need has been diagnosed these do not require extensive investigation.

Chapter 5 describes the important technique of representing the data by means of performance charts whereby the pavement characteristics are plotted against chainage. Such charts are used for several aspects of the evaluation, not only to identify uniform sections of road but to interpret the causes of deterioration and therefore the potential methods of rehabilitation.
Chapter 6 describes the non-destructive structural tests that may be required, namely deflection tests and tests with the dynamic cone penetrometer to determine layer thicknesses and strengths, and also the use of test pits to obtain more detailed data.

At this stage all relevant data should be available. Chapter 7 describes how to identify sections of road that are sufficiently uniform to be treated in the same way.

Chapter 8 is the key chapter where the causes of deterioration in each uniform section are identified together with the possible methods of rehabilitation or repair.

Chapter 9 is concerned with designing the rehabilitation treatments themselves whilst Chapter 10 provides advice on how to select the best option if several are feasible.

Part B of the manual deals with rigid pavements and comprises five chapters. The introduction in Chapter 11 is followed by a defect catalogue of photographs. Chapter 13 describes how to identify the main defects and their causes. Several of the important methods of testing a road that are used for flexible pavements are not possible or not suitable when dealing with concrete pavements, for example, excavating test pits and

driving a DCP through to the subgrade, hence more emphasis has to be placed on the visual condition survey.

Having identified the problems, Chapter 14 describes how to determine the most appropriate remedial techniques. Finally Chapter 15 describes how to use overlays for the rehabilitation of concrete pavements.

The Appendices provide additional detailed information about many aspects of the rehabilitation process.

1.7 The pavement management context

Pavement management encompasses all the activities involved in the planning, design, construction, maintenance, evaluation and rehabilitation of the road network of the country. A pavement management system (PMS) is a set of tools or methods that assist decision makers in finding optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a given period of time. The function of a PMS is to improve the efficiency of decision-making, expand its scope, provide feedback on the consequences of decisions, facilitate the coordination of activities within the agency, and ensure the consistency of decisions made at different management levels within the agency.

A PMS usually works at three levels: strategic planning, network management, and at individual project level.

At the strategic level the system provides much of the information needed for long term planning and for monitoring and setting strategic targets. This may include increasing the proportion of roads that are rated as being in good condition, improving road safety and so on. Strategic targets are usually concerned with time scales of five years or more.

At the network level a PMS provides information concerning the selection and preliminary design of projects to be undertaken in the near future, usually the next financial year. It allows comparisons to be made between alternative maintenance and rehabilitation options for a portfolio of roads deemed to be in need of further investment, so that the optimum choice of projects to be included in the next annual programme can be selected based on agreed criteria and available budgets.

Once the projects have been selected a PMS is used for evaluating the detailed pavement condition of each road and for designing in more detail the technical solutions for maintenance or rehabilitation, again according to agreed criteria. Once again, by comparing the benefits and costs associated with several alternative activities, an optimum strategy is identified that will provide the desired benefits at the least cost over a selected analysis period.

In Ethiopia the PMS is operational and is subject to continuous improvement. It provides some data for use in rehabilitation design (see Chapter 2) but at project level the information is unlikely to be sufficiently comprehensive. Chapter 10 provides further information about selecting the preferred rehabilitation options based on the same principles as are used in the PMS, but at a project level where much more detailed design and performance data are available.

2. DATA COLLECTION

2.1 Overview

This chapter provides a general review of the data that might be required for the evaluation of road pavements and the design of rehabilitation measures. It is primarily concerned with data that are available beforehand, and gives guidance on the 'desk study' that should be carried out before field work is undertaken. Such a desk study can influence the choice of field investigations that are needed and can improve the effectiveness of the site investigations. Detailed descriptions of how to carry out site investigations and to undertake the various individual tests that are usually required are contained in the Appendices.

The data collection includes a search through the PMS Branch records. Conversely, the results of the data collection exercise should be made available to the PMS Branch. This includes the results of the surveys described in Chapter 3 to the extent that they can update and/or complete the PMS Branch records.

2.2 PMS Branch records

These records are among the first sources of data that should be obtained and reviewed. PMS data are regularly updated but the road condition data are usually obtained at less frequent intervals along the road than required for rehabilitation design purposes. However these data are invaluable in the first stages of the investigation. They fall into five categories, as follows:

1) **Fixed inventory data**. These data include carriageway and shoulder widths, alignment details, number and type of structures, etc.
2) **Road condition data from non-destructive testing and measurement**. These data include road roughness, deflection, deformation, details of cracking, potholes, rutting, gullies, etc.

   Roughness is normally measured using a Bump Integrator and expressed through the International Roughness Index (IRI) (Appendix C). Typical values of the IRI for paved roads and its condition are as shown in Table 2.1.

Table 2-1 Road Condition and IRI for Paved Roads

IRI Ranges	Road Condition
Lower than 4	very good
4 to 7	good
7 to 10	fair
10 to 13	poor
Larger than 13	very poor

Road roughness is a very direct indicator of the condition of the road. All deterioration increases roughness in some way hence the roughness profile is a very good indicator of the comparative condition of all sections of the road and a very good indicator of the boundaries between sections that are relatively uniform.

Deflection surveys for the PMS are usually carried out using a Benkelman beam (Appendix D). When interpreting deflection data it is vital to check the wheel loads that were used. Normally deflection increases linearly with wheel load therefore measurements taken with one load can be easily converted to those that would have been obtained with a different load.

3) **Road condition data from destructive testing.** These data are obtained from coring the asphalt, digging test pits or trenches and the laboratory testing of the samples so obtained. They includes some or all of the information about the subgrade, pavement materials, and pavement structure as follows,

   a) Type of material, layer thicknesses, maximum size of aggregates of each layer, density and strength of each layer, etc., including details of any asphalt layer.
   b) For the subgrade: density, in situ moisture content, gradation, Atterberg limits, classification according to AASHTO and/or USCS systems, and in situ CBR obtained from correlation with DCP testing.

   The amount of such data that is available is likely to be quite small because it is not normally collected routinely as part of the PMS.

4) **Traffic and axle load data**. Such data have usually been collected separately and then forwarded to, or obtained by, the PMS Branch. The data should normally include AADTs, classification of the traffic among the various vehicle categories, and axle load data within each category.

   The traffic data available from the PMS Branch may already have been processed and may be available only in terms of broad traffic classes and cumulative equivalent axles. It may be necessary to collect the raw data in order to reprocess it in terms of other parameters.

   The usefulness of the existing traffic data will depend greatly on whether they are sufficiently up-to-date. Nevertheless, they should be a valuable complement to the evaluation of the traffic made for the specific project under consideration. In some cases they may be sufficiently recent, reliable, and complete to be used, at least in part, for the project.

5) **Archive data**. The final category of data normally available from the PMS Branch includes data such as: the submitted as-constructed drawings; ages of the pavement structures; unit prices of road works; vehicle operating costs; geo-climatic data and any other data about the road that may have been acquired since construction. General information about pavement structure and history should normally have been made available to the PMS through ERA's Contract or Maintenance Divisions and includes:

   a) Number and thickness of the pavement layers (surfacing, roadbase and sub-base), together with a description of the material of each layer (asphalt, concrete, surface treatment, crushed basalt aggregates, etc.).
   b) The date of construction or rehabilitation of the pavement structure.
   c) The type and date of major periodic maintenance activities.

   Unit rates of road works should also be available among the data in the PMS records. These costs are useful in the economic comparison of the feasible rehabilitation alternatives.

Vehicle operating costs (VOC) should not normally have an impact on the design of a specific project already selected for rehabilitation.

2.3 Other data

In addition documents available from the PMS Branch, a review of any other relevant existing documents should be conducted prior to field activities. Such a review is similar to the desk study undertaken for the feasibility study of any road project as outlined in the ERA's *Route Selection Manual and Site Investigations Manual.*

The geological environment of the project should be researched, together with the geotechnical characteristics corresponding to the geological formations along the alignment.

Sources of information regarding past investigations, published or not, should be obtained and reviewed. This may include activities only loosely related to the construction of the road (e.g. agricultural, hydrology, mining).

Existing reports, maps, mineral resource surveys, borehole logs, and other relevant data need to be collected and compiled. This is particularly valuable in identifying road construction materials.

Of particular importance are as-built plans of the road section under consideration or of adjacent projects, together with any memoirs or design reports (including pavement design reports and/or pavement evaluation reports) and maintenance records.

2.4 Traffic data

Although some traffic data is likely to be available from the PMS Branch, for major projects additional traffic data is likely to be required. The rehabilitation procedures require characterization of the traffic expected to be carried by the road after rehabilitation. Such characterization is done in terms of the cumulative number of equivalent standard axles. The process by which the cumulative number of ESAs is determined is as described in the ERA's *Pavement Design Manual Volume I*.

The design period of a rehabilitation project may differ from that which normally applies to new construction. For example, it may be an interim solution to make further use of an old road whilst a new alignment is being planned and designed for construction in the near future.

3. PAVEMENT EVALUATION PROCEDURE

The process of selecting appropriate methods of maintenance or rehabilitation and their design is shown in Figure 3.1 and is summarised below:

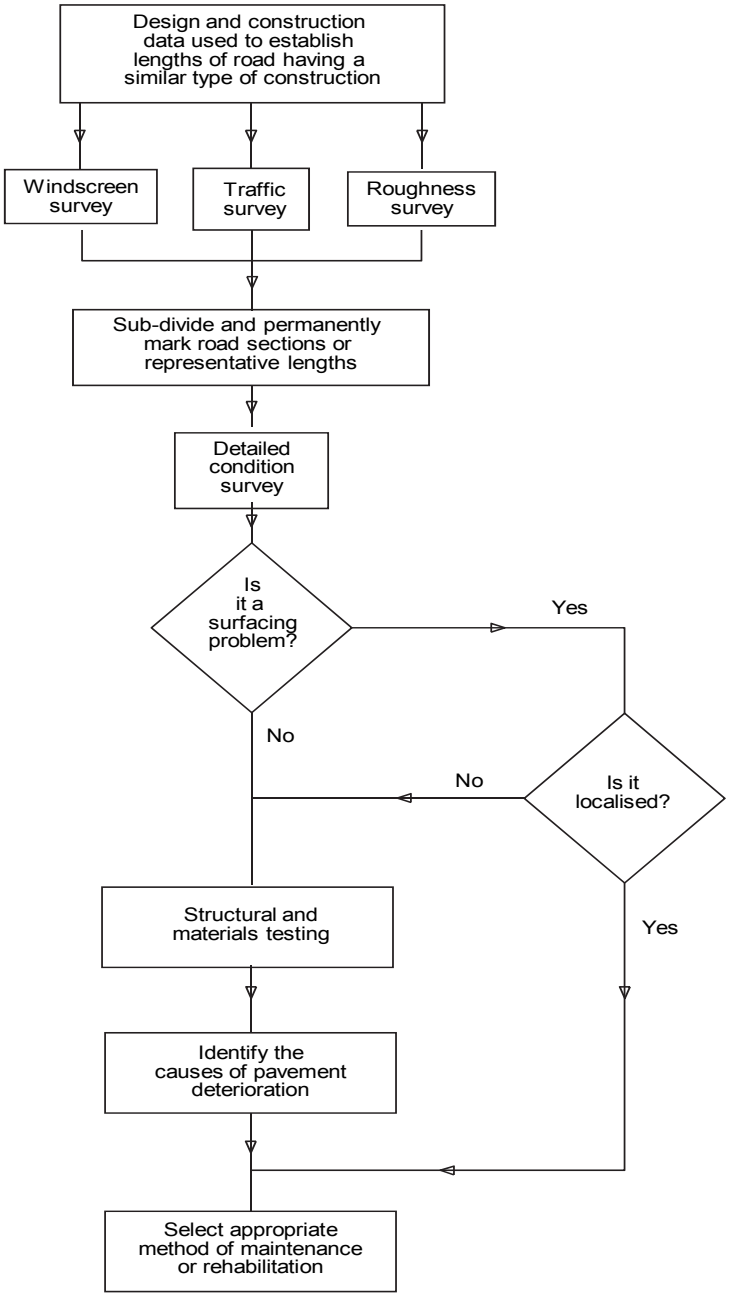


Figure 3-1 Road pavement evaluation and rehabilitation procedure

1) Collect and interpret existing data as described in Chapter 2.
2) Carry out initial (windscreen) surface condition survey, roughness and traffic surveys (Chapter 3) followed by a detailed surface condition survey.
3) Identify any simple surfacing problems that do not require structural investigation.
4) Carry out non-destructive structural and materials testing.

5) Carry out destructive testing (collect samples for laboratory evaluation).
6) Compile performance charts and divide the road into sections that are homogenous (similar throughout their length).
7) Establish the cause of the pavement deterioration for each uniform section.
8) Select and design appropriate maintenance and/or rehabilitation.
9) Carry out the maintenance and/or rehabilitation.

This procedure is comprehensive and applicable in all cases. However, the effort required at each stage may vary considerably. The causes of deterioration and the selection of appropriate remedial treatments may be very straightforward and require only confirmatory testing. The diagnosis may also be difficult and require much more detailed testing and evaluation. Very detailed testing is usually required if a pavement has deteriorated rapidly to ensure that all problems are identified. Detailed testing and evaluation is also required if there is a contractual issue to be resolved.

3.1 Interpretation of existing data

Design, construction and maintenance data, if available, can be used to establish the type and approximate thickness of the pavement construction. Using these data, those lengths of road having the same nominal thickness and type of construction are identified. Each length of road is then treated as a separate evaluation exercise.

The traffic loading in terms of equivalent 80 kN standard axles (esa) that the road pavement has carried since its construction should be calculated. Often, historical traffic counts are available but reliable axle load data may not be. In this case average esa values for different vehicle classes should be used (Table 3.1).

If neither classified traffic counts nor axle load data are available then surveys should be carried out as part of the evaluation exercise in order to establish current values. Techniques for carrying out such surveys are described in the ERA's *Pavement Design Manual Volume I* and in more detail in TRL's Road Note 40, *A Guide to the Measurement of Axle Loads using a Portable Weighbridge*.

It is important that, wherever possible, axle load data should be separated by direction of travel because any differences in axle loads can be useful in identifying the causes of pavement deterioration. Significant differences can occur on roads that lead to quarries or major ports where, for example, raw materials are being exported or imported.

Table 3-1 Average esa values for each vehicle class

Class	Type	No of axles	Average esa per vehicle - all loaded	Average esa per vehicle - half loaded(1)
1	Car	2	-	-
2	4-wheel drive	2	-	-
3	Minibus	2	0.3	0.15
4	Bus/coach	2	2.0	1.0
5	Small truck/PU	2	1.5	0.7
6	Medium truck	2	5	2.5
7	Large 2-axled truck	2	10	5
8	3-axled truck	3	12	3.5
9	4-axled truck	4	15	7.5
10	5-axled truck	5	17	8.5
11	6-axled truck	6	17	8.5
12	2-axled trailer	2	10	5
13	4-axled trailer	4	12	6

Note 1 It is common to find that vehicles have no back load hence half the vehicles are likely to be empty, or nearly so.

3.2 Three important principles

3.2.1 *Variability and road deterioration*

A road pavement is a very variable structure. Although the materials from which it is made are required to meet certain specifications, the range of values of their properties within those specifications is wide. Construction processes cannot be precise, and considerable variability also occurs during construction. For example, a road base will exhibit a range of densities and thicknesses, ideally all above the minimum specified requirements. The largest element of variability is usually the strength of the subgrade which varies along the road and from month to month. Good pavement design deals with these variabilities, and pavements are designed with levels of reliability that are high for main roads but a little lower for secondary and tertiary roads. Reliability is essentially the probability of a road reaching its design life in terms of traffic without reaching a defined failure condition. For main roads this may be 95%, 98% or even higher. In practice this means that, from a statistical point of view, a very few percent of the total length of the road is likely to reach the failure condition before the end of the design life. This small percentage should be repaired through maintenance.

The failure condition that defines the life of the pavement is itself a relatively small amount of failure in terms of road area. Consider, for example, deep shear failures in the road base. The deep ruts that result are unsightly and dangerous and increase road roughness considerably; but it requires only two or three such areas, each extending for, say, 3 metres in every 100m of road, for the road to be considered to be in very poor condition (well beyond a normal failure condition). Yet such a level of deterioration may amount to only about 2% of the road area and less than 10% of the road length. Thus

pavement evaluation is about identifying the behaviour of the worst few percent of the road. Average values of parameters are of little use: 5 or 10-percentiles are required.

3.2.2 *Correlate measurements*

Associated with the variability of the road described in the previous section is the necessity of making point specific measurements such as rut depths, deflections, DCP's (and test pits) at exactly the same location. In other words, at the testing point at each chainage, values of rut depth, deflection, DCP and test pits (although there will be relatively few of these) are all made within a few centimetres of each other. This is because the values are dependent on each other and the combined information at each point is many times more valuable than it would be if the data for each characteristic were from different locations with different properties. Rut depth is a good illustrative example. A rut is usually very variable in depth and the deepest part may only extend for a metre or so at very few chain ages along a section. However, that deep rut is symptomatic of the deterioration that may occur elsewhere along the road, but is showing itself initially at the weakest and most vulnerable section. The values of deflection and DCP at that point compared with the values at other points where the rut depth is low provide valuable clues as to the cause of the rutting and the probable behaviour of the road in the future.

3.2.3 *Where to test*

When a road is failing it is, perhaps, quite natural to want to concentrate investigations on the failed areas. However this is not always the best option, especially if structural failures are occurring. This is because such areas will display cracks, ruts, potholes and so on. Water will have already entered the structure through the failures and therefore the properties of the pavement materials will have changed considerably, hence it will be impossible to identify the primary cause of the problems. The areas that will prove to be the most informative will be those that are beginning to show signs of failure because they are likely to display only one form at the early stage. This will be the primary or main reason for the failure.

3.3 Initial surface condition survey and roughness survey

3.3.1 *Windscreen Survey*

After dividing the road into lengths of nominally similar construction, it may be necessary to subdivide it again based upon the current condition of the road. This can be done by carrying out a windscreen survey. The best way to do such a survey is for the survey vehicle to *stop* at 500 metre or one kilometre intervals to enable the condition of the road pavement to be recorded accurately using a selection of the road pavement deterioration criteria. Important aspects of road deterioration may be missed if the vehicle is not stopped and survey staff given the opportunity to inspect the road closely. Surface condition data may be available from a PMS survey but it may not be up-to-date.

The roughness of the road should also be measured at this stage in the evaluation (Appendix C) unless recent roughness survey data are available from a PMS survey. These measurements are necessary for the economic appraisal and are useful in defining sections of road in similar condition. The road can then be subdivided into shorter uniform sections based upon the following;

- time since construction
- traffic loading

- type of road deterioration and
- topography

3.3.2 *Roughness measurements*

It is well established that vehicle operating costs increase as the roughness of the road pavement increases. Most road defects contribute in some way to increasing the roughness of the road pavement, although in its early stages cracking may cause little or no change. However, without proper maintenance, the cracked surfacing deteriorates and the resulting potholes and subsequent patching cause a rapid increase in roughness. Surface texture and variability in rut depth also have a significant effect on the roughness of a road pavement.

The standard measure of road roughness is the International Roughness Index (IRI) which was developed during *'The International Road Roughness Experiment'* in Brazil (Sayers et al, 1986). It is a mathematical quarter-car simulation of the motion of a vehicle at a speed of 80 km/h over the measured profile and can be calculated directly from road levels measured at frequent intervals. Devices for measuring levels are usually either slow and labour intensive, or fast, automatic and expensive. Hence, the roughness of the road is usually measured using a Response Type Road Roughness Measuring System (RTRRMS) which must be periodically calibrated to allow the values of roughness to be reported in terms of IRI. Suitable methods of calibration include a rod and level survey (ASTM, E 1364-95) or the MERLIN (Machine for Evaluating Roughness using Low-cost INstrumentation) (Cundill, 1996). Both the roughness survey and calibration procedures are described in Appendix C.

The roughness of roads with similar pavement construction is a good measure of their relative pavement condition, but it does not identify the nature of the failures or their causes. However, if resources for the surface condition survey are limited or if the sections of the road under investigation are very long, roughness and windscreen survey data can be used to establish those lengths of road having failures of differing severity. This allows representative lengths of road to be selected which can then be used to identify the cause or causes of deterioration.

3.4 Detailed surface condition survey

The next step is to carry out a detailed surface condition survey. When the uniform sections are relatively short, the detailed condition survey is best carried out over the entire length of the section. However, where resources are limited, a number of representative one kilometre lengths of road can be used to identify the cause of pavement distress. The length of road investigated by this method should represent no less than 10 per cent of each section.

Before the detailed surface condition is carried out, the section or representative one kilometre length should be permanently marked into 'blocks' of equal length. For inter-urban roads the maximum block length should be either 50 or 100 metres, however, the length may be reduced to as short as 10 metres if the road is severely distressed.

During the detailed surface condition survey the nature, extent, severity and position of the following defects are recorded.

1) Cracking (Section 3.5.1 to 3.5.5).
2) Pot holes and patching (Section 3.6).

3) Edge failures and shoulders (Section 3.7).
4) Rut depth (Section 3.8).
5) Deformation (excluding rutting) (Section 3.8).
6) Surfacing defects; e.g. bleeding, fretting, stripping (Section 3.9).
7) Surface texture and aggregate polishing (Section 3.9).

Rutting is measured at specific chainages. It is recorded once at the beginning of each of the blocks. It is important that rutting is measured at a known point because its severity may need to be compared with other non-destructive tests carried out at the *same* location.

The recommended form for recording the surface condition data is shown in Figure 3.3. The form is designed to be as flexible as possible since the nature of paved road deterioration varies depending on factors such as the type of construction, climate and traffic levels. There are, however, a number of defects that tend to be common to all road pavements and these are described in Table 3.2. The extent of the defect is recorded as shown in Table 3.3

There are three blank rows on the surface condition form for each chainage. These should be used if other defects not included in the form need to be recorded.
The resources and the equipment required for the detailed condition survey and the operational details are described in Appendix E.

Table 3-2 Data and common surface defects to be recorded

Road number	The Nationally accepted route number
Form number	Numbers to run consecutively
Date	Day/month/year
Inspector	Name of inspector
Start location	If an established marker is available it should be used. If not, permanent markers such as junctions should be used.
Direction	The direction towards a permanent feature, preferably a large town.
Road width	Road width should be recorded at the beginning of each form
Surfacing	Type (asphalt/bituminous seal)
Shoulder	Type (gravel/sealed) and width
Chainage	Chainage 0+000 is at the start point. If 50m blocks are used then following chainages will be 0+050, 0+100 etc.
Crack type	Letters L, T, B, C or P (Section 3.5.1)
Crack intensity	Nos. 0-5 (Section 3.5.2)
Crack position	Letters V, O or CW (Section 3.5.3)
Crack width	Nos. 1-4 (Section 3.5.4)
Crack extent	Nos. 1-3 (Section 3.5.5) Extent as in Table 3.3
Pot holes and patching	As defined in Section 3.6. Extent as Figure 3.2
Edge failures	F or S (Section 3.7). Extent as Table 3.3
Rut depth	Section 3.8. Maximum value recorded in either the verge side or offside wheel path. If shoving is occurring the value should be circled.
Depressions	As defined in Section 3.8. Extent as Table 3.3

Table 3-3 Extent of defects

Extent	Length of block affected (%)
1	<10
2	10-50
3	>50

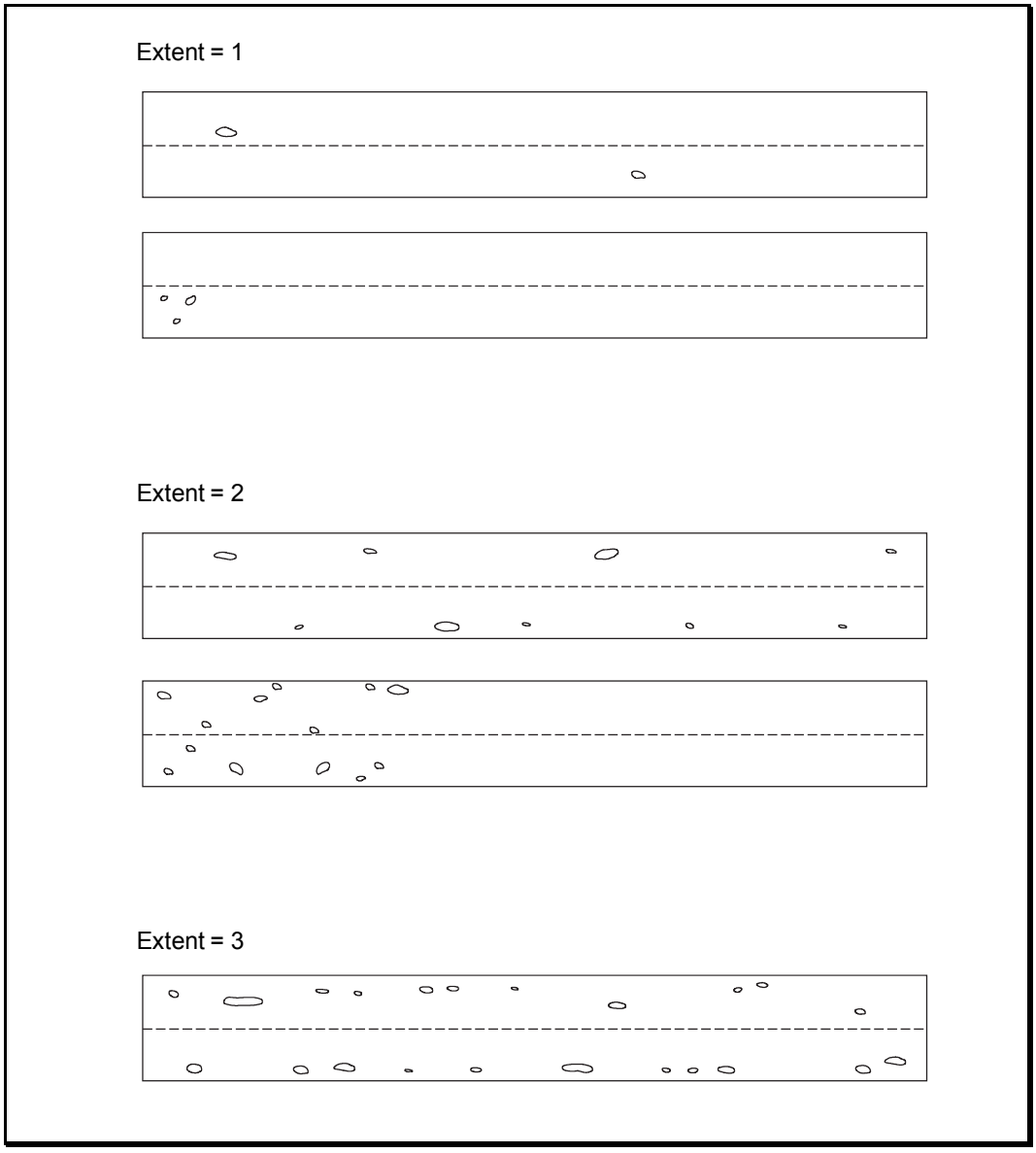


Figure 3-2 Extent of potholes and patching

Paved Road Condition Survey				**Date**		**Inspector**		**District**	
Road No		Start Km		Direction	From		To		
Block No				Left		Right		Comments/Actions	
				Severity	Extent	Severity	Extent		
	Side drain		Silt						
			Erosion						
	Shoulder		Deform						
			Erosion						
			Vegetation						
	Edge step								
	Edge damage								
	Rut depth								
	Cracking	Type							
		Intensity							
		Position							
		Extent							
		Width							
	Potholes/patching								
	Bleeding								
	Fretting								
	Corrugations								
	Surface texture								
	Aggregate polishing								
Block No				Left		Right		Comments/Actions	
				Severity	Extent	Severity	Extent		
	Side drain		Silt						
			Erosion						
	Shoulder		Deform						
			Erosion						
			Vegetation						
	Edge step								
	Edge damage								
	Rut depth								
	Cracking	Type							
		Intensity							
		Position							
		Extent							
		Width							
	Potholes/patching								
	Bleeding								
	Fretting								
	Corrugations								
	Surface texture								
	Aggregate polishing								

Figure 3-3 Field Survey Form for Pavement Evaluation

Table 3-4 Other surfacing defects

Item	Reference
Bleeding/fatting up	Section 3.9.1. Extent as Table 3.3
Fretting/stripping	Section 3.9.2. Extent as Table 3.3
Loss of stone	Section 3.9.3. Extent as Table 3.3
Corrugations	Section 3.9.4. Extent as Table 3.3
Surface texture	Section 3.9.5 and Table 3.5
Polishing	Section 3.9.6 and Table 3.6

3.5 Cracking

The assessment of cracking should fulfil two objectives. Firstly, it should identify whether the road pavement is suffering from load or non-load associated distress. Secondly, it should establish whether the severity of cracking will affect the performance of any subsequent new pavement layer by causing reflection cracking. These objectives are best achieved by identifying *five* characteristics of the cracking.

(i) Type
(ii) Intensity
(iii) Position
(iv) Width
(v) Extent.

3.5.1 *Type*

The appearance of a crack can provide a guide to its likely cause. It is recommended that five types of crack are defined. These are listed below and illustrated in Figure 3.4.

L longitudinal cracks
T transverse cracks
B block cracks
C crocodile cracks
P parabolic cracks

3.5.2 *Intensity*

The intensity of cracking is defined by six levels described below. If the intensity of cracking varies within any block, the predominant form should be recorded.

0 no cracks
1 single crack
2 more than one crack - not connected
3 more than one crack - interconnected

4 crocodile cracking

5 severe crocodile cracking with blocks rocking under traffic

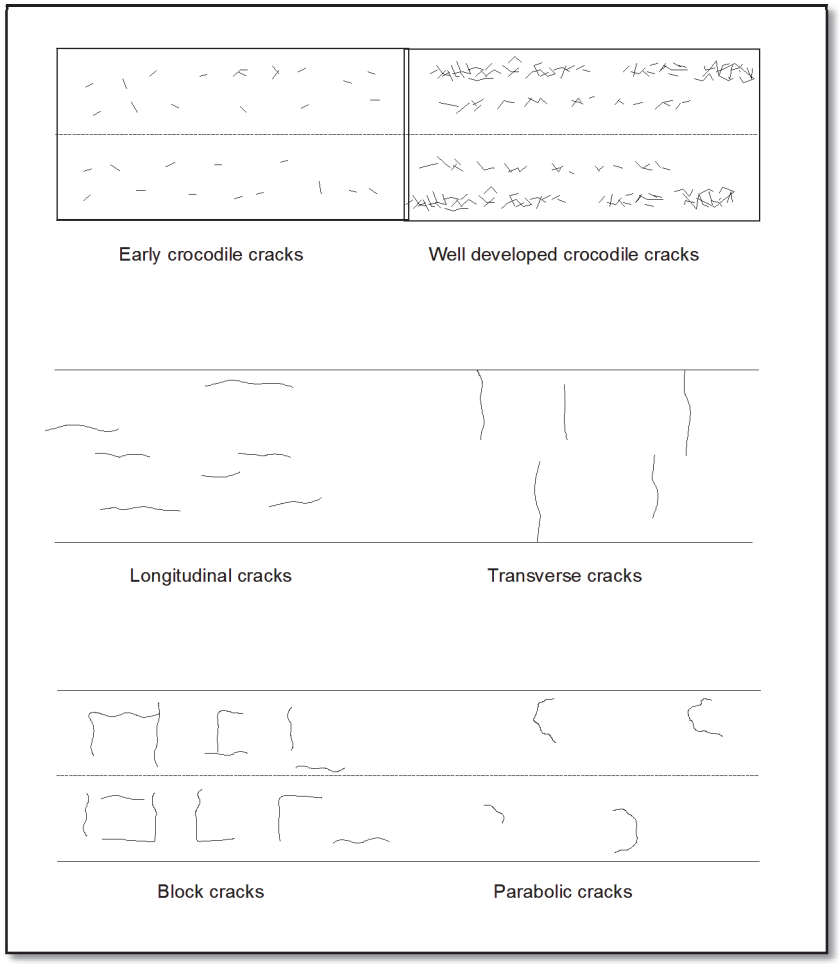


Figure 3-4 Types of cracking

3.5.3 Position

The position of the cracking is recorded. The cracking can be confined to either or both of the verge side (V) and offside (O) wheel paths, or can be spread over the entire carriageway (C/W).

3.5.4 *Width*

The measurement of crack width is difficult, but it is important because the width partly determines whether a crack can be sealed effectively. Four categories are recommended as shown below. The first three are for cracks which are not spalled; cracks with substantial spalling are classified as width 4. The widths of the cracks usually vary within any block, and so it is the width of crack that predominates that is recorded. Initially, until the technicians are familiar with the system, the widths of the cracks can be measured with a simple Go/No-go crack width gauge shown in Figure 3.5.

1 crack width < 1mm

2 1mm < crack width < 3mm

3 crack width > 3mm
4 cracks with spalling

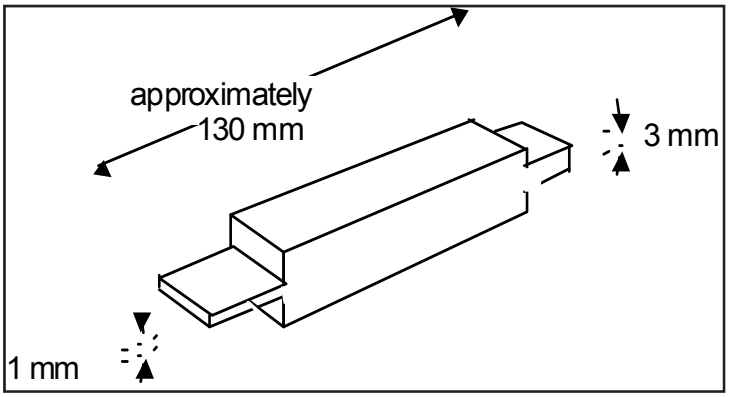


Figure 3-5 Crack width gauge

3.5.5 *Extent*

The extent of the cracking is defined as the length of block affected as shown in Table 3.3. The extent of cracking should be recorded irrespective of intensity.

3.6 Potholes and patching

Potholes are structural failures which include both the surfacing and roadbase layer. They are usually caused by water penetrating a cracked surfacing and weakening the roadbase. Further trafficking causes the surfacing to break up and a pothole develops. Because of the obvious hazard to the road user, potholes are usually patched as a matter of priority. Although patches are not necessarily defects, they do indicate the previous condition of the road and are included in the assessment. The extent of potholes and patching is recorded as shown in Figure 3.2.

3.7 Edge failures and shoulder condition

Edge failures are caused by poor shoulder maintenance that leaves the surface of the road pavement higher than the adjacent shoulder. This unsupported edge can then be broken away by traffic, narrowing the running surface of the road. Edge failures (F) are recorded when they exceed 150mm in width at their maximum point or when the vertical step from the surfacing to the shoulder is greater than 50mm (S). It is convenient to measure the defects with the scale on the side of the calibrated wedge. The length of the road affected is recorded according to Table 3.3.

The condition of the shoulders is also recorded. Whilst not necessarily contributing to structural failure at this stage, the repair of shoulder defects will be an important part of the rehabilitation and therefore the extent of the required repairs needs to be measured.

3.8 Surface deformation

In terms of its assessment, pavement deformation divides into two groups. Firstly, those defects with short wavelengths where severity can be measured by the use of a simple 2-

metre straight-edge and calibrated wedge (Figure 3.7). Secondly, those defects with longer wavelengths that are best quantified by the use of more sophisticated road profiling instruments. This is discussed in Appendix C on roughness measurements.

3.8.1 *Rutting*

Load associated deformation or rutting appears as longitudinal depressions in the wheel paths. It is the result of an accumulation of non-recoverable vertical strains in the pavement layers and in the subgrade caused by traffic loads. In its early stages this type of rutting is not associated with shear failure (or shoving) in the upper layers of the pavement until it becomes very severe.

Sometimes distinct wheel paths do not exist, for example, because of a large volume of non-motorised traffic causing motorised traffic to wander. In these circumstances the pattern of road deterioration will be different and some of the important clues relating to the position of the deterioration on the carriageway will be absent.

Rutting can also be the result of shear failure in either the unbound or the bituminous pavement layers resulting in shoving at the edge of the road pavement. Where the shear failure is occurring in the unbound roadbase or sub-base the displaced material will appear at the edge of the surfacing. Where the failure is occurring in the bituminous material, the displaced material will be evident in the surfacing itself. This is illustrated in Figure 3.6. The severity of the shoving is difficult to measure without taking levels. However its occurrence, together with the depth of rutting, should be recorded thereby clearly identifying the cause of the failure. This can be simply done by putting a circle around the value of rutting recorded on the surface condition form.

The width of the running surface and the traffic flow govern the number of observable wheel paths on paved roads. For example, a 3-metre carriageway will have two wheel paths but at road widths greater than 6.5 metres there are generally four. At intermediate widths and low traffic flows there may be three wheel paths, with the central one being shared by traffic in both directions. Rut depths should be recorded in the wheel path showing the most rutting. On most roads this is usually the verge side wheel path because here the road pavement is generally weaker as a result of higher moisture content and less lateral support. The straight-edge is placed across the wheel path at right angles to the direction of traffic and the maximum rut depth recorded. If the ruts are greater than 40mm deep, the wedge can be held vertically and the depth recorded to the nearest 10mm.

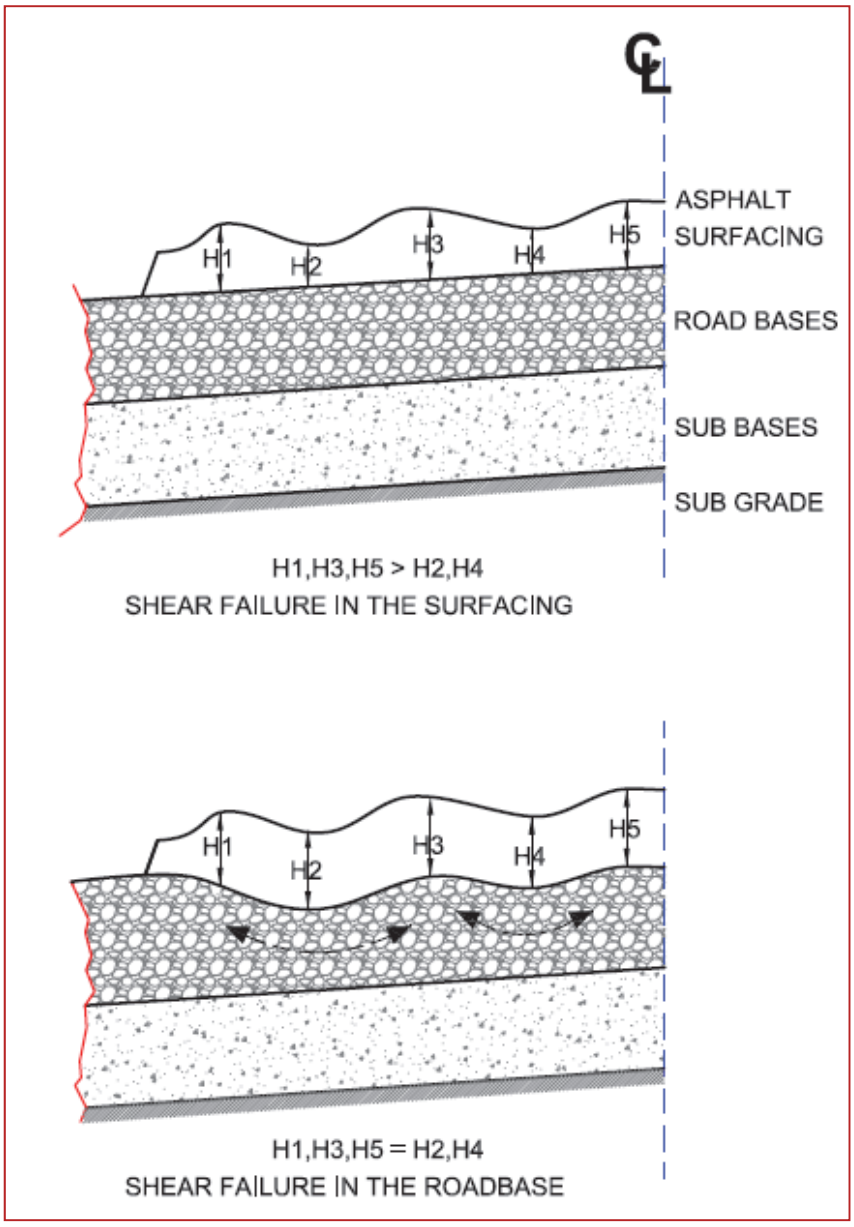


Figure 3-6 Transverse core profile to investigate rutting

3.8.2 *Depressions*

Localised depressions caused by settlement of the pavement layers, construction faults and differential movement at structures, particularly culverts, should be recorded. These are easy to see after periods of rain because they take longer to dry than the rest of the road. When the road is dry, they can sometimes be identified by the oil stains that occur where vehicles cross the depression. The depth should be measured using the 2-metre straight-edge and calibrated wedge.

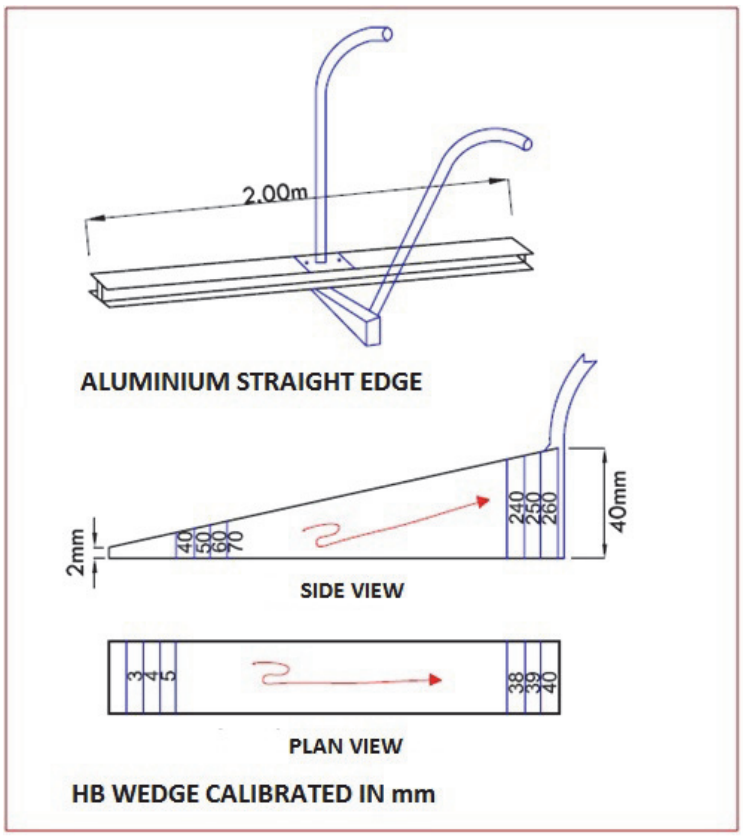


Figure 3-7 Straight edge and calibrated wedge

3.9 Surfacing defects

There are a number of surfacing defects that are not symptoms of structural failures and therefore do not require structural surveys although they indicate problems with the surfacing materials.

3.9.1 *Bleeding and fatting-up*

Bleeding is usually observed first in the wheel paths and is the result of bitumen being forced to the road surface by the action of traffic. Fatting-up of the surface is a less extreme form of bleeding where the surface becomes very smooth but there is insufficient binder to form a continuous film on the surface. The following definitions are recommended:

Bleeding: A continuous film of binder covering the aggregate.

Fatting-up: Smooth and shiny appearance but aggregate visible.

Bleeding and fatting-up can often be discontinuous. In asphalt surfacings this can be the result of variations in the mixing process, local over application of tack coat or secondary compaction by traffic. In surface dressings it can be caused by variability in the prepared surface or poor quality control during the spray and chip operation.

3.9.2 *Fretting and stripping*

Fretting is the progressive loss of fine aggregate from the road surface and occurs when the small movements of individual particles, under the action of traffic, exceeds the breaking strain of the bitumen. It tends to occur later in the life of the surfacing after the bitumen itself has deteriorated with age and usually begins in areas of high traffic stress such as

sharp bends. The loss of fine aggregate at the surface results in lack of mechanical interlock which can eventually lead to the loss of coarse aggregate and the formation of potholes.

Stripping in asphalt surfacings is the result of the displacement of binder from the surface of the aggregate caused by the combined action of water and traffic. In most cases there is a migration of the binder towards the surface of the road resulting in localised bleeding at the surface and unstable poorly coated aggregate beneath. These areas then disintegrate under traffic and develop into shallow potholes. The introduction of denser asphalt mixes and the use of cement and hydrated lime as filler have largely reduced the occurrence of stripping in asphalt surfacings.

Although the mechanisms of failure differ, the result of both of these types of deterioration will be a shallow pothole or a series of potholes. The extent of the defect is recorded as shown in Figure 3.3.

3.9.3 *Loss of stone chippings from a surface dressing*

The loss of chippings from a surface dressing resulting from poor adhesion between the binder and the aggregate appears early in the life of the surfacing. It starts in the wheel paths but, with time, the problem may spread across the carriageway making it difficult to differentiate between this type of failure from bleeding. However, it can often be identified by an accumulation of chippings at the edge of the road pavement. The extent of the defect is recorded according to Table 3.3. The following definition is recommended:

Loss of stone: Continuous film of bitumen visible due to the loss of aggregate.

3.9.4 *Corrugations*

Corrugations consist typically of a series of ridges perpendicular to the centre line of the road and usually extend across the whole width of the carriageway. Their spacing, or wavelength, is usually in the range of 0.5-1.0 metres but can, in some circumstances, be as much as 10 metres. In paved roads they are caused by instability in either the asphalt surfacing or the unbound roadbase under a thin seal. There is generally no need to measure the severity of the corrugations because it will not affect the selection of the remedial treatment. The extent of the defect is recorded as shown in Table 3.3.

3.9.5 *Skid resistance and aggregate polishing*

The ability of a bituminous surfacing to provide the required skid resistance is governed by its macrotexture and microtexture. The macrotexture of the surfacing, as measured by its texture depth, contributes particularly to wet skidding resistance at high speeds by providing drainage routes for water between the tyre and the road surface. The surface condition survey should include a qualitative assessment of texture in the wheel paths so that it can be used to trigger quantitative testing if required (Appendix E). As a guide, the categories shown in Table 3.5 are suggested.

Table 3-5 Visual assessment of surface texture

Texture	Description
Fine (F)	The surfacing is smooth and the coarse aggregate (if present) in the surfacing is not visible, e.g. a sand seal, fine slurry seal or smooth asphalt.
Medium (M)	The road has a smooth appearance and will generally comprise fine aggregate. If present, any coarse aggregate is visible but the surface does not appear coarse because of fine aggregate between the coarse particles, e.g. a new 6mm single surface dressing or 13/6mm double surface dressing.
Coarse (C)	The surfacing has a coarse appearance, with coarse aggregate clearly visible, e.g. a new 13mm surface dressing.

3.9.6 *Aggregate polishing*

The micro-texture of the surfacing, as measured by the resistance to polishing of the aggregate, is the dominant factor in wet skidding resistance at lower speeds. The assessment of polishing is more difficult than that of the surface texture but will be unnecessary if surfacing aggregates having a satisfactory minimum Polished Stone Value were used during construction. When marginal quality aggregates have been used or if increased traffic flows have resulted in an increased state of polish, skid resistance will be reduced (Appendix E). The qualitative assessment will depend on the judgement of the technician, and Table 3.6 is suggested as a preliminary guide.

Table 3-6 Visual assessment of aggregate polishing

Condition	SRV[1]	Description
Harsh (H)	>75	Stones very harsh[2], edges sharp to touch
	55 - 75	
Angular (A)	45 - 55	Stones sharp and angular but not harsh
	35 - 45	
Smooth (S)	< 35	Stones rounded and smooth to the touch

Notes 1. Skid resistance value (SRV) measured by the portable skid-resistance tester
2. Harsh stones have surfaces that are rough to the touch.

3.10 Deterioration caused by poor drainage

Localised pavement failures are often caused by the poor design or maintenance of side and cut-off drains and cross drainage structures. When side drains and culverts silt up, water ponds against the road embankment, eventually weaken the lower pavement layers. Conversely, if the water velocity in the side drain is too high it erodes the road embankment and shoulders. More general failures occur when there is no drainage within the pavement layers themselves. Paved roads do not remain waterproof throughout their lives and, if water is not able to drain quickly, it weakens the lower pavement layers and results in rapid road failure. However, pavement deterioration as a result of poor drainage may not be obvious in the dry season hence discussions with local people may be necessary to establish the situation in the wet season.

Erosion and siltation are the principal forms of deterioration of the side drains and mitre drains and the extent and severity should be recorded as shown in Table 3.3. The effectiveness of any scour checks must also be recorded.

Culverts may be blocked, partially blocked or clear. If blocked they will almost invariably be contributing to deterioration of the pavement. They may also be causing erosion because of inadequate or damaged inlets or outlets, and they themselves may be damaged. All such data should be recorded because it will be needed to estimate repair costs.

4. TREATMENT OF LOCALISED SURFACING DEFECTS

There are some surfacing defects which, if localised, can be treated at this stage without the need for further testing. Suggested treatments for these types of pavement distress are summarised in Tables 4.1 and 4.2.

Table 4-1 Surfacing defects - roads with thin bituminous seals

Defect	Extent	Maintenance treatment	Notes
Fretting	<10%	Local patching	A fog spray may be sufficient to rejuvenate the surface and prevent further fretting.
	>10%	Surface dressing or slurry seal	
Loss of stone, bleeding and fatting-up	<10%	No action	Local application of heated aggregate may be required if poor skid resistance is a problem.
	>10%	Additional tests required	A new surfacing may be required
Loss of texture and/or polishing of aggregate	<10%	No action	
	>10%	Additional tests required	A new surfacing may be required
Potholes	Any	Patch	Potholes are the result of other failures such as cracking and deformation and additional tests will usually be necessary
Edge failures	Any	Patch the road and reconstruct the shoulder	

Table 4-2 Surfacing defects - roads with asphalt surfacings

Defect	Extent	Maintenance treatment	Notes
Fretting or stripping	<10%	Local patching	Application of a proprietary rejuvenator may prevent further fretting.
	>10%	Patching followed by surface dressing or slurry seal	
Bleeding or fatting-up	<10%	No action	Local application of heated fine aggregate may be required if poor skid resistance is a problem.
	>10%	Additional tests required	A new surfacing may be required
Loss of texture and/or polishing of aggregate	<10%	No Action	
	>10%	Additional tests required	A new surfacing may be required
Potholes	Any	Patching	Potholes are the result of other failures such as cracking and deformation and additional tests will usually be necessary
Edge failures	Any	Patch road and reconstruct the shoulder	

5. PERFORMANCE CHARTS

The Performance Chart is a graphical representation of an aspect of pavement condition plotted against chainage. It is an invaluable aid to identifying both the cause or causes of deterioration and the scale of the problems. For rehabilitation design it is usually necessary to divide the pavement into homogenous sections, each of which is likely to require the same treatment throughout. These sections are usually of similar construction, strength, condition, roughness, and so on. It is often helpful during the diagnostic process to use a variety of different data sets and to compare the results. This can provide valuable clues about the modes of deterioration and also on the rehabilitation required.

For example, the cracking, rutting and roughness recorded during both the windscreen and the detailed condition survey should always be displayed in the form of performance charts. These enable the length of road affected by each form of deterioration to be quantified and, most importantly, identify which characteristics are associated with each other.

However, care is required in the interpretation of these data. Apart from the surface defects described in Tables 3.2 and 3.4, bituminous surfaced roads will generally deteriorate either by rutting or by cracking. It is important that the initial form of deterioration and its cause is identified because this determines the type of maintenance and repair that is most appropriate. After further trafficking, the initial cause of deterioration can be masked by subsequent deterioration. An illustration of this is shown in Figure 5.1 where the final appearance of the road deterioration is similar despite having different initial causes. It is also important to establish if the failures are localised, perhaps because of poor drainage, or whether they are affecting the road in a more general manner.

When an evaluation takes place there will often be considerable lengths of road that have reached a terminal level of deterioration similar to that shown in Figure 5.1. However, even within nominally uniform sections, road pavements are inherently variable, having a range of pavement thicknesses and material properties. This results in differential performance, with some areas deteriorating less rapidly than others. It is in these areas that the initial form of deterioration can be most easily identified.

Particularly important are performance charts showing the structural characteristics of the road namely the deflection values and DCP strength values.

An example of the use of performance charts is illustrated in Figure 5.2 for a 20km section of paved road having a mechanically stabilised gravel roadbase with a thin bituminous surfacing. The initial form of deterioration was rutting which was associated with shoving whenever the failure became severe. Although there is some cracking which is coincident with high values of rutting, there is no cracking in areas of less severe rutting, suggesting that the rutting preceded the cracking. In addition to the rutting, substantial lengths of the surfacing are suffering from bleeding. However, the charts show that there is *no correlation* between the bleeding and the rutting, indicating that the shoving is in a lower granular layer, not the bituminous surfacing.

Using performance charts similar to this, the road section under investigation is divided into subsections having failures of differing type and/or severity. A programme of

additional tests (see Chapter 6) is then prepared to identify the causes of the differential performance between the sub-sections.

There may be some cases where the complete section of road will have reached a failed condition, for example, when the road pavement has been under designed or where there are serious material problems. In such cases the cause of the deterioration can only be established by comparing the thickness of the road pavement or the material properties of the pavement layers with relevant design standards and material specifications.

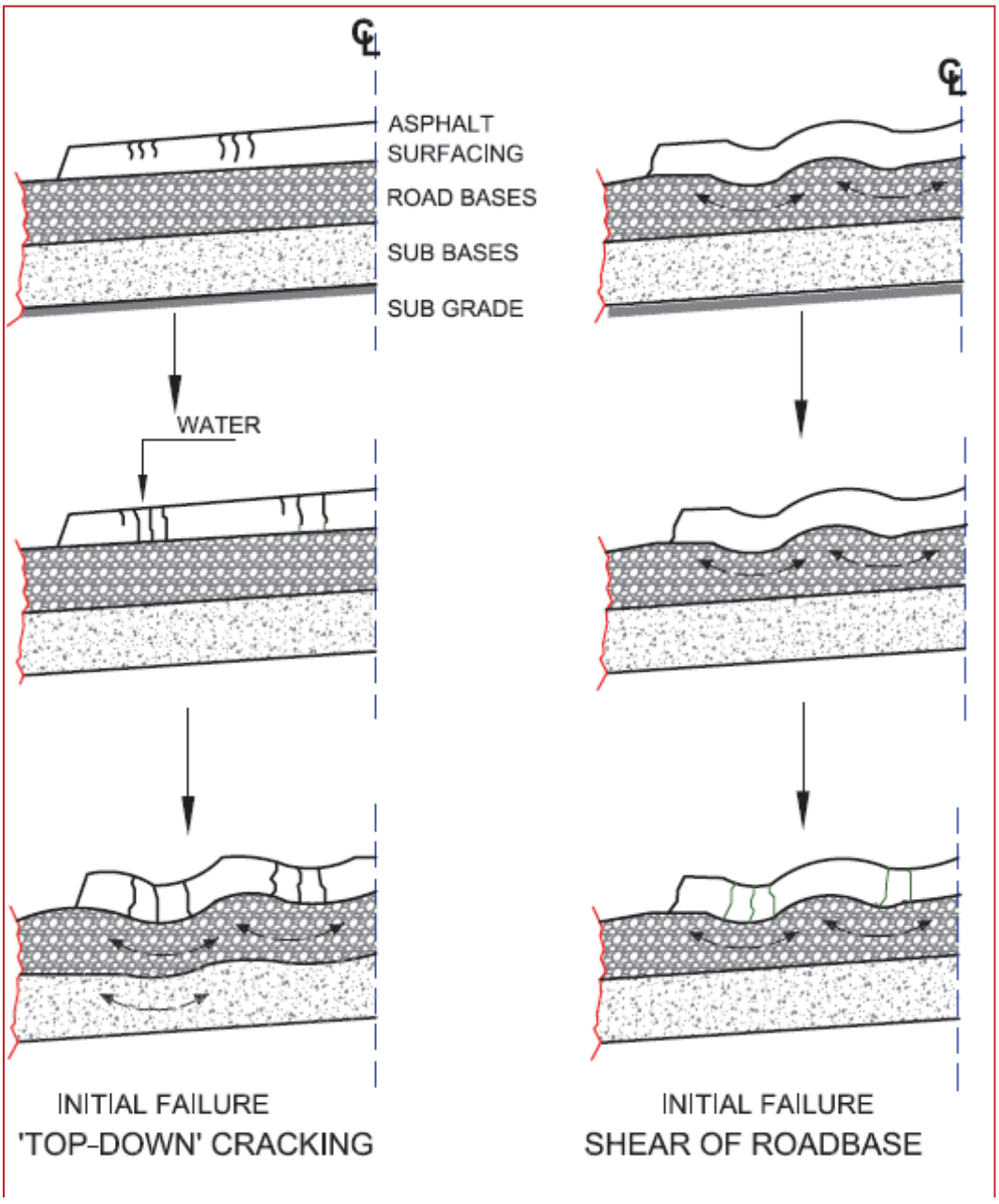


Figure 5-1 Development of road failure

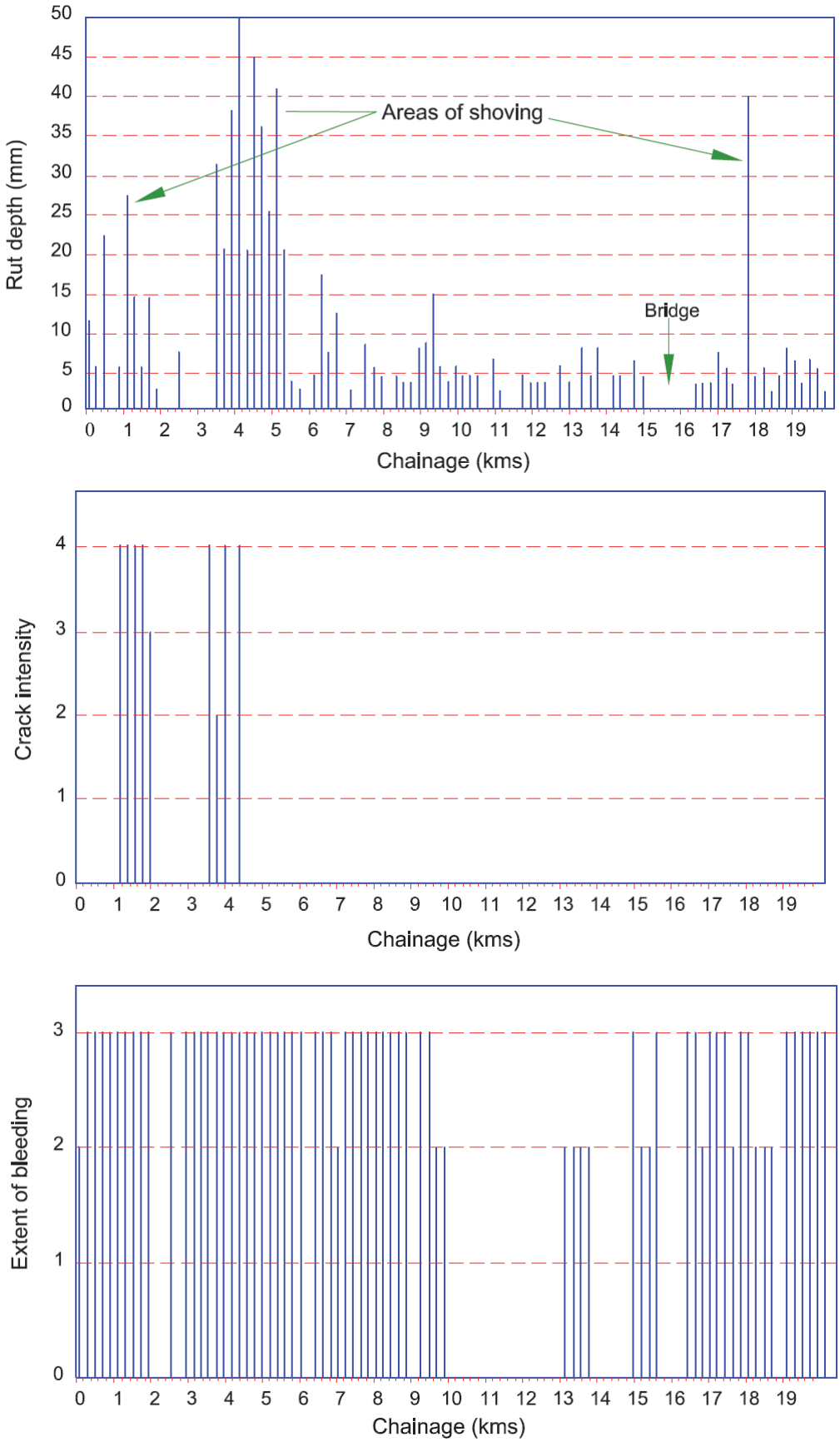


Figure 5-2 Example of performance charts

6. STRUCTURAL MEASUREMENTS

Deflection measurements and Dynamic Cone Penetrometer (DCP) tests are used to obtain important information about the structural properties of the road pavement, to identify the cause of differential performance between sub-sections and to provide information for the maintenance or rehabilitation of the section.

In many cases the structural properties vary throughout the year as a result of changes in the moisture content of the road pavement, especially the subgrade. In these circumstances the tests should be carried out towards the end of the rainy season, when the road is at its weakest. The results from these non-destructive tests are usually confirmed by a limited number of destructive sampling and material testing.

6.1 Deflection tests

The strength of a road pavement is inversely related to its maximum vertical deflection under a known dynamic load. Table 6.1 lists the more common deflection devices, their loading regimes and output.

Table 6-1 Deflection measuring methods

Device	Type of applied load	Output	Comments
Deflection beam	Moving truck wheel	Maximum deflection only	Slow and relatively inaccurate but inexpensive. Still in common use
Deflectograph	Moving truck wheel	Deflection bowl (partial)	Essentially an automated deflection beam. Travels at 5km/hr but takes a measurement every 2 metres.
Road Rater	Vibratory	Deflection bowl	Does not simulate truck loads and now not in common use
Dynaflect	Oscillatory	Deflection bowl	Does not simulate truck loads and now not in common use
Falling Weight Deflectometer	Impact	Deflection bowl	The instrument of choice. Accurate and reliable but has to be stopped for 2-3 minutes to take a reading so is sometimes inconvenient on main roads.

The least expensive of these instruments is the deflection beam. This is a mechanical device that measures the maximum deflection of a road pavement under the dual rear wheels of a slowly moving loaded lorry. The recommended test and survey procedures for the deflection beam are given in Appendix D. In the past, a rear axle load of 63.2 kN was usually used and hence is still in common use because it is easier to load the truck and because engineers are familiar with the deflection values that it produces. However, in recent years heavier axle loads have also been used, most commonly 80 or 100 kN. Over this range of loads the maximum deflection is usually linearly related to the applied load. Therefore, for structurally adequate pavements where over-stressing is not a danger, deflection values can be measured with these higher loads and then normalised to any standard load for comparison purposes.

The maximum deflection under a moving wheel load is an indicator of the elastic properties of the pavement and therefore a good indicator of its overall load spreading ability. Although it is not a direct measure of strength it has been shown to correlate well with long term performance of pavements under traffic. For example, if a road is under-designed for the traffic it is carrying for any reason (e.g. incorrect assessment of subgrade strength or traffic loading) the stresses in the lower layers of the pavement will be too high and the pavement will deteriorate through the development of ruts. Under such circumstances the deflection will correlate with rut depth, as shown schematically in Figure 6.1, and such a correlation provides an indication of the reasons for failure (see Chapter 8).

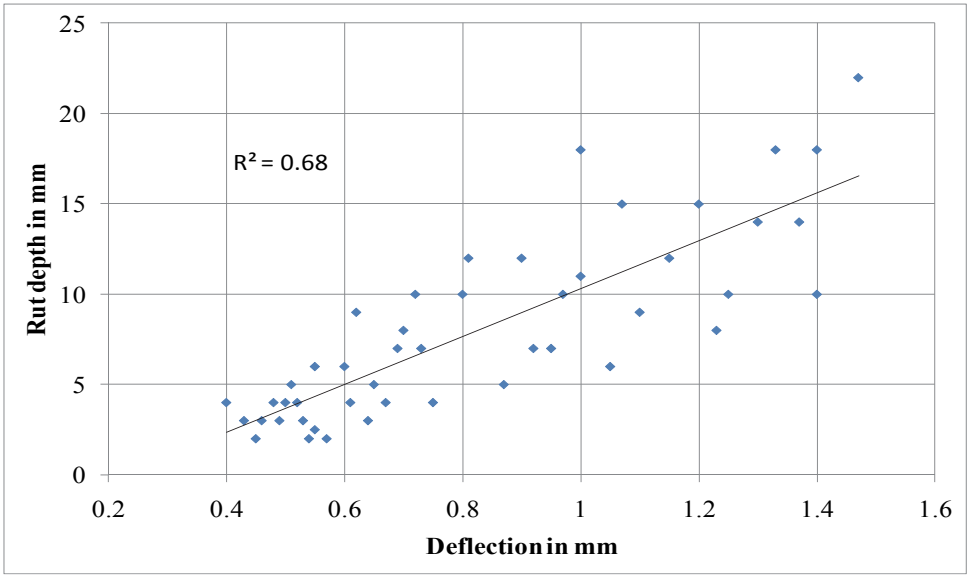


Figure 6-1 Example of a good correlation between deflection and rut depth

There are advantages in using deflection equipment capable of measuring deflection bowl parameters other than just the maximum deflection. The Falling Weight Deflectometer (FWD) and the Deflectograph are the most widely used for this purpose, the FWD being by far the most popular. It has the advantage of being able to apply impact loads which more accurately simulate the effect on pavements of heavy vehicles moving at normal traffic speeds than the slowly moving load applications associated with the Deflectograph or the deflection beam. Procedures for using FWD equipment for road surveys are given in Appendix D.

Full analysis of deflection bowl data is dependent on a suitable model to calculate the response of the pavement to the applied load. Most analysis programs are based on the assumption that the pavement behaves, in the first instance, as a multi-layer structure made up of linearly elastic layers. Using such a model, it is possible to calculate the effective elastic modulus of each pavement layer from knowledge of the shape of the deflection bowl. This 'back-analysis' procedure requires accurate deflection data extending from the central maximum deflection out to deflection values at radial offsets of as much as 2.1 metres.

However, the linear elastic model is a very simple model of road pavements. Road materials display properties that do not comply with the assumptions of the model. For

example, the elastic modulus of unbound materials is not constant but depends on the stresses to which the material is subjected at each point in the structure. The materials are not linear. This is a particularly important consideration for the subgrade because the modulus of the subgrade has a very strong influence on the shape of the entire deflection bowl. Errors or inaccuracies in the assumptions here give rise to errors in the calculations of the moduli of all other layers. A further consideration is the capability of the computer programs to handle complex structures. The more layers that are present, the more difficult it is for the programs to identify a suitable unique solution.

The acceptability of the results of the pavement analysis depends more on the skill of the analyst than the sophistication of the analysis program. As part of the Strategic Highway Research Program in the USA, guidelines for estimating pavement layer elastic moduli by back calculation from deflection bowl data were developed (FHWA-RD-01-113 (2002). *Back-calculation of layer parameters for LTPP test sections. Volume II: Layered elastic analysis for flexible and rigid pavements*). These guidelines provide a reasonable basis for the back-analysis of road pavements but it should be borne in mind that there are many examples of very poor interpretation of the deflection bowl data and many serious and expensive errors resulting from over reliance on the back analysis programs. Basically the FWD measures the deflection bowl accurately but its proper and reliable *automatic* interpretation requires more sophisticated analysis programs than are currently available. Therefore good analysis relies on the skill of the analyst who will make use of the deflection data but only as *one of the various data sets* at his disposal.

Nevertheless the value of the central maximum deflection is essential for analysing road pavements and determining appropriate rehabilitation measures. Other characteristics of the deflection bowl can provide extremely valuable *comparative* information. A performance chart of FWD deflection data may be plotted to show the variation of pavement response along the road. With reference to Table D1 (Appendix D), the chosen deflection criteria are usually d_1, d_6 and (d_1-d_4).

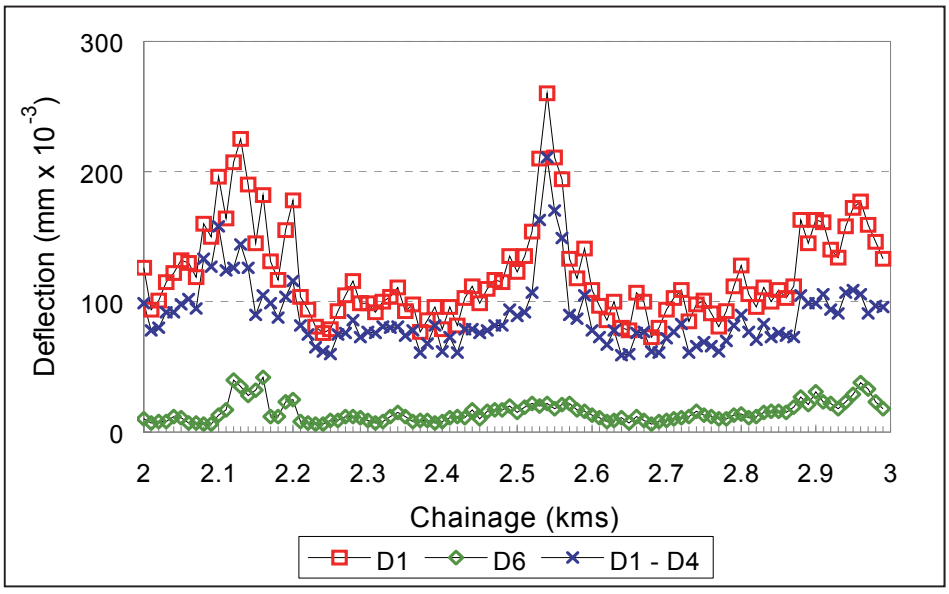


Figure 6-2 Deflection parameters plotted along the road

The maximum deflection, d_1, gives an indication of overall pavement performance whilst the deflection difference (d_1-d_4) relates to the condition of the bound pavement layers.

Deflection d_6 is an indicator of subgrade condition. Although actual values of deflection will depend on the type and condition of the pavement layers, such plots show relative differences in their condition and give an indication of any structural weaknesses.

6.2 Dynamic cone penetrometer tests

The DCP is an instrument which can be used for the rapid measurement of the in situ strength of existing pavements constructed with unbound materials. Measurements can be made down to a depth of approximately 800mm or, when an extension rod is fitted, to a depth of 1200mm. Where pavement layers have different strengths, the boundaries between them can be identified and the thickness of each layer estimated. The operation of the DCP and the analysis of the results are described in Appendix B.

DCP tests are particularly useful for identifying the cause of road deterioration when it is associated with one of the unbound pavement layers, e.g. shear failure of the roadbase or sub-base. A comparison between DCP test results from subsections that are just beginning to fail and those that are sound will quickly identify the pavement layer which is the cause of the problem.

In some circumstances it is convenient to convert the individual pavement layer thicknesses and strengths measured in the DCP test into a simple numeric which represents the combined strength of the pavement layers. This is done by calculating the Structural Number (SN) as shown in Equation 6.1.

$$SN = a_1 h_1 + a_2 h_2 + a_3 h_3 + \qquad \text{Equation 6.1}$$

Where

h_1, h_2 h_3, etc are the thicknesses of each layer in *inches*

a_1, a_2, a_3 etc are the strength coefficients for each layer.

The layer coefficients are related to standard tests for the pavement materials and are fully described in the AASHTO Guide for Design of Pavement Structures (1993) and shown in Appendix A. To take into account variations in subgrade strength, the modified structural number (SNC) can also be calculated (Hodges et al, 1975), as shown in Equation 6.2.

$$SNC = SN + 3.51 (\log_{10} CBR) - 0.85 (\log_{10} CBR)^2 - 1.43 \qquad \text{Equation 6.2}$$

Where

CBR is the in situ CBR of the subgrade at each DCP test point.

If it is suspected that the road failures are related to the overall structural strength of the pavement, the Modified Structural Number of different sub-sections can be readily compared to identify the weakness.

6.3 Destructive sampling and material testing

When the results of the condition survey indicate that the properties of the asphalt surfacing could be the cause of differential performance between sub-sections (see

Sections 8.1-8.6) then this should be confirmed by further testing. Sufficient 150mm diameter core samples need to be taken from each subsection to ensure that representative values for the composition and properties of the asphalt surfacing are obtained. Prior to testing, the cores must be examined to establish the following:

1. thickness of each bound layer;
2. degree of bonding between asphalt layers;
3. occurrence of any stripping; and
4. depth of cracking (if required).

Where only the thickness of the asphalt surfacing is to be measured, then 50-100mm diameter cores are satisfactory. Similar cores can be used for transverse core profiles, such as those shown in Figure 3.6, which are used to confirm whether shoving is the result of shear failure in the surfacing or in one of the lower unbound pavement layers.

When deflection measurements and DCP results indicate that either the thickness or properties of the lower pavement layers are the cause of the differential performance, then test pits are needed to obtain additional material information to confirm these results. The recommended procedure for carrying out test pit investigations is given in Appendix F.

All these investigations are used both to provide an explanation for the present behaviour of the pavement and to provide information for its rehabilitation. Each test pit will provide information on the thickness of each pavement layer and properties of the material. These can then be compared to specified values**.**

6.4 Skid resistance tests

When the detailed surface condition survey indicates that the surfacing has poor texture or polished aggregate then a quantitative survey will usually be required. This survey can be dispensed with if the road is suffering from other failures that require the road to be resurfaced.

The texture depth of bituminous surfacings is measured by the sand patch test (BS 598, 1990). The test procedure is described in Appendix E. There are also other relatively low cost instruments, such as the Mini-Texture Meter (Department of Transport, 1994a), which give continuous measurements of surface texture and are quicker and more convenient to use. However, the results from texture meters need to be calibrated against the sand patch test if they are to be compared with specifications. The sand patch test gives a single value of texture at one point and therefore a number of tests are needed to give a representative value for the road. This is done by selecting sections of road, 50 metres long, which cover the range of severity of the defect recorded during the detailed condition survey. A mean of ten tests, usually in the verge side wheel path, should be used to characterise each 50-metre section. Sections should also be chosen in hazardous areas such as the approaches and crowns of bends. These values can then be compared to national standards, where they have been established, to identify the lengths of the road that need resurfacing.

The microtexture, in terms of the ‘skid resistance’ value (SRV), can be measured using the portable skid resistance tester (RRL, 1969) (ASTM, E 303-93). The test procedure is described in Appendix E. A representative value of SRV can be obtained in a similar way to that described for texture depth, with the mean value of ten results being used to characterise a 50-metre section of road. These values can then be compared to national

standards, where they have been established, to identify the lengths of the road in need of resurfacing.

If national standards are not available then those recommended in the UK may be used as a guide, Table 6.2.

Table 6-2 Suggested minimum 'skid resistance' values

Type of site	Minimum SRV @ 20°C
Roundabouts,	65
Bends with radius <50m,	
Gradients > 1 in 20 and longer than 100m	
Approaches to traffic lights	
Roads carrying more than 2000 vpd	55
All other sites	45

7. SEPARATING DATA INTO HOMOGENOUS SECTIONS

It will be noted that at various stages during the data collection procedures it has been recommended that the road is divided into uniform or relatively homogenous sections based on the data collected to date. At each stage the use of 'performance charts' (Chapter 5) has been recommended to assist with this. Thus relatively homogeneous sections have been defined based on similar construction, current strength, surface condition, deflection and roughness. There are a number of statistical techniques that can be used to do this. One of these techniques is the 'cumulative sum' method, where plots of the cumulative sums of deviations from the mean against chainage can be used to identify the uniform sections. The cumulative sum is calculated in the following way.

$$S_i = (x_i - x_m) + S_{i-1}$$

where

x_i = data value at chainage i
x_m = mean value
S_i = cumulative sum of the deviations from the mean value at chainage i.

Using the cumulative sums, the extent to which the measured data on sections of road varies from the mean value of the whole road can be determined. Changes in the slope of the line connecting the cumulative sums will indicate a change in characteristics and the boundary between homogenous sections. Figure 7.1 illustrates the method.

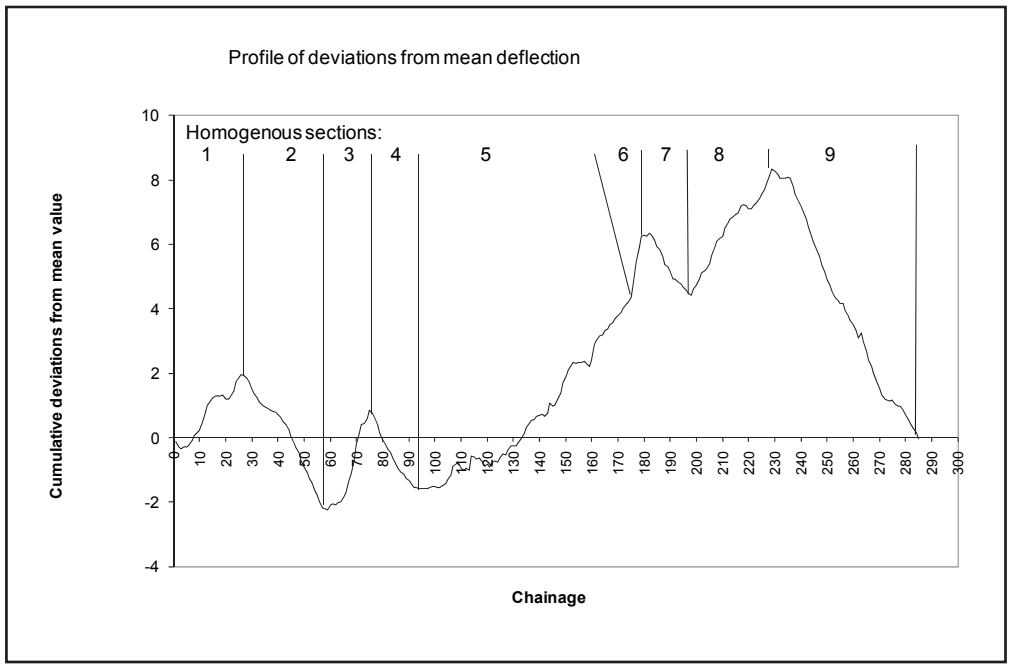


Figure 7-1 A typical cumulative sum plot

The coefficient of variation (CoV = standard deviation/mean) may also be used to determine the level of homogeneity using following guidelines.

CoV < 0.20	good homogeneity
0.2 < CoV < 0.3	moderate homogeneity
CoV > 0.3	poor homogeneity

CoVs greater than 0.3 usually indicate a highly skewed distribution. This can be produced, for example with deflection data, by a number of relatively 'stronger' points within a weaker section. Other authorities have recommended that a homogeneous section is one where the values have a CoV of 0.25 or less.

The final stage of the procedure is to calculate the representative data values for each homogeneous section of the road. The proposed method will tend to separate out areas of extreme data values in areas that warrant special treatment or reconstruction and therefore the distribution of the remaining data measurements will approximate to a normal distribution. The representative data value, which is usually the 90th percentile value, can then be calculated as follows:

Representative data value = mean + 1.3 standard deviation.

8. IDENTIFYING CAUSES OF PAVEMENT DETERIORATION

8.1 Introduction

The last stage in the evaluation procedure is to establish the cause or causes of the pavement deterioration by interpreting the data collected during the surface condition survey and the additional testing. The causes of deterioration combined with the extent of the failures must be considered together when selecting the most appropriate method of maintenance or rehabilitation. These are summarised in Table 8.1 and 8.2.

Besides the surface defects such as ravelling and bleeding, bituminous surfaced roads will generally deteriorate either by rutting or by cracking. To help identify the cause of the deterioration, rutting and cracking have been subdivided into six categories based on the nature of the failure, its position and the type of road construction. These are;

1) rutting without shoving
2) rutting with shoving
3) wheel path cracking - asphalt surfacing
4) wheel path cracking - thin bituminous seal
5) non-wheel path cracking - asphalt surfacing and
6) non-wheel path cracking - thin bituminous seal.

A method of establishing the probable cause or causes of pavement deterioration is given in the flow charts shown in Figures 8.1-8.8. These charts do not cater for all the types and stages of pavement deterioration. In particular, when a road has received a series of maintenance treatments or when the initial deterioration is masked by further progressive failures, the problem of identifying the initial cause of failure becomes more complex. However, the charts provide a framework that enables highway engineers to develop their own pavement evaluation skills. The charts identify general causes of deterioration but do not attempt to establish specific material problems because this can only be done by further destructive sampling and subsequent laboratory testing.

8.2 Rutting without shoving

These ruts are usually wide because they are caused primarily by movement deep in the pavement structure, and there will be little or no evidence of shoving at the edge of the ruts. This type of rutting is the result of two possible causes, either insufficient load spreading or secondary compaction.

Insufficient load spreading is the result of the pavement layers being too thin to protect the subgrade. It is characterised by an increase in rutting with traffic loading. Where there is historical data on the progression of rutting and traffic, or where there is a significant difference in traffic loading between the two lanes, then this relationship can be established. More usually this information will not be available and it will then be necessary to show a relationship between the severity of rutting and the deflection of the road pavement at the time of the evaluation (see Figure 6.1). If deflection equipment is unavailable, a similar analysis can be completed by relating the severity of rutting to the strength of the road, as measured by the DCP.

If the severity of rutting does not relate to the strength of the road pavement, the most likely cause of the rutting is secondary compaction of one or more of the pavement layers

by traffic during the early life of the road. In this case the rate of increase in rutting will decrease after the initial compaction phase.

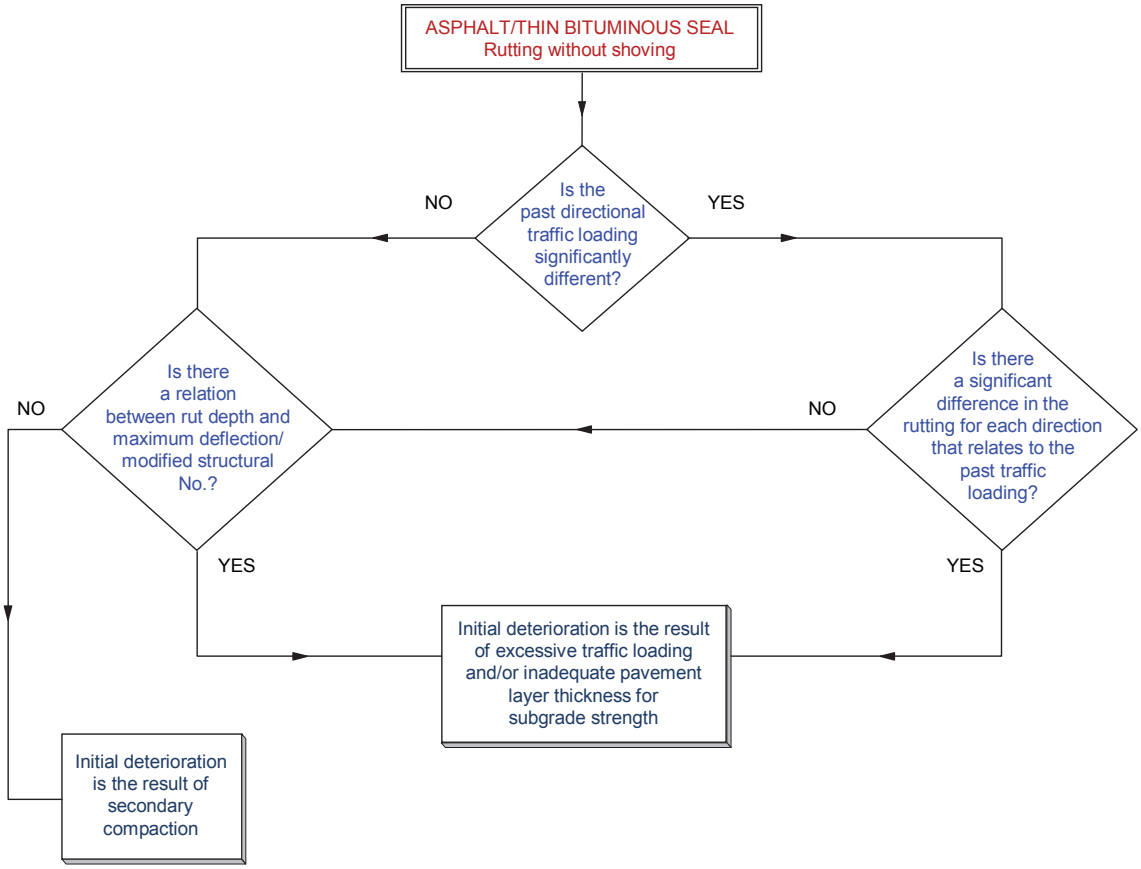


Figure 8-1 Rutting without shoving

8.3 Rutting with shoving

Shoving parallel to the edge of the rut is indicative of a shear failure in one of the pavement layers and is caused by the pavement layer having inadequate shear strength to withstand the applied traffic stresses at that particular depth in the pavement. Unlike the rutting described in Section 8.2, the severity of the rutting will not usually be related to the overall strength of the pavement as indicated by either its deflection or modified structural number.

The failures are usually confined to the upper pavement layers where the applied traffic stresses are at their highest. A process of elimination is used to identify which layer has failed. If the pavement has asphalt surfacing then a transverse core profile can be used to establish in which bituminous layer, if any, the failure is occurring. If the failure is not in the asphalt surfacing then the DCP can be used to identify which of the underlying pavement layers is the cause of the failure. This is done by comparing the strength of the layers in the failing (but not completely failed) areas with those that are sound.

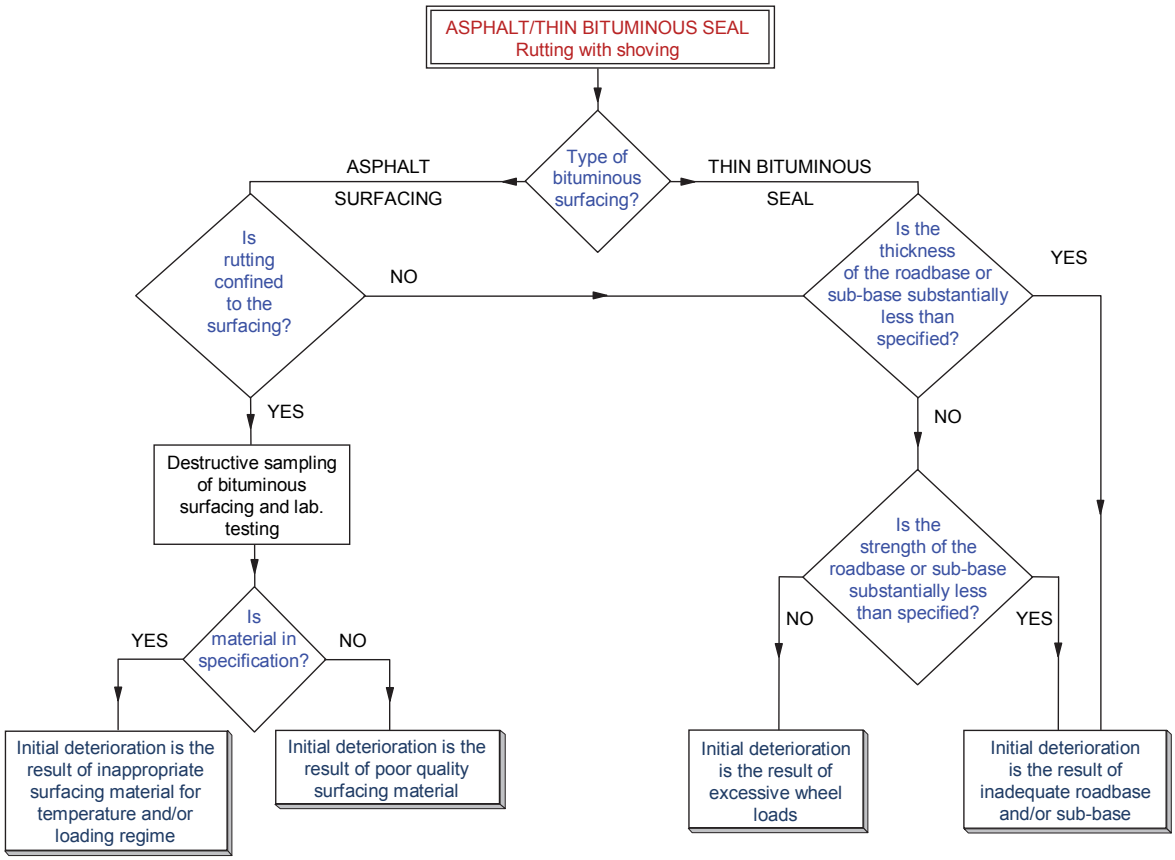


Figure 8-2 Rutting with shoving

8.4 Wheel path cracking - asphalt surfacing

If cracking is caused primarily by traffic it must, by definition, originate in or near the wheel paths. In severe cases it is sometimes difficult to be sure whether the failures start in the wheel path or whether they are a progression of another form of cracking.

Short irregular longitudinal cracks in the wheel paths are often the first stage of traffic induced fatigue of the surfacing. After further trafficking these interconnect to form crocodile cracks. Although caused by the flexure of the surfacing, they are not necessarily 'traditional' fatigue cracks which start at the bottom of the asphalt surfacing and propagate upwards. In tropical climates the bitumen at the top of asphalt wearing courses oxidises rapidly. This causes the material to become brittle and results in cracking being initiated at the top of the surfacing rather than at the bottom, despite the strains being lower.

Where crocodile cracks are shown, by coring, to have started at the bottom of the asphalt layer, then they are likely to be 'traditional' fatigue cracks caused by excessive strains at the bottom of the surfacing (coring confirms this only if all cracks are found to extend right through the cores and, even then, it is not certain that they started at the bottom). Excessive strains can be caused by a weak subgrade, giving rise to large maximum deflections, or a weak roadbase leading to small radii of curvature. This type of crocodile cracking very rarely occurs without any rutting; in the former case, because of insufficient load spreading; in the latter case, because of shear failure in the roadbase.

In some circumstances traditional fatigue cracking can occur simply because the road has reached the end of its design life; in other words no other form of failure has occurred beforehand. This is a relatively rare phenomenon and for this reason is sometimes difficult to identify because of the need to calibrate standard asphalt fatigue relationships for local conditions. However, the age of the surfacing and the traffic carried should provide the most important clues.

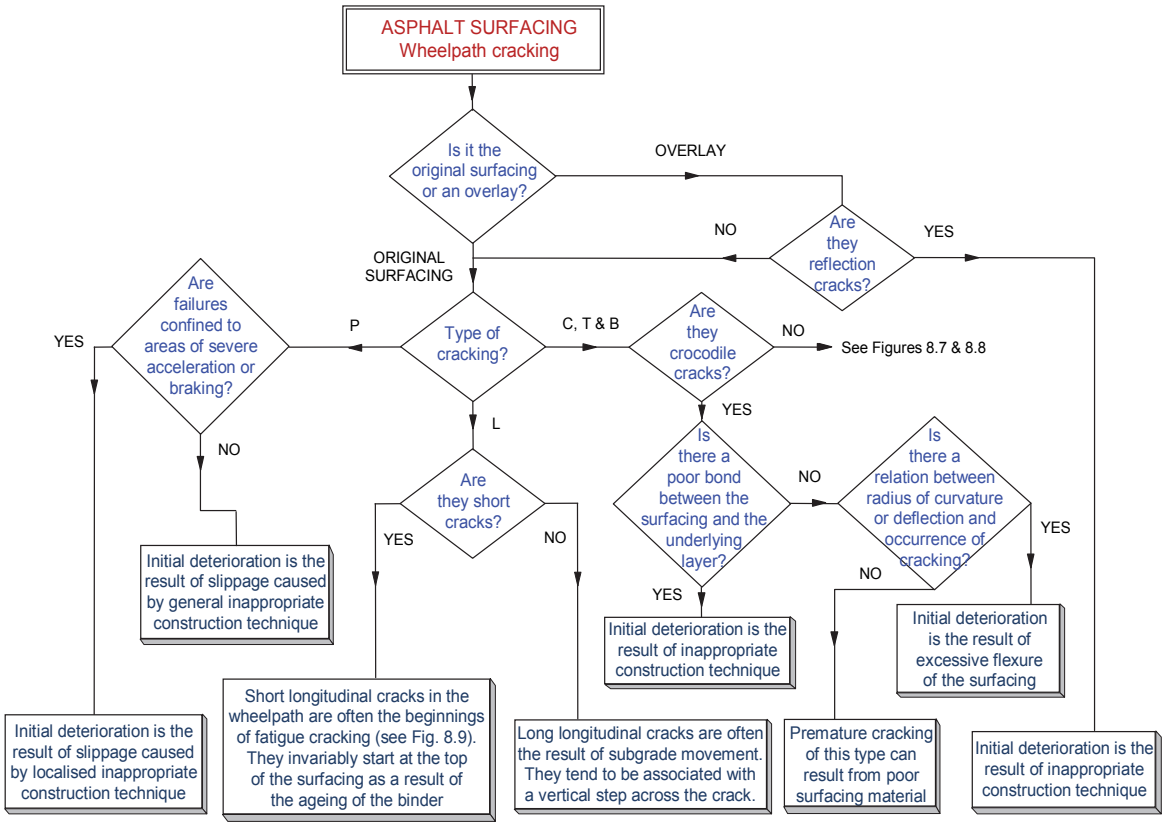


Figure 8-3 Wheel path cracking - asphalt surfacing

Poor quality surfacing materials can also result in crocodile cracking. Inadequate quality control exercised during the manufacture and construction of dense surfacings can lead to poor particle size distribution, low bitumen contents, segregation and poor compaction, all of which will make the material more susceptible to cracking. Failures of this type can occur in areas where deflections are satisfactory and where little or no rutting is occurring.

If the bond between the asphalt surfacing and the underlying layer is poor then the surfacing can effectively 'bounce' under traffic. This quickly results in crocodile cracking in the wheel paths and is characterised by blocks of less than 200mm square. The cause of the poor bond can be ineffective priming of the roadbase or deficient tack coat prior to placing an overlay. Often the cracking will progress to laminations, which are shallow potholes that are clearly the result of the surfacing 'peeling' off.

Parabolic shaped cracks in the surfacing which occur in areas of severe braking such as the approaches to junctions and sharp bends are caused by slippage and are also the result of a poor bond. Small areas of parabolic cracking are not indicative of serious failure. However, if it is more extensive, the probable cause is an inadequate tack coat or the use of soft

aggregate in the surfacing which, in breaking down, results in a poor bond and subsequent slippage.

Cracking in bituminous overlays, particularly in the wheel paths, can be caused by cracks in the underlying layer 'reflecting' through the overlay. Reflection cracking will generally occur early in the life of the overlay and is often associated with pumping of fine material from a lower granular layer. Cores cut through cracks in the new overlay will establish whether they are being caused by existing cracks in a lower pavement layer.

8.5 Wheel path cracking - thin bituminous seal

The bitumen film in surface dressings is very thick compared to the effective film thickness in asphalt surfacings and it is more tolerant to flexure under traffic. Errors in the design or construction of these seals are more likely to result in failures such as bleeding or loss of stone rather than cracking. However, as the seal gets older, age hardening of the bitumen can result in wheel path cracking or fretting. If cracking is being caused by excessive flexure under traffic then it will be associated with areas of high deflections.

Where the surfacing has been used to seal an existing cracked asphalt layer, any subsequent cracking may be caused by the reflection of cracks from the previous surfacing. Slurry seals are particularly susceptible to reflection cracking.

Bituminous seals having a poor bond with the underlying roadbase will behave in a similar way to that of an asphalt surfacing. In this case any water going through the resultant cracking will aggravate the poor bond, resulting in the rapid formation of potholes. This can be a particular problem with stabilised roadbases if they are not primed effectively prior to surfacing.

8.6 Non-wheel path cracking - asphalt surfacing

The cause of non-traffic associated cracking in an asphalt surfacing is largely established by identifying its type. As traffic has played little or no part in these road failures the cracks will not be confined to the wheel paths and there will not be any substantial rutting. Non-wheel path cracking can take the form of longitudinal, transverse, block or crocodile cracking.

8.6.1 *Longitudinal cracking*

Thermal stresses can cause cracks to appear along poor longitudinal construction joints and in areas of severe temperature gradients, such as the edge of road markings. In their early stages neither of these types of crack is particularly serious; however, if left unsealed, the cracks will eventually spread into the wheel paths where they will result in more serious deterioration.

Where longitudinal and transverse cracks occur in combination, they are likely to be either reflection cracks propagating from a lower stabilised layer or cracks caused by thermal or shrinkage stresses in the asphalt.

Longitudinal cracks caused by subgrade movement will generally be quite long and can meander across the carriageway. They can occur because of poor construction, swelling in a plastic subgrade or embankment materials, and the settlement or collapse of embankments. Cracks caused by the slippage of an embankment will often occur in semi-

circular patterns and both these and cracks caused by subgrade movement will often be associated with a vertical displacement across the crack.

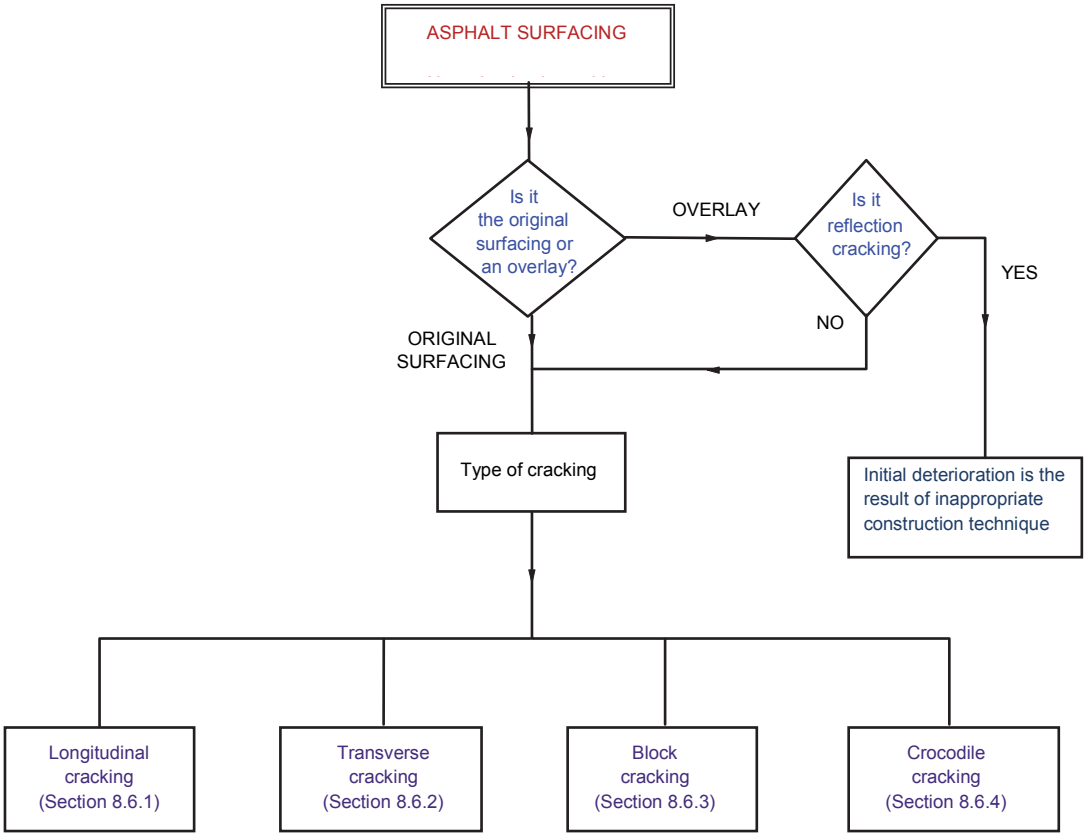


Figure 8-4 Non-wheel path cracking - asphalt surfacing

8.6.2 *Transverse cracking*

Transverse cracks in the surfacing of a road pavement which includes either a chemically stabilised roadbase or sub-base are likely to be reflection cracks from the stabilised layer, particularly if the stabiliser is cement. These forms of transverse cracking are often associated with longitudinal cracks and, in severe cases, block cracking.

If the transverse cracks are irregularly or widely spaced they are likely to have been caused by some form of construction fault. Differential vertical movement caused by consolidation or secondary compaction adjacent to road structures and culverts can cause transverse cracks in the surfacing. These cracks will be associated with a poor longitudinal road profile caused by the differential movement.

Transverse cracks confined to the surfacing and occurring at more regular and short spacing are probably caused by thermal or shrinkage stresses. This type of cracking will most likely occur in areas subject to high diurnal temperature changes, such as desert regions, and will be exacerbated by poor quality surfacing materials. When cracks occur after many years of good performance it is likely that progressive hardening of the binder has made the surfacing more ‘brittle’ and therefore more susceptible to cracking. As transverse thermal cracks progress, they will link up with longitudinal ones to form block cracking. Thermal stresses can also cause cracks to open up at transverse construction joints.

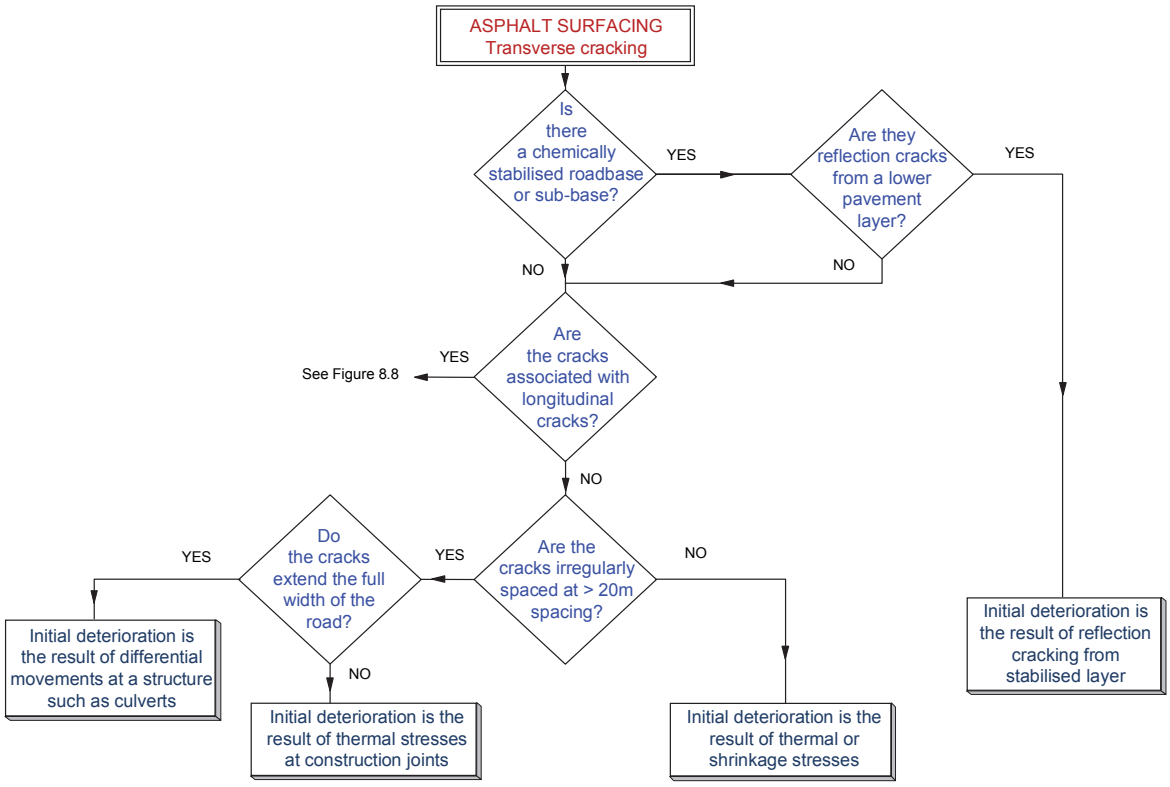


Figure 8-5 Transverse cracking

8.6.3 *Block cracking*

Block cracking, when confined to the bituminous surfacing, is usually the final stage of cracking due to thermal stresses. These cracks almost always start at the top of the surfacing and propagate downwards. Block cracking can also occur through reflection of the shrinkage crack pattern in lower chemically stabilised layers.

8.6.4 *Crocodile cracking*

Crocodile cracking that is neither confined to the wheel paths nor associated with rutting is indicative of a fault in the construction of the surfacing. The more common production faults are poor particle size distribution, low binder contents, overheated bitumen and the use of absorptive aggregate. Construction faults include poor compaction, segregation of the mix and poor bonding, either between layers of bituminous material or the granular layer beneath. In these cases the precise cause of failure can only be determined by destructive sampling and laboratory testing.

8.6.5 *Non-wheel path cracking - thin bituminous seal*

Roads having thin bituminous seals are less susceptible to the non-traffic associated failures described in Section 8.6.1 and 8.6.2 because their thicker bitumen film results in a higher strain tolerance. Surface dressings, in particular, are less likely to crack either at construction joints or alongside road markings. They are also less susceptible to thermal or shrinkage cracking. Where strains are large, however, as in the case of reflection cracking from a stabilised roadbase or from subgrade movement, the surfacing failure will be similar to that described for asphalt surfacings.

8.7 Summary of the diagnostic process

The selection of appropriate rehabilitation or maintenance is based on a number of considerations. Firstly, the cause of deterioration in the existing pavement must be correctly identified and its importance assessed. Secondly, attention should be given to the nature, extent and severity of the deterioration to check what effect it will have on the treatments that are being considered. Finally, the strategy must be economically viable taking into consideration both the costs of maintenance and the vehicle operating costs over a number of years.

The results of the diagnostic process described above and the appropriate method of repair or rehabilitation are described in Table 8.1 for surface dressed roads and in Table 8.2 for roads with an asphalt surface.

8.8 Construction considerations

The careful and correct preparation of the existing pavement prior to the construction of an overlay is essential to optimise overlay performance. Much of the deterioration that occurs in overlays results from deterioration that was not repaired adequately in the existing pavements.

The condition of the existing pavement consists of three basic types namely;

i) Deep structural weakness.
ii) Surfacing defects that are structural in nature.
iii) Relatively superficial surface defects that are not themselves structurally critical but can affect the behaviour of the overlay.

8.8.1 *Deep structural weakness*

The overlay thickness is designed to correct a below-average pavement condition but not to provide the extra structural strength needed for *localised* weak areas. These weak areas should have been identified either during the condition surveys and/or also during the design of the overlay itself. They must be corrected to provide a uniform foundation for the overlay. This is done by means of patching. The depth of patching depends on the magnitude of the weakness and the layer or layers that are defective. A measurement of these aspects should be available from DCP measurements and therefore the depth required can also be determined.

In general, such weak areas are likely to require deep patching to sub-base level or below. The aim should be to design the patch to be at least as strong as the strongest part of the existing pavement and made with the same type of material e.g. crushed stone if the existing base is crushed stone or cement-stabilised gravel if the existing base is cement-stabilised.

The main problem is estimating the extent of the weak area and therefore the size of the patch. The surface condition often provides valuable clues but great care should be exercised to make sure that the size of the weak area is not under-estimated. Additional DCP or deflection measurements may be required.

8.8.2 *Structural surfacing defects*

This includes most forms of cracking and rutting. The severity determines whether a layer is 'beyond repair' and must be removed. In general, low levels of cracking can be covered with an overlay but deforming asphalt cannot. All the conditions and options are described in Tables 8.1 and 8.2.

8.8.3 *Reflection crack control*

Reflection cracks are a frequent cause of overlay deterioration. Pre-overlay repairs (patching and crack filling) help to delay or prevent the occurrence (and subsequent deterioration) of reflection cracks but the possibility of occurrence is high and it is often better to remove a cracked AC and replace it entirely as recommended in Table 8.2. However, research over many years has shown that some forms of reflection crack control can be effective. These include:

i) Use of pavement fabrics
ii) Crack relief layers. These are usually composed of open-graded coarse aggregate with a small percentage of asphalt cement or merely a strong granular unbound layer.
iii) Increased overlay thickness.

The use of pavement fabrics is not an easy option and specialist expert assistance should be sought before employing them.

Crack relief layers are often recommended for severe cases of cracking. However, they are effectively a reconstruction option. This is because two complete layers need to be constructed (the crack relief layer and the asphalt overlay), and the crack relief layer needs to be at least 100mm thick. The existing pavement layers offer more support than a basic sub-base but are likely to be very variable in properties. Completely removing the old surfacing, re-working the existing base and designing as for a new pavement may be the least expensive option.

8.8.4 *Sub-drainage*

The existing sub-drainage condition of the pavement should be evaluated since it has a great influence on how well the overall pavement will perform. Removal of excess water from the pavement cross-section will increase the strength of the pavement layers and subgrade, and reduce deflections (ERA *Drainage Design Manual and Pavement Design Manual*).

8.8.5 *Milling-Recycling*

Milling the existing surface can improve the performance of the overlay, whether or not the intention is to recycle the milled materials. Milling removes cracked and hardened material and minimizes the existing rutting or other significant distortions. It also has the advantage that, because of the reduced level of irregularities, the need for excessive overlay thickness to remove them is decreased. The resulting overlay or any regulating layer can then be thinner.

8.8.6 *Surface recycling*

Recycling of existing surfacings is now a realistic option, especially where good quality aggregates are scarce. The use of recycled materials is described in the ERA *Pavement Design Manual*.

8.8.7 *Shoulders*

Overlaying traffic lanes generally requires that the shoulders be overlaid to match the grade line of the traffic lanes. In selecting an overlay material and thickness for the shoulder, the designer should consider the extent to which the existing shoulder has deteriorated and the amount of traffic that will use the shoulder. For example, if trucks tend to park on the shoulder at certain locations, this should be considered in the shoulder overlay design.

If an existing shoulder is in good condition, any deteriorated areas should be patched. An overlay may then be placed to match the shoulder grade to that of the traffic lanes. If an existing shoulder is in such poor condition that it cannot be patched economically, it should be removed and replaced.

8.8.8 *Reconstruction of flexible pavements*

If the pavement is to be reconstructed with new materials, only the subgrade of the existing pavement will remain. Thus reconstruction does not differ from designing an entirely new road as described in the ERA *Pavement Design Manual*.

Table 8-1 Repair and rehabilitation of surface dressed roads (without an AC surface)

Primary failure	Remedial treatment	New surfacing	Comments
Rutting without shoving (Section 8.2 and Fig 8.1)			
Secondary compaction		Thin overlay.	
Excessive traffic loading or inadequate pavement thickness		Strengthening overlay possibly also with regulating layer	See Section 9.5 in addition to Sections 9.2 to 9.4
Rutting with shoving (Section 8.3 and Fig 8.2)			
Excessive wheel loads	Remove surfacing and replace or modify existing roadbase.	Double surface dressing or depending on traffic.	Existing roadbase may be suitable for stabilisation with cement
	Add bituminous roadbase.	AC wearing course (depending on traffic)	Check that existing roadbase is suitable to be re-worked for use as sub-base
Inadequate roadbase			
Too thin	Remove surfacing and increase roadbase thickness with granular overlay.	Double surface dressing	Existing roadbase must be scarified and re-compacted
Too weak	Remove surfacing and replace or modify existing roadbase	Double surface dressing	Existing roadbase may be suitable for mechanical stabilisation or modification with cement or lime
	Remove surfacing and construct new roadbase	Double surface dressing	Check that existing roadbase is suitable to be re-worked for use as sub-base
Inadequate sub-base			
Too weak			Regard sub-base as subgrade and re-design whole pavement accordingly
Wheel path cracking (Section 8.5)			
Poor bond	Remove surfacing where the bond is poor and patch		
Excessive flexure exacerbated by age hardening of the binder	Remove areas of cracking of intensity 4 or greater and patch. Chase out any cracks >3mm wide and seal with crack sealant	Surface dressing (Note 2)	Check whether road needs strengthening
Reflection cracking	Remove areas of cracking of intensity 4 or greater and patch. Chase out any cracks >3mm wide and seal with crack sealant	Surface dressing (Note 2)	
Non wheel path cracking (Section 8.6.5)			
Reflection cracking	Chase out any cracks >3mm wide and seal with crack sealant	Surface dressing if reflection cracking has extent > 1 (Note 2)	
Subgrade movement	Immediately chase out and seal all cracks to prevent the ingress of water		Pavement should be sealed with a double surface dressing after crack development has stabilised (Note 2)

Notes

1 The design of the new seal must account for the hardness of the existing surface (see surface dressing design in the ERA *Pavement Design Manual)*

2 Some organisations recommend the use of fabric stress absorbing layers. However, these are not recommended until they have been introduced on a pilot scale and contractors are suitably trained.

Table 8-2 Repair and rehabilitation of roads with an AC surface

Primary failure	Remedial treatment	New surfacing	Comments
Rutting without shoving (Section 8.2 and Fig. 8.1)			
Secondary compaction.		Thin overlay.	
Excessive traffic loading or inadequate pavement thickness.	Reflection crack treatment if necessary.	Strengthening overlay with regulating layer.	See Section 9.5 in addition to Sections 9.2 to 9.4
Rutting with shoving (Section 8.3 and Fig. 8.2)			
Inappropriate surfacing material or surfacing out of specification.	Remove surfacing that has failed.	Replace with a new asphalt surfacing layer.	Make sure rutting is confined to the surfacing. See Note 1
Inadequate roadbase.			
Too thin.	Remove surfacing and increase roadbase thickness with granular overlay.	New asphalt surfacing.	
Too weak.	Remove surfacing and replace or modify existing roadbase.	New asphalt surfacing.	Existing roadbase may be suitable for mechanical stabilisation or modification with cement or lime.
	Remove surfacing and construct new roadbase.	Double surface dressing.	Check that existing roadbase is suitable to be re-worked for use as sub-base.
Wheel path cracking (Section 8.4 and Fig. 8.3)			
Isolated slippage.	Remove affected surfacing and patch.		
Extensive slippage.	Remove surfacing and replace.	Asphalt surfacing.	
Cracks confined to the top of the surfacing.		Double surface dressing (Note 2) or milling of top 25mm and replacement with new AC.	
Poor bond.	Remove surfacing where the bond is poor and patch.		Where the failures are extensive the surfacing must be removed and the road resurfaced with asphalt.
Poor surfacing material.	Remove areas of cracking of intensity 3 or greater and patch. Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing or asphalt surfacing (Note 2).	Where the failures are extensive the surfacing must be removed and the road resurfaced with asphalt.
Fatigue cracking.	Remove areas of cracking of intensity 3 or greater and patch. Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing or asphalt surfacing (Note 2).	Where the failures are extensive check whether the road needs strengthening.
Reflection cracking.	Remove areas of cracking of intensity 3 or greater and patch. Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing or asphalt surfacing (Note 2).	If a crack relief interlayer is to be used under an asphalt surfacing then areas of crack intensity 4 or greater should be removed and patched.
Non wheel path cracking (Section 8.6 and Fig. 8.4)			
Longitudinal cracks			
(i) At construction joints and road markings.	Chase out cracks and seal with crack sealant.		
(ii) Subgrade movement	Immediately chase out		Pavement should be sealed with

	and seal all cracks to prevent the ingress of water.		a double surface dressing after crack development has stabilised (Note 3).
(iii) Reflection cracking	Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing if reflection cracking has extent > 1 (Note 2).	
Block cracking.			
(i) Thermal or shrinkage cracks.	Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing (Note 2).	If the block cracking is severe the surfacing must be removed and replaced.
(ii) Reflection cracking.	Chase out any cracks >3mm wide and seal with crack sealant.	Double surface dressing if reflection cracking has an extent greater than 1 (Note 2).	If the cracking is severe the surfacing must be removed and replaced.
Crocodile cracking.	Remove surfacing.	New asphalt surfacing.	Check condition of underlying layers.

Notes

1 Mix designs for severe conditions are described in the ERA *Pavement Design Manual*. Many authorities also use modified binders for additional reliability.

2 Some organisations recommend the use of fabric stress absorbing layers. However, these are not recommended until they have been introduced on a pilot scale and contractors are suitably trained.

9. OVERLAY AND REHABILITATION DESIGN PROCEDURE

9.1 Surfacing problems

Localised surfacing problems are discussed in Chapter 4. More extensive surfacing problems are included in Table 8.1 and Table 8.2 and discussed in Chapter 8.

9.2 Overlay design

Chapter 8 describes the causes of pavement deterioration and their characteristics thereby providing the engineer with the information required to make a full diagnosis of the causes and an evaluation of the severity of the deterioration. The principle of overlay design is that if the failure of the existing pavement is not too far advanced, it should be possible to strengthen the road so that it can carry traffic for many more years. If the deterioration is far advanced then a more substantial form of rehabilitation will be required.

There are three general methods of overlay design as follows;

1. An empirical pavement strength/thickness approach using the AASHTO concept of Structural Number or an equivalent empirically based pavement design method.
2. An analytical approach based on reducing calculated critical stresses to safe levels.
3. A load spreading approach based on elastic deflections.

The first two methods are essentially the same as methods for designing new roads and depend on knowledge of the strength of the subgrade, the strength of the pavement layers, their thicknesses and so on. The additional element for rehabilitation design is simply the method for determining the 'residual strength' of the existing pavement which is needed in order to determine the extra strength required. However, a considerable amount of additional information is available for designing rehabilitation that is not available when designing a new road. This comprises the elastic deflections of the existing road and knowledge of the performance of the road to date.

Deflection-based methods have been developed by many agencies and have been used successfully for many years.

The problem is that the three methods rarely result in the same design solution. This is not surprising because empirically based design methods depend on measurements of the *strength* of pavement layers whereas deflection based methods depend on the *elastic* properties of the layers. Unfortunately the elastic properties of pavement materials are very poorly correlated with strength properties. This is illustrated in Figure 9.1 where an example of the relationship between modified structural numbers (*SNC*) of existing pavements and their maximum deflections under a standard load (measured at exactly the same locations as the *SNC* values were calculated) is shown. Ideally, for any specific pavement type, there should be little scatter and the points should lie close to a well-defined line.

The analytical methods, despite their apparent attraction, do not solve the problem. A discussion of analytical methods is presented in Section 9.6 but it is appropriate to note here that analytical methods need to be calibrated against empirical evidence before they can be used reliably; hence they need to be calibrated against one or other of the two empirical alternatives. Furthermore, in all analytical methods in regular use today, the

elastic modulus of the subgrade (and usually other pavement layers as well) is assumed to relate linearly to the strength of the layer, usually as measured by CBR. Thus the fundamental problem that the elastic and strength properties are poorly correlated is not addressed.

Quite clearly, effective rehabilitation design requires knowledge of *both* the elastic (load spreading) properties *and* the strength properties of the materials and both are available through deflection, DCP tests and test pit data.

[It is noted that as a result of this, the most common approach in rehabilitation design manuals is to recommend that the design is carried out using several methods separately and then choosing the most conservative solution].

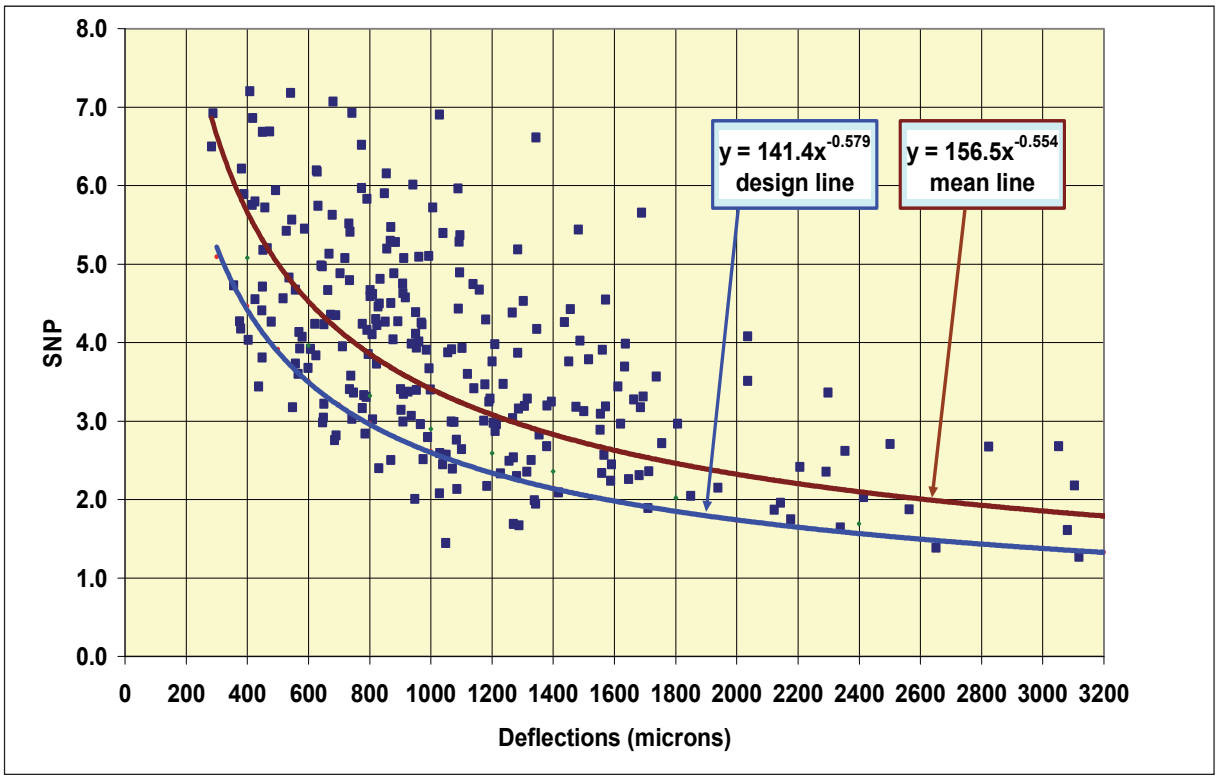


Figure 9-1 Relationship between modified structural number and central deflection (example)

9.3 The empirical design method

The process of designing the rehabilitation for each uniform section of road is as follows;

1. Estimate the design traffic.
2. Determine the target structure that will carry the design traffic.
3. Evaluate the existing pavement to determine its 'residual strength' and the deficiency between this and the strength required.
4. Calculate the strengthening requirements

9.3.1 *Design traffic*

This procedure is identical to the method for designing a new road and is described in detail in the ERA *Pavement Design Manual.*

9.3.2 *Required structure*

The second step is to determine a road structure that will carry the design traffic for the required period of time, usually 15 or 20 years. This process is identical to designing a new road and should be done using the design method described in the *Pavement Design Manual*. This requires knowledge of the subgrade strength. The in-situ values will have been measured during the detailed condition survey using a DCP (Section 6.2 and Appendix B). Equivalent soaked laboratory CBR values will also be available for a few of the chainages from the test pit data (Appendix F). These test pits should have been excavated at exactly the same locations as some of the DCP measurements were taken so the subgrade CBRs are directly comparable. These data can then be used to estimate the equivalent soaked CBR value at each DCP measurement. However, it should be remembered that the empirical pavement design charts in the *Pavement Design Manual* are based not on the soaked CBR value of the subgrade soil (which is an arbitrary value) but on the likely worst *in situ* value. Thus taking the DCP measurements towards the end of the wet season provides a much better estimate of the subgrade strength that should be used for design. This also eliminates the problem of the unknown subgrade density since the in situ strengths are measured at whatever the in situ density happens to be.

Both the required structural number (SN_R) and the required modified structural number (SNC_R) must be calculated for each DCP test chainage as indicated in Section 6.2 and described in detail in Appendix A. The actual value is dependent on the type of structure that is to be built. If no roadbase strengthening is to be carried out then a bituminous overlay will require the designs in Charts C, D or E depending on the type of roadbase present. If the roadbase is to be strengthened, then depending on whether it is to be stabilised with cement, lime or with bitumen, Charts D or E are required. The structural numbers and modified structural numbers required for each subgrade strength, traffic level and structural type are summarised in Appendix A.

If the pavement requires complete reconstruction then the subgrade will be reprocessed and the methods described in the design manual for new roads should be followed.

9.3.3 *Estimating the structural deficiency*

Step three is the critical step where the strength or traffic carrying capacity of the existing pavement is assessed. In many cases the residual strength will be very low because the deterioration will be far advanced (Chapter 8) and the thickness of any strengthening overlay will be excessive. In this situation an overlay is unlikely to be the best option. However, provided the structural deficiency is not too large an overlay is likely to be the best solution.

The required structure at each DCP measurement chainage (i.e. where subgrade strength has been measured) has been determined from the required traffic carrying capacity and the subgrade strength in Step 2 above. The difference between this and the existing structural capacity is the structural *deficiency*. The structural deficiency should be interpreted to indicate three things namely:

i) where the road requires reconstruction of some form
ii) where overlaying is the best solution

iii) the thickness of overlay required.

The structural capacity of the existing road is calculated using a combination of DCP measurements, a structural number approach and deflection measurements.

The DCP is used to determine the thickness and strength of each pavement layer including the subgrade as described in Appendices A and B and the actual modified structural number (SNC_A) determined for each DCP measurement.

The SNC_A values are then plotted against the deflection values measured at the same point resulting in a graph similar to Figure 9.1 but with less scatter (since the structure of the road will usually be of the same type along its entire length). The deflection values are more closely related to traffic carrying capacity and subgrade 'protection' than the *SNC* values. However, the graph shows that at any particular value of deflection there is likely to be a range of SNC_A values. It is the lowest SNC_A value for a particular deflection that determines the 'effective' strength of the pavement or SNC_{eff}. In order to determine this value for every test point a curve must be fitted to the data bounding the lowest SNC_A values as shown in Figure 9.1. In this way the SNC_{eff} value can be determined for each test point.

The structural deficiency at each test point is then calculated as follows;

$$\text{Structural Deficiency} = SNC_R - SNC_{eff}$$

An example of the structural deficiency chart along a road is illustrated in Figure 9.2.

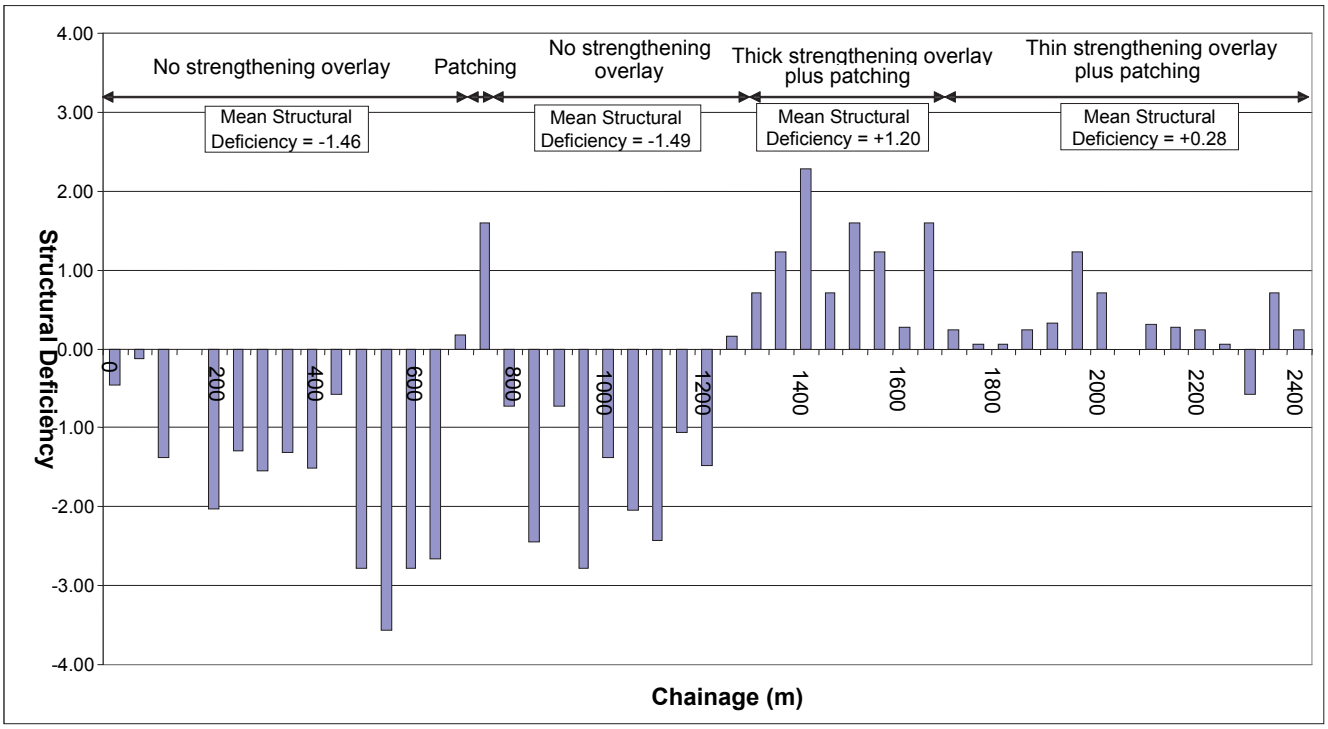


Figure 9-2 Example of a structural deficiency chart

9.3.4 *The overlay thickness*

The required overlay thickness is calculated accurately on a point by point basis as follows;

$$\text{Overlay thickness at test point (mm)} = 25.4*(SNC_R - SNC_{eff})/a_1$$

Where a_1 is the strength coefficient for the asphalt overly (usually 0.35 – see Appendix A). An example of the point by point overlay thicknesses is shown in Figure 9.3.

It is now necessary to re-examine whether the existing selection of uniform sections can be improved. The example in Figures 9.2 and 9.3 illustrates four well-defined areas.

The overlay thickness to be used for each uniform road section is obtained by selecting the appropriate percentile of the thickness distribution depending on the reliability desired.

Adjustments to this calculation are required from a statistical point of view to ensure that an appropriate level of reliability is used. In particular, weak areas that appear to need a very thick overlay should be patched before the overlay is applied. If the patching is done properly they should then be strong enough to require little or no additional strengthening. These areas should be excluded from the calculation of overlay thickness percentiles.

Conversely the structural deficiency of some areas of the 'uniform' section of pavement may be negative, indicating that they do not, apparently, require additional strengthening. Although these areas are currently strong, the durability of the surfacing, as a result of ageing, is likely to be just as low as that of the rest of the pavement. Hence an overlay or other surface treatment is also required in these areas. If the sections not requiring strengthening are relatively small and randomly distributed so that changing the overlay thickness is not practicable (this will be the case if the 'uniform' sections have been selected properly) then these areas should also be excluded from the calculation of overlay thickness percentiles.

Thus by excluding these areas in the calculation of overlay thickness percentiles, the true reliability of the rehabilitated pavement will be slightly higher than determined in the calculations because these areas will be stronger. This provides a small additional safety factor.

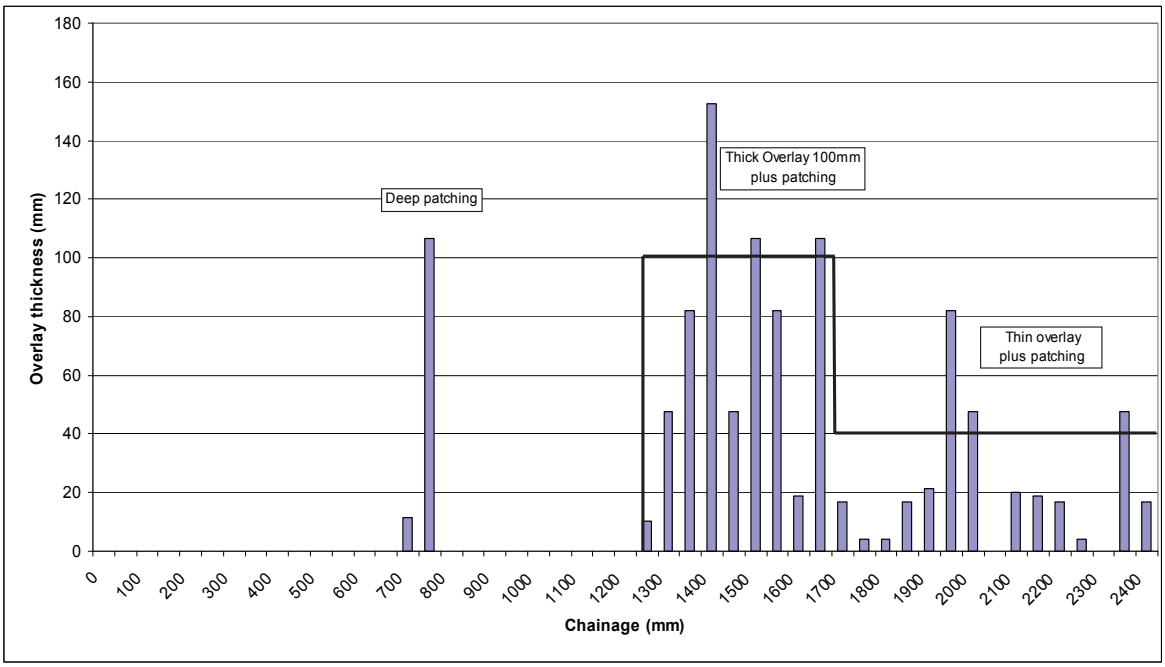


Figure 9-3 Overlay thickness at each deflection point and final selection of uniform sections for overlaying

The newly defined 'uniform' sections are analysed separately to determine the appropriate overlay thickness. For each one a cumulative overlay thickness distribution is plotted and the appropriate percentile selected. Table 9.1 shows the percentiles to use for each class of road. Figure 9.4 illustrates a typical cumulative frequency distribution to determine the overlay thickness for different reliability percentiles.

Table 9-1 Values of overlay reliability factors

Road class	Percentile for design
DC7	98
DC6	95
DC5	90
DC4	85
DC3	80

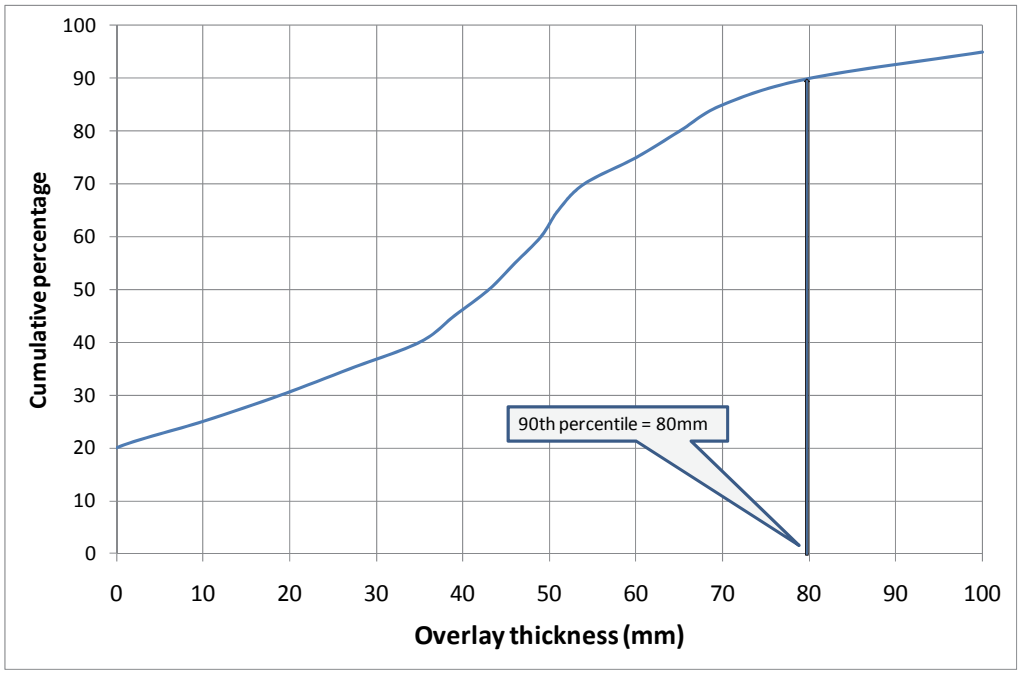


Figure 9-4 Frequency distribution of overlay thicknesses

If the structural deficiency is close to zero and predominantly negative the road may merely display a poor profile (i.e. high IRI value) as a result of surfacing defects and require only a thin overlay to improve the riding quality and to provide a new durable surface. The minimum thickness of thin overlays is governed by the aggregate grading of the overlay material. Where the mix has a Maximum Stone Size of 25mm, the overlay should be 65mm thick. Where the Maximum Stone Size is 19mm, the material can be laid with a minimum thickness of 47mm.

In general, if the mean structural deficiency lies in the range between 0 and 0.6, a thin overlay is also required. If the mean structural deficiency lies between 0.6 and 2.5 then a thick overlay is necessary. The need for partial or full reconstruction is less easy to define but is highly likely if the magnitude of structural deficiency is greater than 2.5. Under such circumstances the visual condition data, DCP and test pit data need to be re-assessed. From the design point of view only, full reconstruction is relatively straightforward. Table 9.2 summarises these recommendations.

Engineering judgement becomes much more important where partial reconstruction is indicated. In general, roads with good foundations can be partially reconstructed by making use of much of the existing material in the form of enhanced sub-base or even lower roadbase layers. With the addition of a stabilising agent, rehabilitation to full roadbase strength is easily achievable. Roads which have a very weak foundation require more elaborate remedial works and full reconstruction is often required. Recycling of any asphalt-based pavement materials could also be considered in a full reconstruction option. The design of roads that require reconstruction should be done in accordance with design recommendations set out in the ERA *Pavement Design Manual Volume 1*.

Table 9-2 Structural deficiency criteria

Mean Structural Deficiency	Action	Notes
Zero or negative	Maintain with a surface treatment (e.g. a surface dressing)	A thin overlay can be used to correct other road defects
0.0 - 0.6	Thin overlay	Remedial works possible
0.6 – 2.5	Design thick overlay (45 - 180mm)	Remedial works probable
> 2.5	Partial reconstruction probable	

9.3.5 *Final iteration and buildability*

The final step is to consider buildability and eliminate too frequent changes in type of rehabilitation. All defined uniform sections for overlay should be as long as possible commensurate with the class of the road. Thickness changes should be very gradual to avoid contributing to long wavelength roughness or undulations which are very uncomfortable on high speed roads. The ERA unevenness specifications apply.

9.4 Using the empirical method with partial data

A situation often arises when some of the information required for a comprehensive analysis as described above is not available. Whilst such a situation is regrettable and leads to less accurate design, it is sometimes unavoidable. This section describes approaches designed to deal with this problem. There are four possible situations.

Situation A. This is the ideal situation where there is a 'full' set of data, namely:

(i) DCP data
(ii) deflection data *at the same points* as DCPs
(iii) surface condition data (including specific data at the test points) and
(iv) test pit data.

This is the situation described in Section 9.3 above.

Situation B. Only DCP data are available, deflection data are unavailable.

In these situations the most critical difference is that the amount of data available is likely to be much less than desired. This is because DCP tests are much more time consuming to carry out than deflection tests and therefore there are likely to be relatively few of them. The absence of deflection tests also means that reliance has to be placed on other ways of estimating SNC_{eff} from the DCP-measured *SNC* values.

The reduced amount of data means that there will be far less opportunity to reduce the variability that has to be catered for in the designs by defining uniform sections. In turn, this means that designs are likely to be more expensive.

It is assumed that a Figure such as Figure 9.1 is available from previous road evaluation studies in the region on the same type of road so that an estimate of the C_f factor at each deflection value is also available, although it will not be project specific. The C_f factor is defined as:

$$C_f = SNC_{eff}/SNC_{\text{mean}}$$

The effective Structural Numbers for rehabilitation design must be obtained by reducing the directly-measured values (by DCP) by an amount similar to the average reductions at each deflection value illustrated in the appropriate local version Figure 9.1.

The C_f factor is quite important and the question arises as to whether it is actually larger than estimated from the SNC/d_0 relationship. During the development of the design process it was found that C_f depended on d_0. In the example shown in Figure 9.1, C_f decreases as d_0 increases from about 0.8 at a deflection of 400 microns down to 0.7 at a deflection of 2000 microns.

Note that in Situation A, the values of SNC_{eff} obtained from deflection values are actually very much better than those obtained by calculating them from a DCP-measured *SNC* by multiplying by C_f. This is because, on the SNC/d_0 plot, there is a range of values of *SNCs* for every deflection value. At each deflection value all these *SNC* values will be reduced to the same effective value (i.e. the one relating to that value of d_0). This is what is required.

However, if deflection values are not available and *all* the *SNCs* at a particular deflection value are multiplied by the *average* C_f, the correct value of SNC_{eff} for design will be the average value so calculated, not the lower limit. The result of this problem is a likely small systematic error because a project specific version of Figure 9.1 is not available.

Situation C. Both deflection and DCP data are available but they do not correspond to the same chainages.

This is a fairly common situation. The best that can be done is to use the nearest deflection value for each of the chainages where DCP measurements were made. Usually there will be one which is nearest or two that are a similar distance away. In the latter case engineering judgement or a simple average needs to be used. The result of this problem is greater scatter in the data and therefore larger values of overlay thickness to achieve the same reliability.

Situation D. Deflection data are available but there are no DCP data.

This situation is similar to Situation A except that use has to be made of an SNC/d_0 relationship obtained from elsewhere. The main problem arises because the strength of the subgrade is not known. It might be argued that this can be estimated from a back-analysis of the FWD data but experience has shown that this is far more difficult to do with sufficient accuracy despite much literature that indicates the contrary. However, the back-calculated subgrade moduli from FWD tests will give an excellent indication of the variability; it will identify the weakest and strongest areas and help to define uniform sections of road.

An estimate of the subgrade strength for designing the new pavement must be made using the methods outlined in the ERA *Pavement Design Manual,* and using advice in Overseas Road Note 31 for estimating the lower 10-percentile of subgrade strength for different types of soil. Test pit data helps enormously. The other steps in the analysis are the same. Insofar as the design subgrade strength is likely to err on the side of safety, the resulting designs will be more conservative than those that would be obtained if DCP data were also available.

9.5 Deflection only approach

The maximum deflection is used by a number of road authorities to estimate the carrying capacity of a road. This is a simpler method than the method described above but it works primarily in situations where the pavement is failing because it is too thin for the current traffic levels and where the individual pavement layers themselves are not failing. In the diagnostic process of determining the cause of failure, this type of failure is characterized by a reasonable relationship between deformation and rut depth as indicated in Figure 9.5.

In general, this type of failure occurs on roads with a flexible bituminous surfacing such as a surface dressing and if the pavement materials are all within specifications but may also occur if the surface is a flexible AC layer. In other words the failure is a traditional failure at subgrade level. The rehabilitation design does not cater for cracking failures in asphalt surfacings or the many other types of failure described in Chapter 8. However, for the type of failure for which it is designed, it works very well because it includes a built-in calibration procedure.

The deflection criteria curve shown in Figure 9.6 (i.e. the relationship between deflection and traffic carrying capacity) is not necessarily directly applicable to road pavements in Ethiopia. However, it is clear that an overlay reduces the stresses in the lower layers of the pavement and therefore, to prevent deformation in these layers and the subgrade, appropriate deflection criteria can be developed as described below.

The deflection and condition surveys must be carried out after the wettest period of the year when the road pavement can be expected to be at its weakest. The severity of rutting is then plotted against the maximum deflection at each test point and a best fit line and confidence limits calculated as shown in Figure 9.5.

The value of critical deflection corresponding to a defined level of critical rutting is then determined for any particular level of statistical reliability. The 90th percentile is recommended with a critical rut depth of 10mm for roads with asphalt surfacings and 15mm for those with thin bituminous seals.

An estimate of the previous total cumulative traffic is also required. One point can then be plotted on the deflection traffic-loading graph corresponding to the deflection at the critical level of rut depth and the traffic to date. This point is unlikely to lie on an existing criterion curve; but assuming a similar form of relationship, a *calibrated criteria curve* can be obtained by drawing a new line through the point and parallel to the existing curve as illustrated in Figure 9.6.

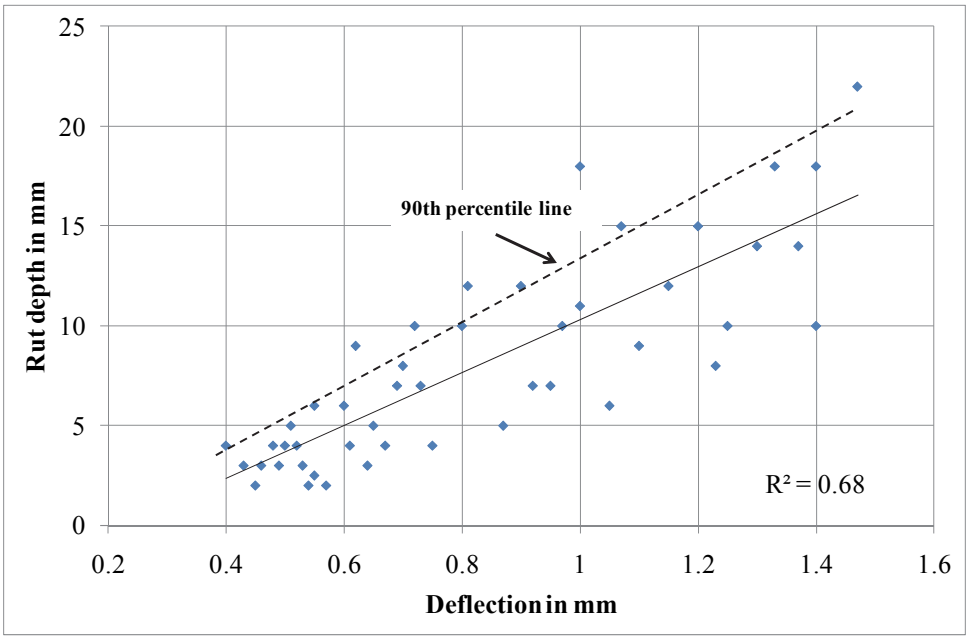


Figure 9-5 Rut depth/deflection correlation

9.5.1 *Deflection procedure*

The deflections can be measured with an FWD or a Benkelman deflection beam. Measurements should be made in both wheel paths of the slow lane on dual carriageways and in both lanes of a two-lane road. The following strategy is recommended.

i) Tests are carried out on a basic pattern of 50 or 100 metre spacing.

ii) Additional tests should be undertaken on any areas showing surface distress.

iii) When a deflection value indicates the need for a significantly thicker overlay than is required for the adjacent section, the exact length of road involved should be determined by additional tests.

After all measurements have been made, they should be corrected for any temperature effect. This is because the stiffness of the asphalt depends on temperature. The temperature of the bituminous surfacing is recorded when the deflection measurement is taken, thus allowing the value of deflection to be corrected to a standard temperature. It is recommended that 30 or 35^{o}C, measured at a depth of 40mm in the surfacing, is a suitable standard temperature for roads in tropical climates.

The relation between temperature and deflection for a particular pavement is obtained by studying the change in deflection on a number of test points as the temperature rises from early morning to midday. It is not possible to produce general correction curves to cover all roads so it is necessary to establish the deflection/temperature relationship for each project. Fortunately, it is often found that little or no correction is required when the road surfacing is either old and age hardened or relatively thin.

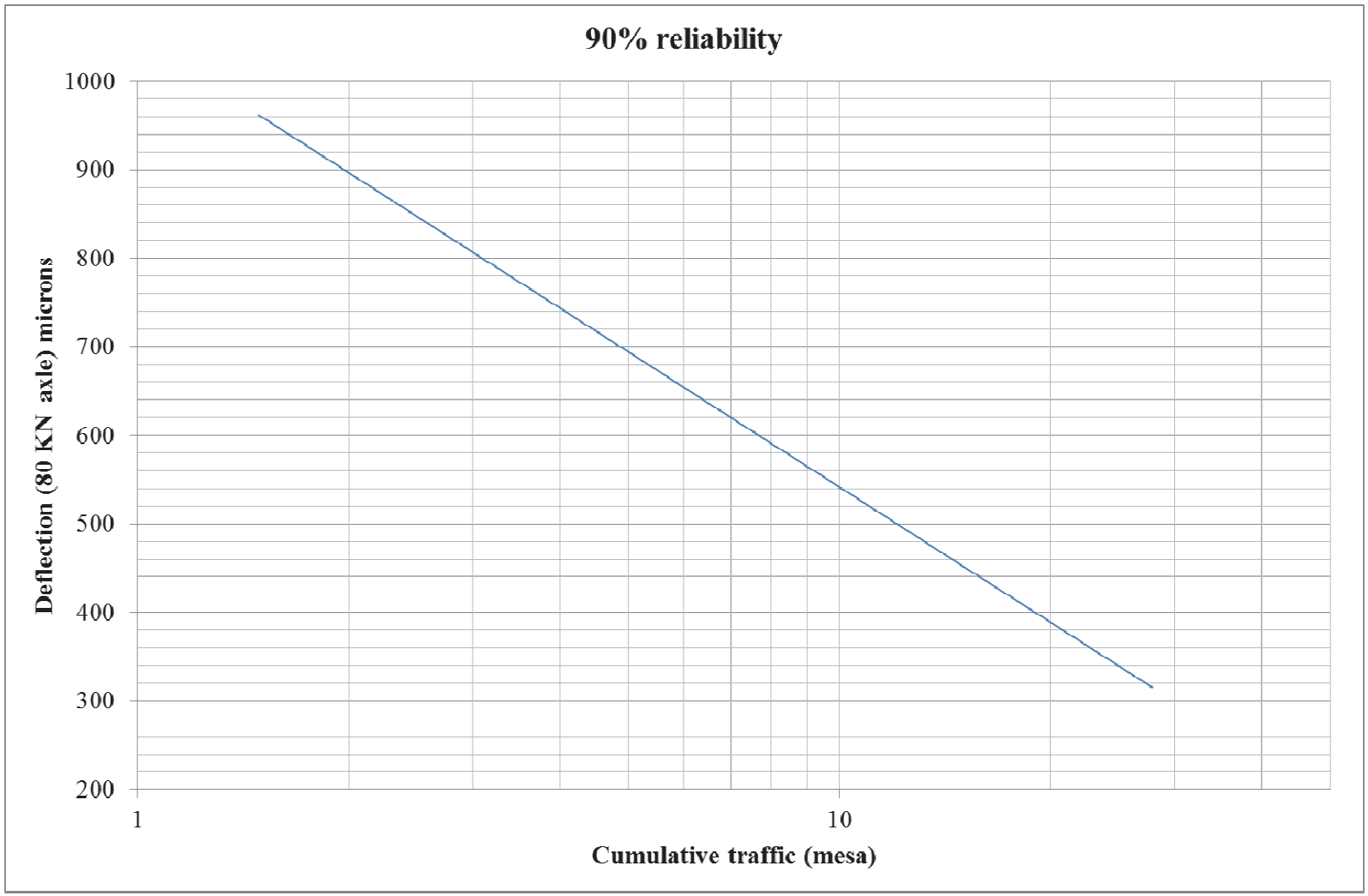


Figure 9-6 Typical relation between standard deflection and traffic carrying capacity for roads with an unbound roadbase and an asphalt surfacing

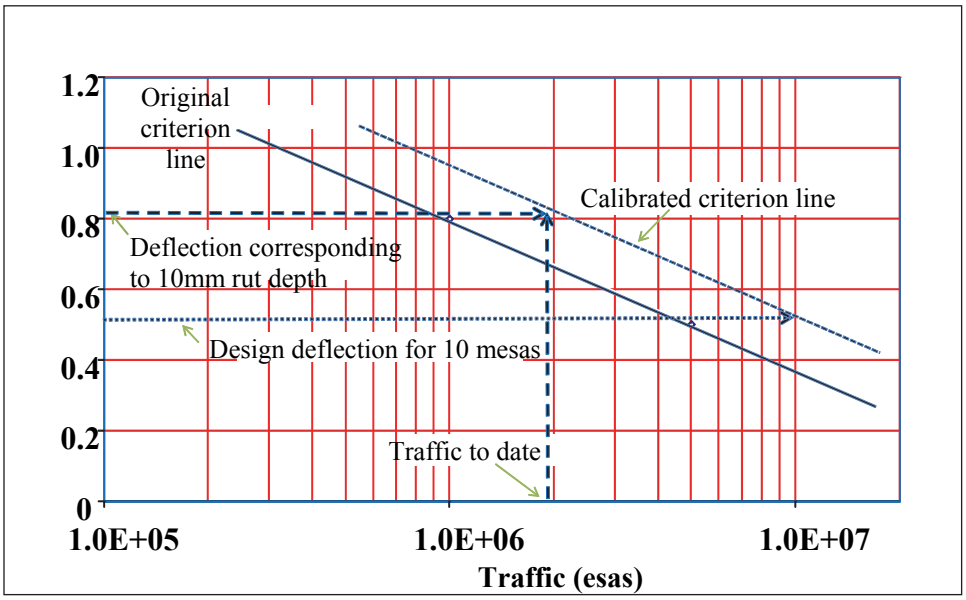


Figure 9-7 Calibration of deflection life criterion

9.5.2 *Performance chart*

A performance chart should be plotted (Chapter 5) of the deflection profile of the road for each lane, using the larger deflection of either wheel path at each chainage. Any areas showing exceptionally high deflections which may need reconstruction or special treatment can then easily be identified. The deflection profile is then used to divide the road into homogeneous sections in such a way as to minimise variation in deflections within each section (Chapter 7). The minimum length of these sections should be compatible with the frequency of thickness adjustments which can sensibly be made by the paving machine whilst still maintaining satisfactory finished levels. When selecting the sections the topography, subgrade type, pavement construction and maintenance history should all be considered (see Chapter 5).

The final stage of the procedure is to calculate the representative deflection for each homogeneous section of the road. The proposed method tends to separate out areas of very high deflections that warrant special treatment or reconstruction and therefore the distribution of the remaining deflection measurements will approximate to a normal distribution. The representative deflection, which is the 90th percentile value, can then be calculated as follows:

Representative deflection = mean + 1.3 x standard deviation

9.5.3 *Traffic carrying capacity*

The traffic carrying capacity of the road, in terms of rutting, can be estimated by comparing the representative deflection of homogeneous sections with the calibrated deflection criteria curve as shown in Figure 9.7. The traffic carrying capacity represents the total traffic loading that the road will carry from construction. Therefore the future traffic carrying capacity is the total traffic loading minus the traffic loading that the pavement has carried prior to evaluation.

9.5.4 *Overlay thickness*

The thickness of any necessary strengthening overlay can be determined based on reducing the representative deflection of the pavement to the design deflection obtained from the calibrated deflection curve. The relation between the thickness of a dense bituminous overlay and the reduction in deflection, under a 62.3kN axle load, is;

$$T = \frac{0.036 + 0.818\ D_r - D_d}{0.0027 D_r}$$

where D_d = Design deflection (mm)

D_r = Representative deflection (mm)

T = Overlay thickness (mm)

This relation is valid between representative deflection values of 0.25 - 1.2mm and overlay thicknesses of 40 - 150mm.

If deflections are measured using a different axle load the results should be directly scaled to give the equivalent value under the 62.3 kN axle.

9.6 Analytical approach

The traffic carrying capacity of an asphalt pavement is governed by how effective the pavement layers are in preventing;

i) fatigue cracking of the asphalt surfacing
ii) shear failure of the granular materials
iii) fatigue cracking or crushing of lightly cemented materials and
iv) wheel path rutting resulting from subgrade failure.

Theoretical models to predict the behaviour of granular and lightly cemented materials under the action of traffic are not well defined and therefore specifications for such layers have always been set in such a way that failures are unlikely. This has mitigated against possible risks in the use of lower quality materials, and has theoretically restricted the range of likely failure modes.

The performance of road pavements has traditionally been dependent on the stress/strain values at two locations in the structure. Thus the horizontal tensile strain at the bottom of the asphalt layer controls one type of fatigue cracking and the vertical compressive strain at the top of the subgrade controls rutting.

For roads having a thin bituminous seal the traffic carrying capacity is determined only by resistance to rutting. The performance of the surface seal itself will generally depend on environmental effects rather than traffic loads.

The traffic carrying capacity of an asphalt surfaced road will be determined by both its resistance to fatigue cracking and wheel path rutting. However, research has shown that the predominant form of surface distress of asphalt surfacings in tropical climates is not fatigue cracking starting at the bottom of the asphalt layer but ‘top-down’ cracking which is initiated at the surface of the layer (Rolt et al, 1986) (Smith et al, 1990). The type and

severity of this form of cracking is a complex function of material properties and both environmental and traffic stresses and its development has yet to be successfully described by means of a practical analytical model. However, such cracking affects only the surface (if dealt with early enough) and therefore a straightforward replacement of the cracked surfacing is required (Table 8.2). On major roads a milling operation is often the best option provided that the cracks do not penetrate through the whole asphalt layer.

Top-down cracks often develop long before other types of cracks and thus the performance of asphalt surfaced roads rarely agrees with the analytical models. Nevertheless it is important that rehabilitation design takes account of all possible modes of future failure and therefore it is important to ensure that traditional fatigue failure of the surfacing and failure through inadequate protection of the subgrade do not occur within the design life required. In order to do this, analytical procedures *properly calibrated* to local conditions provide a suitable method.

The analytical approach requires a suitable mathematical model to describe the pavement. Almost all methods use the multilayer linear elastic model. This model requires, as input, the thickness, elastic modulus and Poisson's ratio of each layer of the pavement. The moduli of asphalt surfacings can be calculated based on mix constituents and binder properties at the design temperature or direct laboratory measurements of modulus made on samples of material extracted from the road. The moduli of other layers can be estimated from DCP tests or from laboratory tests on materials extracted from test pits. However these methods are not straightforward and considerable engineering judgement is required.

The most likely method to be used to determine the effective elastic modulus of each pavement layer is back-analysis of FWD deflection bowls. FWDs are supplied with computer models for this purpose and these are used to estimate the elastic moduli of all the pavement layers. Very thin layers such as an existing seal are normally incorporated with the underlying roadbase or ignored.

The computer programs supplied with most FWDs can also be used to calculate the stresses or strains at the critical points in the pavement under the application of a standard load designed to replicate a 40kN wheel load (80kN axle load). These strains are then used to calculate the 'life' of the structure using relationships between stress/strain and pavement life of the form:

Asphalt fatigue criteria: $\text{Log } N_f = a + b \text{ Log } e_t$

Where N_f = fatigue life in esa

e_t = horizontal tensile strain at the bottom of the asphalt layer

a and b are constants associated with material properties.

A suitable asphalt strain criteria is:

$$N = \left[\frac{6918(0.856 * Vb + 1.08)}{\mu\varepsilon * Smix^{0.36}} \right]^5$$

Subgrade deformation criteria: $\text{Log } N_d = c + d \text{ Log } e_v$

Where N_d = deformation life in esa

e_v = vertical compressive strain at the top of the subgrade

c and d are constants associated with material properties.

The computer programs usually include their own failure criteria of this form but allow the user to input his (or her) own values of the coefficients a, b, c, and d if required. Most programs will also calculate the thickness of overlay based upon additional user specific input data on traffic, design life and so on.

Despite the apparent sophistication of such methods, there are a number of problems. For example it has proved quite difficult to develop subgrade strain criteria for different subgrades. The subgrade at the AASHO Road Test was very weak and the criterion developed there is very conservative. Most subgrades are stronger and less prone to failure and recent research has shown that the range of subgrade criteria can cover three orders of magnitude in terms of traffic. Despite this, the default criterion in most analytical methods is the same for all subgrades and this is usually based on the original criterion developed from the AASHO Road Test. The recommended criteria for Ethiopia are, however, based on more recent research (Janoo, V and E R Cortez (2003) and are as follows:

Subgrade classes S2

$$N = \left[\frac{6000)}{\mu\varepsilon} \right]^{5.714}$$

Subgrade classes S3, S4, S5 and S6

$$N = \left[\frac{10{,}900)}{\mu\varepsilon} \right]^{5.714}$$

Suitable calibration procedures are included in some of the programs supplied with FWDs so that, after adjustment of the pavement model, they can be used with more confidence to estimate the future traffic carrying capacity of a strengthened road and to determine overlay thickness, where necessary. However, the user of such programs should be aware that they are based on assumptions and simplified models that can sometimes lead to erroneous conclusions hence alternative methods of designing overlay thicknesses should always be used in parallel.

10. SELECTION OF PREFERRED ALTERNATIVE

10.1 Introduction

It is assumed that the decision to carry out rehabilitation or major maintenance on a road has been made as a result of a planning process that has identified the 'best' projects to be undertaken in the financial planning period. Such a process will have identified roads in need of restoration and rehabilitation and evaluated them based on economic, strategic and political criteria. The economic aspects will have been based on short term agency costs, long term agency costs (whole life agency costs) or, preferably, total whole life costs (including road user costs and any additional social costs). In order to carry out any whole life cost analysis, the performance of the road following the construction of any of the candidate rehabilitation and maintenance strategies must be predictable and, in order to choose the 'best' projects to be undertaken each year, such calculations need to be carried out for many potential projects. This is one of the goals of a pavement management system but it can only be realised fully by developing the system continuously, calibrating road performance models with up to date performance data resulting from research carried out by the Research Department and from regular improvements based on feedback from the users.

An example of a well-developed system is that based on the current global HDM 4 model. This model was originally developed by the World Bank and has been under continuous development for many years, beginning with the original pilot version (HDM I) and continuing with HDM II (1976) and HDM III (1986). It provides a very comprehensive system for road network management purposes. It has been incorporated in the PMS systems in many countries and used by consultants to develop investment strategies for highway agencies and for packages of projects to be funded by external aid donors. Such planning models however, are *not* design models. They predict *average* road performance and are designed to work at network level, allowing the user to determine, *on average*, the best rehabilitation and maintenance strategy for each of the roads in the network. They therefore obtain the best network level investment strategy but, for each type of road some will perform better than average and some worse than average.

Strictly, they are not designed for use at individual project level because they do not deal with the issue of variability to allow designs to be made for defined levels of reliability and there are usually too many issues affecting individual projects that cannot be taken into account in such a network level model. However, since no similarly comprehensive detailed design models are available, designers use such models for evaluating options.

The following excerpt from Part III of the AASHTO Guide (see References) is pertinent:

> "*….the engineer must recognize that it may be impossible to accurately determine the optimal rehabilitation solution from a rigorous analytical model. However, the user should not be discouraged from employing this approach but rather feel encouraged to use every available tool at his/her disposal to determine the problem cause, identify potentially sound and economic solution alternatives, and then select the most preferred rehabilitation strategy from sound engineering experience*".

Thus for most rehabilitation projects it is expected that some form of whole life cost analysis will be carried out based on expectations about the likely performance of the various options. In other words, the same process that was used in the planning phase to select the roads for inclusion in the current programme is used at project level. The main difference, and it is an important difference, is that at project level much more data will be available and therefore decisions should be improved.

Information concerning road performance must be obtained from the planning branch and/or the research branch of the road authority. If such information is not available at the present time, then the use of the models in HDM 4 or similar models developed elsewhere should be attempted. Most such models utilise calibration factors and therefore they can be calibrated for local conditions. If information is not available for this then the designer must use his/her engineering judgement or advice from elsewhere. Thus an initial whole life cost evaluation of alternatives is possible (sometimes with difficulty) but there are project level issues that need to be taken into account and these usually require judgement rather than calculation.

10.2 Project level issues

The preceding chapters have shown how to determine the causes of deterioration, to determine appropriate rehabilitation options and to design them. Sometimes, from an engineering point of view, there is only one option. This is usually only true if the road requires quite simple repairs or if the diagnosis has not clearly identified the problems with the existing road. The designer may perceive that there are unacceptable risks unless he chooses a conservative option for rehabilitation (e.g. complete reconstruction). Where major works are required and though investigations are carried out, there are almost always several alternatives.

Another common problem is that the designer will be aware of limitations in the local contracting industry and, to avoid the risk of proposing a solution that might not be constructed properly, will propose only tried and tested methods that may be far from the most cost effective.

Limitations also exist in the availability of materials and equipment in the country as a whole, often severely restricting the options available.

If there is an optimum solution from an economic point of view (e.g. cost - benefit ratio or whole life costs analysis) it may not be attainable because of budget constraints.

These issues highlight the need to develop the contracting industry within the country to provide reliable options for rehabilitation and maintenance. Research, demonstration and training are fundamental to this.

However, the design engineer should be able to suggest candidate solutions from the technically feasible options described in Chapter 8 and, on the basis of the above factors and constraints should be able to carry out the necessary whole life cost calculations to arrive at a preferred solution.

10.3 Whole life cost analysis

10.3.1 *Major Costs*

With or without the benefit of a whole life cost model, the designer needs to carry out a cost analysis. This requires inputs of both cost and road performance. Unfortunately, both of these elements are subject to a large degree of uncertainty. For example, the effective life of a rehabilitation technique is subject to the following influences:

i) skill and care with which the work is performed
ii) quality of the materials used
iii) environmental conditions prevalent in the region where the pavement exists
iv) traffic which uses the pavement
v) other rehabilitation and maintenance work being performed concurrently

To eliminate as much uncertainty as possible, it is essential to collect rehabilitation performance data whenever available (e.g. from the PMS Branch, Research Branch).

The major costs to consider in the economic analysis include:

(A) Government costs

i) Initial rehabilitation costs. These should include the costs of pavement preparation (repairs, etc.) required prior to overlay, if this is the alternative considered in the analysis.
ii) Future rehabilitation costs (after the selected design period for the rehabilitation design).
iii) Maintenance costs, recurring throughout the design period.
iv) Salvage return or residual value at the end of the design period.
v) Engineering and administrative costs.
vi) Traffic control costs, if applicable.

(B) User costs

(a) Travel time
(b) Vehicle operation
(c) Accidents
(d) Discomfort
(e) Time delay and extra vehicle operating costs during resurfacing or major maintenance.

There are a number of methods of economic analysis that are applicable to the evaluation of alternative strategies (all are usually available as outputs from the available models).

(1) Equivalent uniform annual cost method, often simply termed the annual cost method.
(2) Present worth method for:
    (a) costs
    (b) benefits
    (c) benefits minus costs, usually termed the net present value method.
(3) Rate-of-return method.
(4) Benefit-cost ratio method.
(5) Cost-effectiveness method.

Either the net present value or the equivalent uniform annual cost may be used to determine life cycle costs for comparisons of alternate pavement rehabilitation strategies. In either case, it is essential that comparisons only be made for analysis periods of equal length. Details of the equations used in the methods are beyond the scope of this manual. They may be found in the HDM models developed by the World Bank.

10.3.2 *Discount factors and other 'central policy' issues*

There are a number of factors required in whole life cost analysis that should be provided by central government to ensure that all projects (at least projects of a similar nature) are compared on the same basis. The discount factor, for example, has a significant effect on the analysis.

To illustrate this, consider the service life of a rehabilitation or maintenance treatment. This is always an important factor and is particularly significant for high-volume roads for which lane closures and traffic delays pose considerable difficulties whenever maintenance or rehabilitation is required. The normal economic principle of discounting future expenditures and benefits means that the importance of the downstream benefits of using long life rehabilitation options is considerably reduced. Whole life cost methods apply a cost to such issues as traffic delays caused by road works in the future and therefore these will, apparently, be properly accounted for. However, such calculations rely on a value being attributable to road user's 'time' and this is always a contentious issue in both developed and developing countries.

These are just two example of factors required in the analysis that should be provided by 'government'. Another example concerns road accidents and their costs. The point is that close liaison with the planning branch is required to ensure that all such input data is consistent with government planning procedures.

10.4 Preferred rehabilitation option

The result of a whole life costing exercise is sometimes inconclusive in that the difference between the candidate treatment strategies, or perhaps just two of them, is very small. The designer should be aware of the uncertainties in some of the input data and in the models being used and aware of the size of the difference between two project options that is statistically significant. Sensitivity studies are always advised to emphasise this issue and, if in doubt, should be carried out. Experience indicates that unless the differences exceed 5% they are probably not significant hence other considerations will become more important.

This is often done by applying agreed weighting factors to the factors that are not included in the whole life cost analysis. Such factors might include;

i) Preference for construction that utilises most local labour.
ii) Preference for using SME's to develop local expertise.
iii) Dislike of the vehicle noise generated by surface dressings (in comparison to asphalt).
iv) Environmental or health concerns about using hot mix rather than cold mix.
v) Environmental considerations associated with opening new quarries and a preference for using recycled materials.
vi) Duration and inconvenience of the construction process.
vii) Other environmental and social issues.

If the whole life costing method that was used excludes factors that are usually costed in such an exercise, these too must be included in this final analysis. In some models, for example, traffic delays at future maintenance activities are not accounted for because, nationally, no value is attached to time costs, hence this may be an important 'non-monetary' factor.

The relative importance of each criterion is assigned by the design team in consultation with ERA. Next, the alternatives are rated or marked according to their anticipated performance against the criteria. Then, the rating of each alternative against each criterion is multiplied by the assigned weight of that factor to achieve a 'score'. Finally, all of the scores for an alternative are summed, and the alternative with the highest score is the preferred solution.

Part B

Maintenance and Rehabilitation of Unreinforced Concrete Pavements

11. REHABILITATION OF UNREINFORCED CONCRETE PAVEMENTS

11.1 Introduction

A correctly designed and properly constructed unreinforced concrete carriageway should provide many years of satisfactory service with limited maintenance. However, if such maintenance is neglected or the techniques are inappropriate, the potential life is unlikely to be realised. This section of the manual provides guidance on maintenance and rehabilitation so that unreinforced concrete roads can achieve the longest possible economic life.

The full rehabilitation of a badly damaged concrete pavement is a major highway engineering project in comparison with the relatively simple Hot Mixed Asphalt (HMA) overlays that are the primary means of rehabilitating flexible pavements. Because of this, rehabilitation of concrete roads is also often restricted to simply overlaying with a relatively thin layer of HMA. Such overlays are used to restore an uneven surface and to maintain deteriorated pavements. In rare cases these overlays can be successful. However the defects in the underlying damaged rigid pavement layer often cause rapid failures in the new HMA overlay. Relatively thick HMA overlays have been used to counteract this but, if the underlying concrete pavement is not stable, increasing the thickness to 100mm does not prevent reflection cracking for any substantial length of time. Therefore HMA overlays on unreinforced concrete are not generally recommended.

As with flexible pavements, identifying the cause of the deterioration is fundamental to rehabilitation design. This manual gives highway engineers guidance on the identification of the common modes of distress and the most appropriate maintenance and rehabilitation methods. The guidance is based on a combination of research programmes undertaken by TRL in tropical environments, the experiences of major highway authorities in countries with a successful history of using concrete pavements and on the experience of the Highways Agency in the UK.

For recommendations on the maintenance of reinforced concrete pavements and continuously reinforced concrete the reader is referred to other sources (e.g. Highways Agency, 2001).

11.2 The maintenance and rehabilitation process

As with all roads, concrete pavements should be assessed and evaluated against the performance criteria appropriate to their traffic levels and location. Their ability to accommodate the desired traffic loading safely, along with an economically justifiable level of maintenance, is therefore key aspects of the optimum design.

The timing of periodic maintenance and rehabilitation is important and the choice of technique may change as deterioration progresses. Each strategy has an appropriate time during a pavement's life and proper timing is essential to good service performance. This is illustrated in Figure 11.1. It is common for pavements to be allowed to deteriorate to a point at which only a restricted number of strategy choices remain. In many cases, a less costly strategy adopted earlier will result in significantly improved whole-life cost benefits.

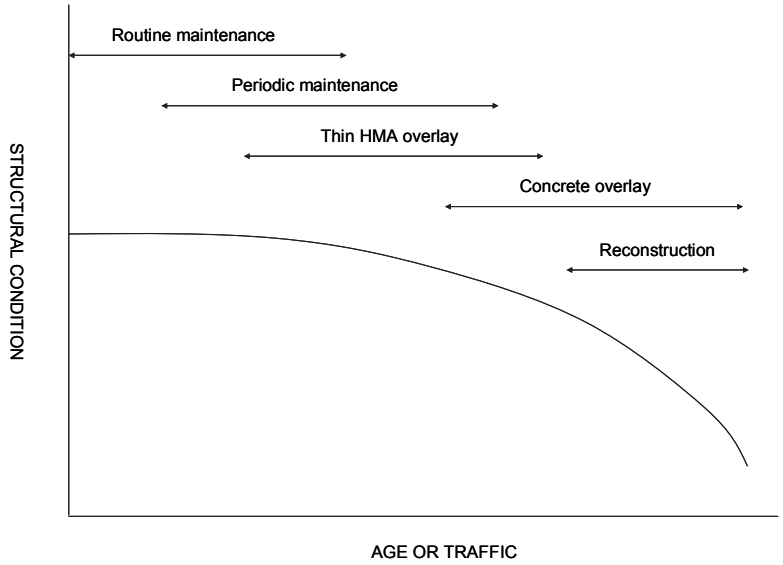


Figure 11-1 Timing for the maintenance and rehabilitation of concrete roads

It is therefore necessary to develop a programme of maintenance and rehabilitation for the road network which addresses both the current condition of the roads and the benefits that can be accrued from the civil works. These benefits are primarily those of the road user, and are quantified by calculating the savings to the road user when any particular road is improved.

Whole life costing has been used by international aid agencies for evaluating highway investments ever since the first operational models were developed in the early 1970s. The whole life costing process compares the total transport costs over the life of a road. This comprises the rehabilitation and maintenance costs and the resulting benefits of reduced road user costs over, typically, a 20-year period for flexible pavements and up to 40 years for rigid concrete pavements. However, few suitable models are available for concrete pavements.

HDM-4 is an economic road investment tool which has been developed to assist engineers/planners with road investment and does, now, include roads with a concrete pavement. HDM-4 predicts the rates of deterioration of roads under alternative maintenance strategies and the road user costs for each alternative. It then conducts an economic analysis of the various alternatives and provides the user with economic indicators to assist with road investment planning. Most importantly it deals with situations where budgets are constrained to a greater or lesser degree.

Maintenance strategies will vary depending on the type of pavement, traffic levels and levels of funding available, both over the short and long term. For example, roads carrying relatively low levels of traffic are likely to be considered less important than roads carrying high levels of traffic. Therefore, because maintenance budgets are limited, the roads with less traffic will generally be allowed to deteriorate more than roads with high levels, prior to rehabilitation.

Using these principles, the PMS in combination with an investment model such as HDM 4 should be used to examine alternative maintenance strategies and then produce a prioritised

programme of works based on economic considerations. This programme of works will identify whether any particular road requires periodic maintenance or rehabilitation. The evaluation and design process is based on Figure 11.2 and assumes that the roads in the network have been selected for civil works through such a prioritised programme of works based on whole life costing.

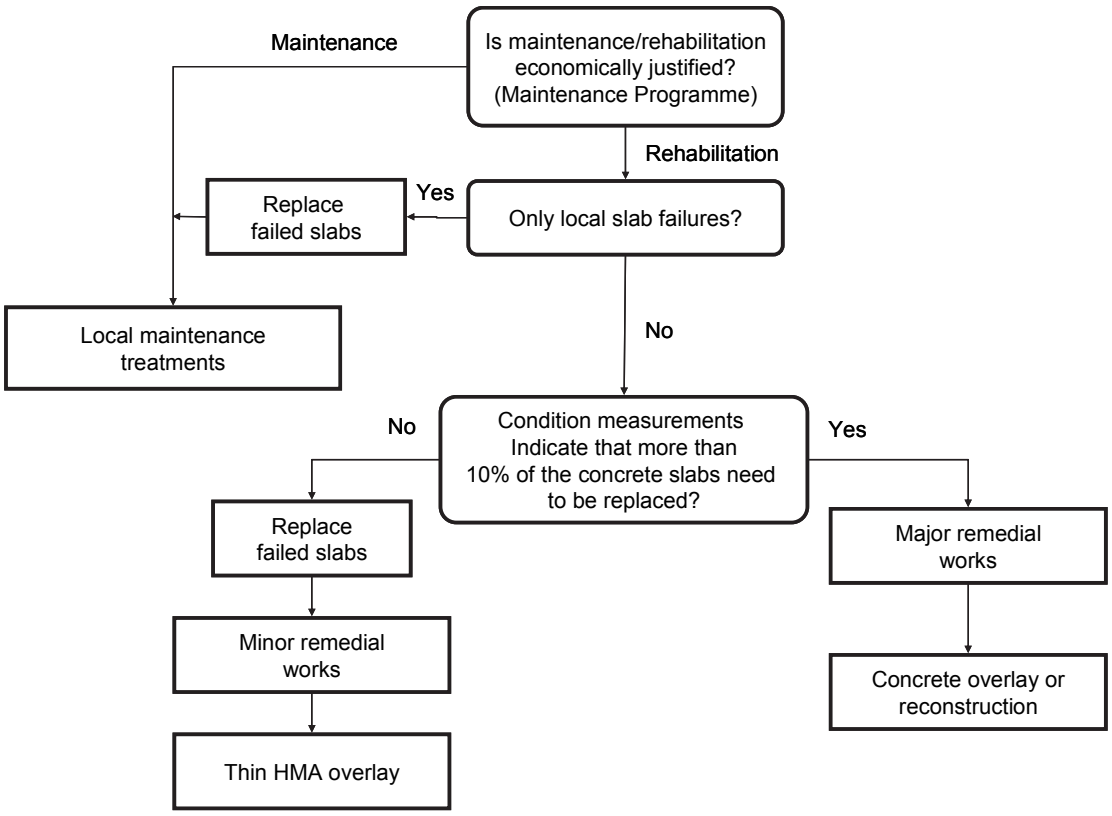


Figure 11-2 Maintenance and rehabilitation process

The maintenance process assumes that the majority of slabs in unreinforced concrete roads will eventually crack as a result of a combination of traffic loading, concrete quality and eventual lack of support. Poor support can occur from a number of causes, for example, poor material, poor construction practice, poor drainage, and so on. Where such failures occur it is better to replace the slab rather than attempt to 'bury' the problem only to find it re-occurs at some later date. Where there are many slabs to remove, it will often be more economical to crack and seat the existing unreinforced concrete road and apply a concrete overlay.

Criteria have been developed to identify those slabs to be replaced based primarily on a visual assessment of the concrete slab and FWD deflections measured at the middle of the slab. These criteria are described in this manual and summarised in Table 15.3.

12. DEFECT CATALOGUE

This section provides a series of photographs to enable the highway engineer to identify the common modes of deterioration of unreinforced concrete roads. The defects have been divided into three categories:

i) Joint defects.
ii) Longitudinal and transverse cracking.
iii) Other forms of deterioration.

The defects and how to deal with them are described in more detail in the next chapter.

12.1 Joint defects



Plate 1 Shallow spalling at joint



Plate 2 Partial spalling at joint



Plate 3 Deep spalling at joint



Plate 4 Faulting at joint



Plate 5 Sympathetic cracking (no groove cut)



Plate 6 Groove cut but too late after slab casting

12.2 Longitudinal and transverse cracking



Plate 7 Typical longitudinal cracks



Plate 8 Extensive longitudinal cracking



Plate 9 Mid-slab transverse crack



Plate 10 Multiple cracking

12.3 Other forms of deterioration



Plate 11 Diagonal crack



Plate 12 Corner crack



Plate 13 Plastic cracking



Plate 14 Depth of plastic cracking

13. DEFECT DIAGNOSIS

Maintenance of a concrete pavement that treats the *symptoms* of distress but does not address the underlying cause of the defect will not be successful. It is important to find out why the defect has occurred before selecting the maintenance treatment. This chapter describes the probable causes of the common modes of failure of unreinforced concrete pavements and the flow charts provide a means of selecting the most appropriate maintenance treatment. The maintenance treatments are referred to as Medium Term (MT) or Long Term (LT). Medium Term is considered to be a repair that is effective for 5-7 years and Long Term is a repair that is effective for more than 7 years.

13.1 Joint seals

Concrete slabs expand and contract as the temperature rises and falls. They also warp, or curl, when the upper surface temperature is substantially different from that of the underside. Longitudinal and transverse joints permit the different movements to occur and so they must be well maintained. Chart 1 identifies the likely problems, their scale and their remedial treatment.

13.1.1 *Transverse joint seals*

Defective joint seals allow silt, grit, stones and water to enter between the slabs and infiltrate the lower levels of the pavement. An accumulation of these materials can stop the joint closing and lead to spalling of concrete or, if several slabs are affected, a 'blow-up' slab compression failure. Penetration of water into the joint can lead to corrosion of steel dowels and tie bars. Where concrete roads are constructed on unbound granular sub-bases, it is particularly important that the joint sealants are effectively maintained. Where the joint sealant fails, water can flush out the fine material immediately under the joint. This results in lack of support at the end of slab and subsequent cracking.

Sealant that is missing or has failed (whether in adhesion or cohesion) may simply have done so because of age, particularly if it has been in place for five or more years. Modern sealants that have been correctly applied (i.e. good supervision during installation) are expected to last seven to ten years unless called upon to accommodate movements outside their design range. Sealant failures within about five years should therefore be further investigated before replacement.

Premature sealant failure may result from:

i) A significant proportion of transverse joints being 'locked up'. Where this happens the remaining joints are subjected to excessive horizontal movements. As a result, the sealant in these joints is likely to fail through excessive extension or compression.

ii) Load transfer between adjacent slabs with dowelled joints is normally such that differential vertical movements between the ends of adjacent slabs do not exceed the limits of the sealant. However, where dowel bars are not used, and support from the lower layers is poor, the vertical movement will be higher thereby causing the sealant to fail.

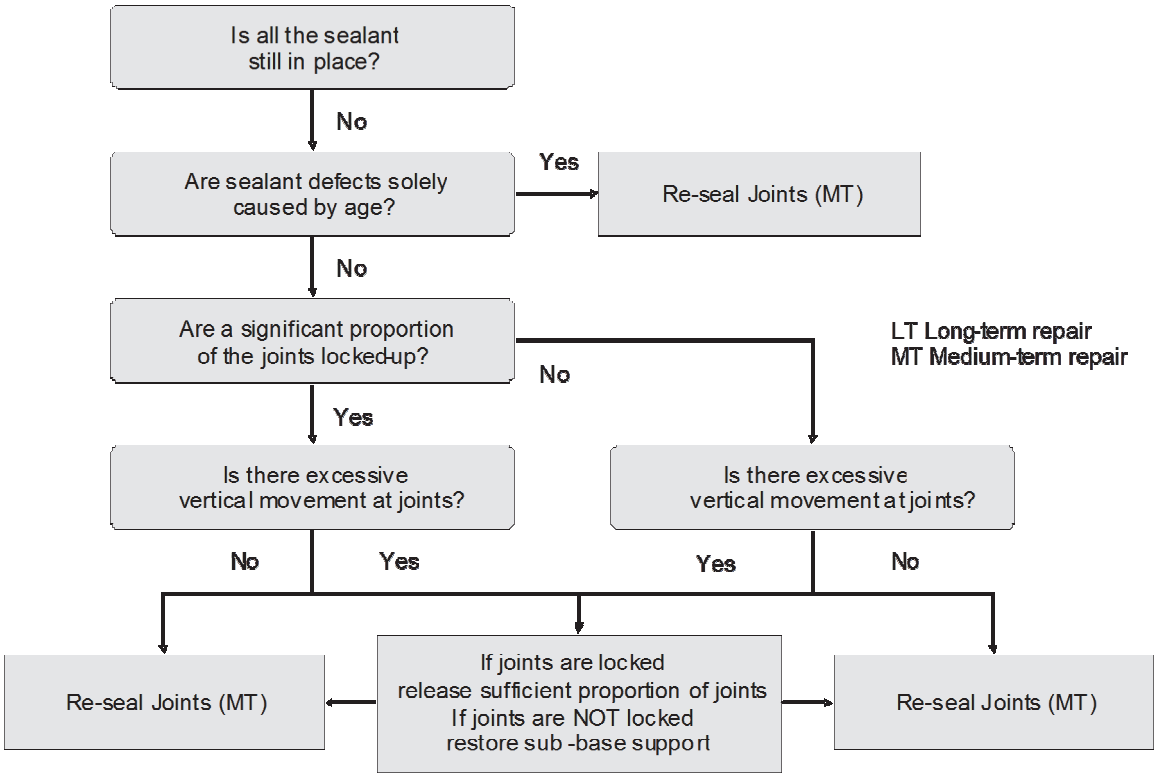


Chart 1 Defective joint seals

13.1.2 *Sealant in longitudinal joints*

The service performance of sealants in longitudinal joints is strongly influenced by the method used to install the joints during construction. The two common methods are:

i) In machine-laid concrete the sealant often takes the form of a continuous strip inserted through a hollow vibrating plough.

ii) Where the carriageway is too wide to be laid in one pass of the paving machine, the sealant is usually glued to the vertical face of the previously laid slab before the second pass of the paving equipment.

Longitudinal joint sealant correctly installed by the first method will often perform satisfactorily throughout the life of the pavement. It acts as a crack inducer, either alone or in conjunction with some form of additional crack inducer at the base of the slab. The total width of practically all carriageways is such that longitudinal joints act as warping joints and opening and closing of the joints is small. The steel tie bars are well bonded to the concrete, and aggregate interlock is created at the induced longitudinal fracture of the slab. The result is that the joint usually remains sealed and the tie bars therefore suffer little corrosion. Because the tie bars retain their effectiveness, aggregate interlock is maintained along with the load transfer efficiency of the longitudinal joint.

The long-term performance of joints constructed by the second method is often less satisfactory. With this form of construction, the second strip of concrete is likely to have been placed some weeks after the first. Tie bars, bent through a right-angle at the joint so that they could be installed within fixed forms are used for placing the strip. The cranked ends of the bars then have to be re-bent to form tie bars that are only approximately straight. In contrast to the joints formed by first method, these joints may have the following characteristics:

i) The 'kink' in the tie bars caused by bending and re-bending permits some initial opening of the joint under tension as the `kink' straightens.

ii) Some early tension across the joint is almost inevitable as the second strip of concrete contracts on cooling from its hydration temperature.

iii) Because the slab placed first has a vertical formed face, there is little aggregate interlock; load transfer across the joint is therefore provided solely by the shearing resistance of the tie bars.

13.2 Shallow spalling joints

Shallow spalling at a joint is defined as spalling which does not extend below the depth of the joint groove. The main causes of shallow spalling are the infiltration of incompressible materials into the joint groove or weak concrete at the joint. Chart 2 summarises the approach to repair.

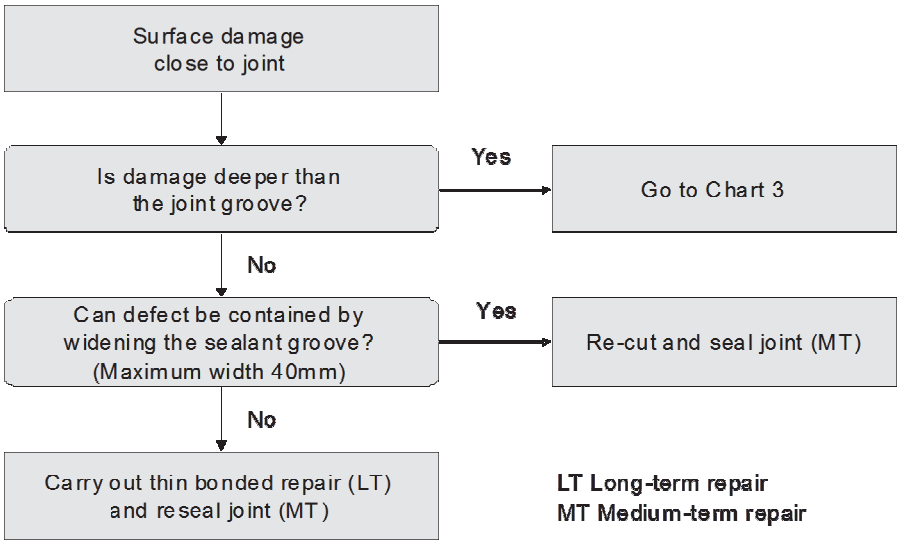


Chart 2 Shallow spalling at joints

Shallow spalling due to the ingress of incompressible material (often pebbles) into the joint groove is usually sudden and often in the form of wedge-shaped concrete fragments. Typically these taper towards the sides of the spall and towards the edge remote from the joint face.

If the original sealing groove was wet-formed, it is possible that some of the concrete around the joint former was weak and contained an excess of fine aggregate and/or that this material was inadequately compacted. It is essential to check that the concrete below the spall is sound. Tapping with a light hammer or, preferably, a Schmidt hammer will indicate whether the substrate concrete is sound or has a hollow ring when struck. Spalling where neither the depth of the original spall nor the depth of the underlying unsound concrete exceeds the depth of the sealing groove should be recorded as shallow spalling.

Minor spalling may be treated by vertical sawing to form a widened sealing groove to remove the spalled edge up to a maximum width, including any chamfers of 40 mm for transverse joints and 25 mm for longitudinal joints. It is normal to continue this treatment for the entire length of the joint. If spalling cannot be corrected within these limits, thin-bonded repairs will be needed at significantly greater cost. It is therefore important to establish the extent to which the spalling falls within or outside these limits.

13.3 Partial depth and deep spalling joints.

Partial-depth spalls are those in which the depth of spalling exceeds the depth of the sealing groove but does not exceed one-third of the slab depth. The causes are usually the same as for shallow spalling but the defect is deeper and often the consequence of inadequate attention to joint re-sealing. The repair cost is significantly greater than for shallow spalling and so it is important to distinguish between the two. Chart 3 summarises the diagnosis.

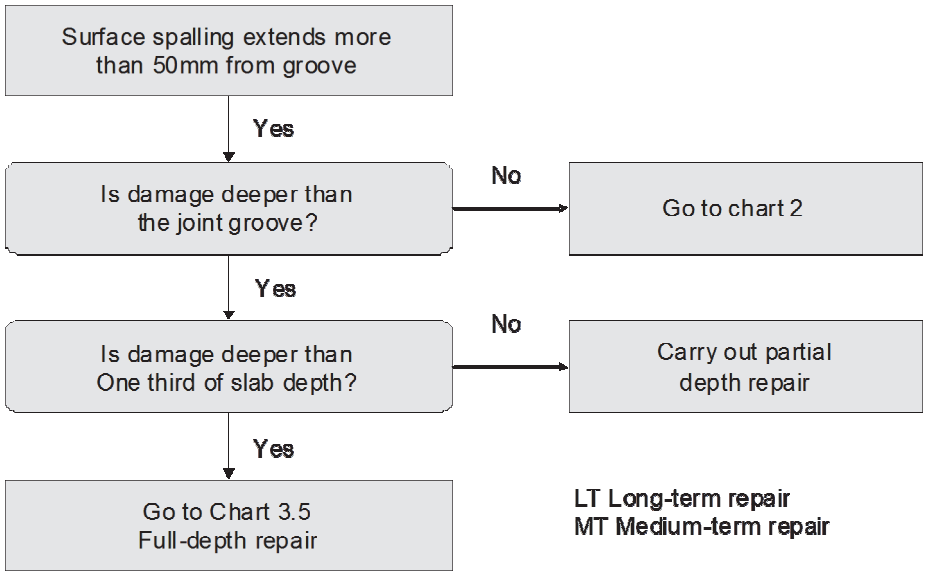


Chart 3 Deep spalling at joints

Deep spalls are deeper than one-third of the slab depth. The depth must be established because a partial-depth repair may be possible if the depth of the spall does not exceed one-third of the slab depth. A full-depth repair is essential for deeper spalling.

Some causes of deep spalling include:

i) dowel bar restraint
ii) ingress of solid particles into the joint crack
iii) loss of sub-base support resulting in excessive load transfer stresses in the concrete around dowel bars.

Dowel bar restraint may be caused by misalignment of the bars or by excessive bond between the bars and the concrete around what was intended to be the debonded length. Such bonding may result from the use of bars that are bent or have excessively burred ends. Alternatively, the dowel bars may become corroded over several years, causing seizure of what was previously a freely moving dowel bar. Dowel bar restraint from any of these causes may result in deep spalling at contraction and expansion joints owing to the high stress developed in the concrete by the restraint.

If the concrete slabs are not connected by dowel bars, deep spalling will usually not occur if the preceding shallow spalling is adequately maintained. However, if the defect is extensive, it is likely to require a prompt full-depth repair.

13.4 Faulting at joints

'Faulting' in the form of permanent relative vertical movement at joints and wide cracks can occur in slabs where there is no effective load transfer in the form of dowels or tie bars at joints. Vertical movement occurs either dynamically under passing traffic or permanently in the form of settlement of the slab or 'faulting' at joints.

Dynamic movements may be associated with mud-pumping which, unless remedied, is likely to lead to multiple cracks. Mud-pumping may also indicate poor pavement or sub-soil drainage; this should be corrected before any remedial work is undertaken. Seepage of water up through joints or along the edges of the slab may also indicate poor drainage.

Dynamic movement may be measured as deflections of the slab at joints using the Falling Weight Deflectometer (Appendix D). In each case, high absolute or relative deflection across joints or cracks indicates poor support and possible voiding (see Chapter 15).

Settlement is normally a result of consolidation or compaction of the fill material in embankments, particularly in the back-fill behind structures or when the pavement is constructed on ground with a low bearing capacity.

Diagnosis and remedial treatments are summarised in Chart 4.

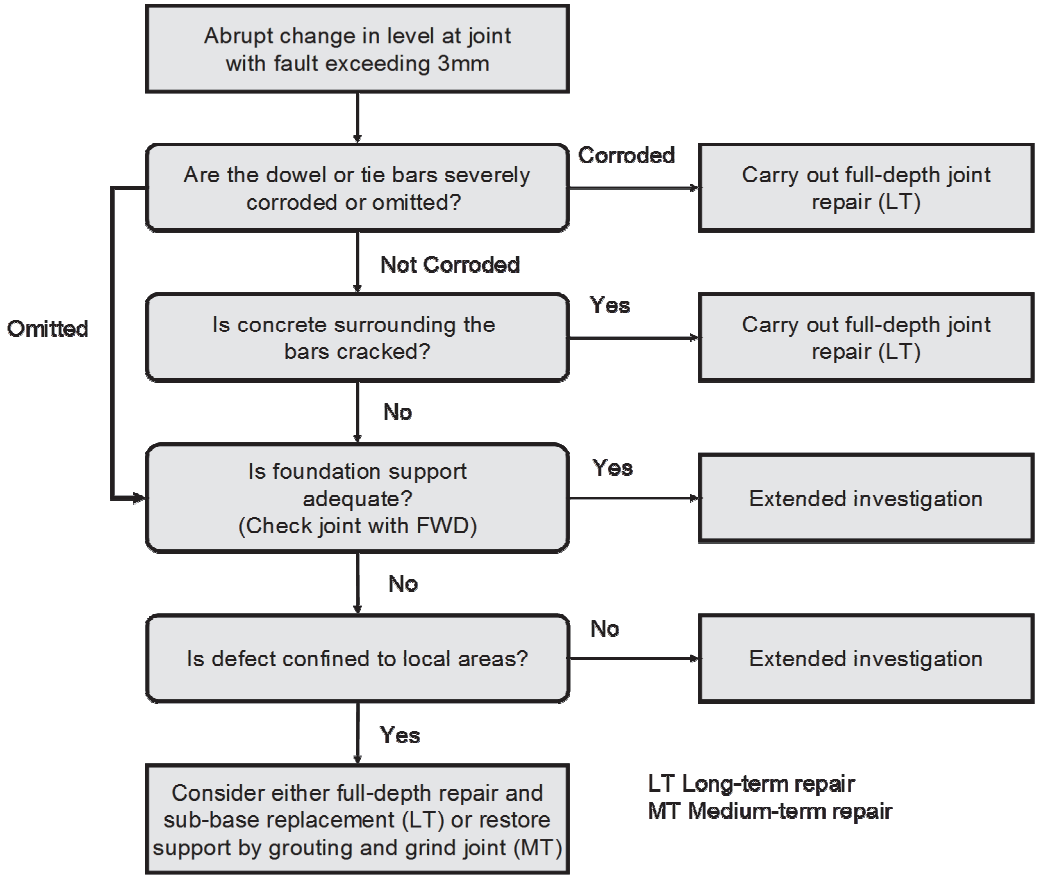


Chart 4 Faulting at joints

13.5 Cracks at transverse joints

13.5.1 *Causes of cracks at transverse joint*

Contraction joints are usually formed by creating a plane of weakness during construction such that the early thermal contraction and drying shrinkage causes a crack at the desired contraction joint position. A crack that extends across the full width of a bay near a transverse contraction joint may be caused by any of the following mechanisms:

i) Contraction joints are generally formed by creating a groove in the top of the slab soon after construction. If this operation is delayed then the crack can occur close to the groove but not directly underneath it. Alternatively the contraction joint can be created by combining a groove in the top surface with a former fixed to and projecting upwards from the sub-base. If, in this case, the top groove and the bottom crack inducer are not vertically aligned, the crack may form over the bottom former and fail to connect with the top groove.

ii) A wheel load applied to the corner or edge of an individual slab causes a tensile stress at least double that induced by the same wheel load applied to the slab interior. The use of dowel bars as load-transfer devices reduces the disproportionate effect of edge loading near transverse joints. For a properly constructed dowelled joint, load-transfer efficiency immediately after construction is typically 80-90% and normally more than 75%: i.e. the distribution of load from the loaded slab to the adjacent slab is at least 75% of that which would occur if the joint were omitted and the slab continuous.

iii) Immediately after construction, the sub-base support to the slab at the transverse joint locations should be equivalent to the support provided to the remainder of the slab. However, if the joint sealant later becomes ineffective and surface water is allowed to penetrate through the joint to the sub-base, this uniformity is unlikely to be maintained as the sub-base near the joint is softened by water penetration. Unbound sub-bases are more likely than bound material to permit significant water penetration. In addition, the unbound granular material may suffer a volume loss as fine material is flushed out, causing a further reduction in slab support. In such conditions, the portion of slab around the joint may be visualised as a beam with its ends built in and a hinge at its midpoint. The tendency for bending tensile cracks to form within the slab and roughly parallel to the joint may then be readily appreciated.

iv) When a pavement is constructed as two or more parallel strips separated by longitudinal joints, it is important that the transverse joints are not staggered but are aligned as a single joint passing through all the strips. Failure to do this will almost inevitably result in `sympathetic cracking' of the bay containing the misaligned part of the joint. Also, where previous remedial work has included full-depth replacement in a bay or bay end but without corresponding joints in the adjacent lane or lanes, sympathetic cracking is likely. It is important to diagnose this as the cause and not to attribute the further cracking to some other effect such as sub-base or foundation failure.

In view of the differing causes of transverse cracks, correct diagnosis is vital. At one extreme, the cause might be a solitary construction defect that may be treated in isolation. At the other extreme, the crack could indicate a developing widespread condition that could severely reduce pavement life. The following sections suggest some appropriate diagnostic considerations.

13.5.2 *Identifying the causes of cracking*

The diagnostic process is shown in Chart 5. Identifying the common causes of cracking described above is as follows:

1 Misalignment between top and bottom crack inducers

First, it is necessary to check whether the original construction procedure included the use of bottom crack inducers. If not, then this cause is eliminated.

If they were used, and the affected slab adjoins the verge or central reservation, exposing the slab edge will show whether an incorrectly located bottom crack inducer is the cause. If it is not practical to inspect the slab edge, an exploratory core hole must be drilled. Such errant cracks are seldom vertical and a core diameter of 150 mm or 200 mm is needed to make reasonably certain that the complete path of the crack can be inspected both in the extracted core and in the core hole.

2 Reduced load-transfer efficiency

When a heavy vehicle passes, if there are either visible or audible indications of relative vertical movement between the slabs each side of a joint, then the load transfer is inevitably severely reduced. In these circumstances it is very likely that full-depth replacement will be the only feasible remedy. Provided this condition is detected when only a few joints are affected, early attention should be focussed on the remaining joints within the same construction contract. An FWD or Benkelman beam deflection survey will be needed to measure Load Transfer Efficiency (LTE) values. Deflection measurements are described in Appendix D and criteria are described in Chapter 15.

3 Inadequate sub-base support

The FWD or Benkelman beam can be used to measure the maximum vertical deflections at the middle of each slab. The level of support can be evaluated by considering the extent of longitudinal cracking and the mid-slab deflection.

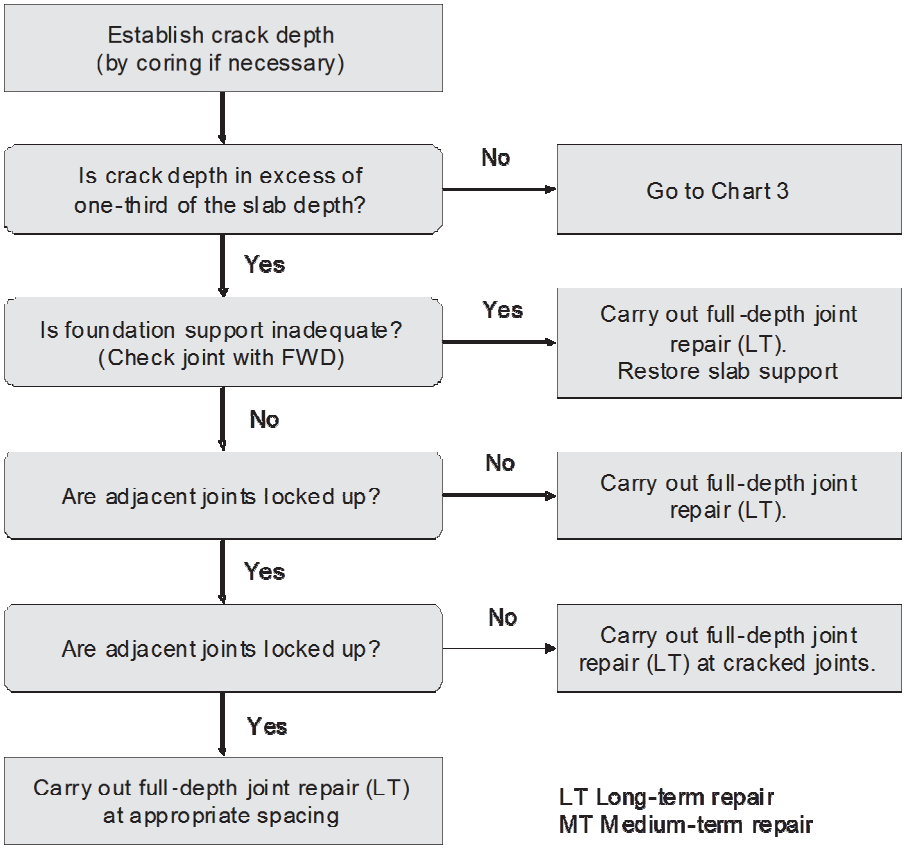


Chart 5 Deep cracking at joints

13.6 Longitudinal cracks

Possible causes of longitudinal cracks include:

i) Poor sub-base support near slab edges or joints, indicated by the results of FWD tests. This is often caused by rain water penetrating to an unbound sub-base or to the formation. It is less likely with a bound sub-base.

ii) Transverse tension induced by the cumulative effects of traffic. This may result in longitudinal cracking along the nearside wheel path, particularly in the absence of a tied shoulder. If this occurs throughout the pavement, it most probably indicates structural failure in a pavement nearing the end of its useful life.

Consideration of the results of FWD tests (Appendix D) will usually indicate which of these possible causes is likely to be responsible. For the first, examination of samples of sub-base and formation material, followed by moisture content tests, will show whether the water content is higher beneath the slab edge than elsewhere. If so, this is the likely cause of the reduction in slab support. Chart 6 indicates the diagnostic procedure and suitable repair options.

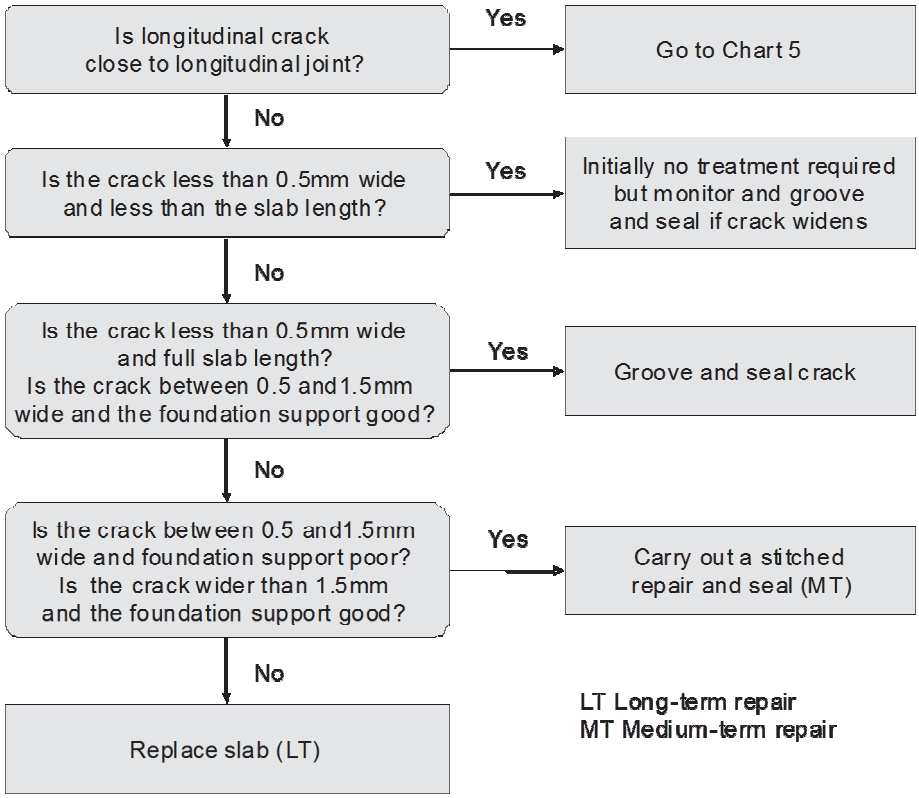


Chart 6 Longitudinal cracks

13.7 Transverse cracks (mid slab)

Transverse cracks remote from the joints may be caused by specific localised problems or may indicate the onset of terminal failure in a pavement nearing the end of its useful life. It is vital to distinguish clearly between these two possibilities before considering options for remedial works.

Some specific local causes that may account for mid-slab cracking are:

i) A location where support conditions change because of a changes in the foundation.

ii) A location where the depth of fill material changes significantly - for example, because of a soft area encountered during construction or on the approach to an under bridge.

iii) A position in the pavement where thermal contraction has been constrained by locked-up joints, frictional restraint or other local effects.

iv) Sympathetic cracking caused by a joint formed in the slab of an adjacent lane that does not correspond in position with a joint in the cracked slab.

If these causes can be eliminated then the possibility that the pavement is nearing the end of its useful life must be considered. Chart 7 shows the logical diagnostic path.

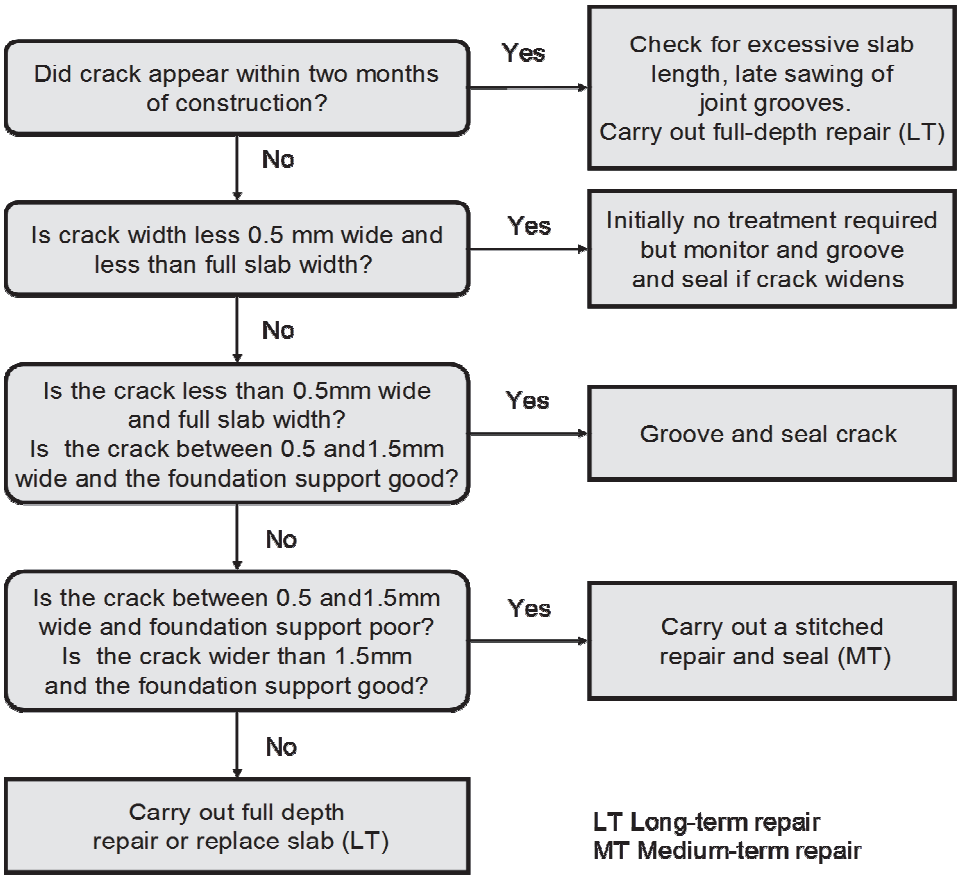


Chart 7 Transverse cracks (mid-slab)

13.8 Diagonal cracks and corner cracks

The term diagonal cracks is intended to include all multi-directional full-depth cracks that are neither generally transverse, longitudinal, nor across bay corners. Corner cracks include single full-depth cracks about 0.3-2m long across the bay corners; if not repaired they will lead to localised deterioration of the sub-base and perhaps subsequent mud pumping.

The most likely cause of diagonal cracking is settlement or heave of the sub-base or subgrade.

Corner cracking can be caused by:

i) Lack of load transfer at joints.
ii) Dowel bar restraint at edge of slab.
iii) Incompressible material in the joint at edge of slab.
iv) Acute angles in non-rectangular slabs.
v) Loss of sub-base support.

Charts 8 and 9 indicate the evaluation process to identify the optimum and repair strategy.

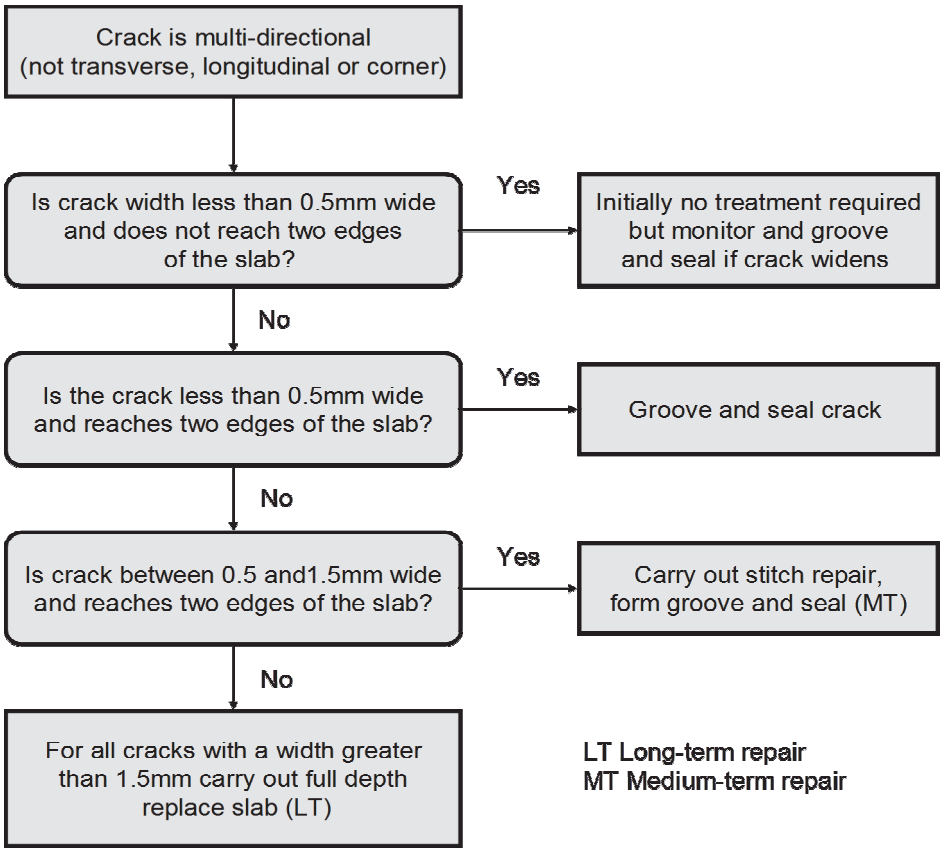


Chart 8 Diagonal cracks

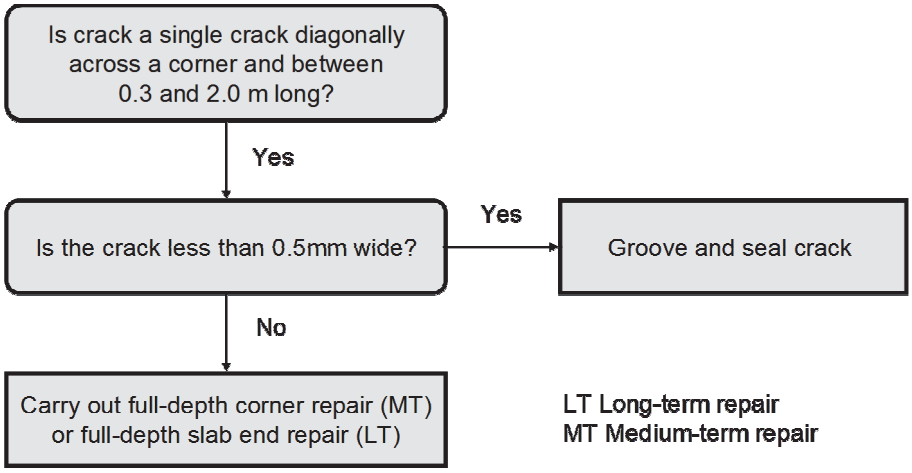


Chart 9 Corner cracks

13.9 Plastic cracking

Plastic cracks are surface cracks that form before the concrete has hardened. They form a pattern of short cracks usually approximately parallel to each other, oriented diagonally to the slab sides and not extending to the edges of the slab. Plastic cracks are caused by the concrete mix being too wet and they appear as the concrete cures. Although these cracks appear as minor defects at construction, they will propagate downwards through the

concrete over time and affect the life of the pavement. If noticed soon after construction, the slab should be replaced. Chart 10 shows the diagnostic process.

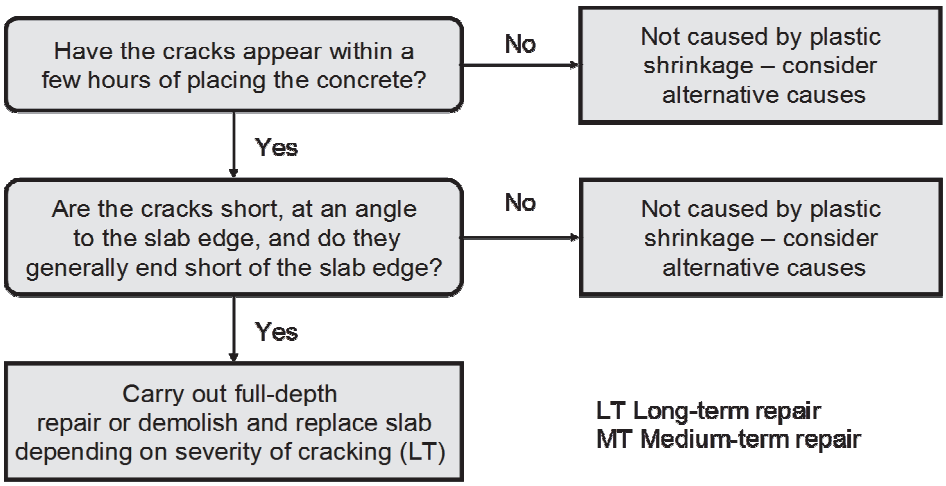


Chart 10 Plastic cracking

14. MAINTENANCE TECHNIQUES AND METHODS

14.1 Techniques at joints

In selecting appropriate remedial treatments for defects at joints, the various functions they are required to perform in a concrete pavement must be considered.

As air temperature rises or falls, concrete slabs expand or contract. A temperature variation also occurs within the body of the slab between the surface exposed to radiant direct heat and the underside where the temperature is raised only by conducted heat. This temperature differential leads to cyclic warping or curling of a slab as the temperature gradient reverses from day to night. Expansion or contraction can also occur as concrete takes up or loses moisture. Joints are provided in concrete pavements to allow these movements, thereby preventing excessive stresses and consequent cracking.

Three different types of movement joint are used; contraction, expansion and warping joints. All three types permit warping movement (i.e. rotation).

Contraction joints enable the slab to contract when its temperature falls and permit subsequent expansion by the same amount provided the joint space has not been filled by detritus or hard particles. Expansion joints allow the slab to shorten and also cater for the expansion that would naturally occur at temperatures higher than that of the concrete when the slab was laid. Longitudinal joints are usually of the warping type only i.e. they tie the slabs together and can be thought of as acting as 'hinges'.

14.1.1 *Failed joint seal (Chart 1)*

Most joint seals do not last as long as the concrete pavement because they tend to harden and become brittle. Seals must therefore be replaced regularly, and Table 14.1 gives a guide to the main types, their relative life and usage. The sealants would generally be expected to last 7-10 years.

Table 14-1 Main types of joint-sealing materials

Classification	Chemical	Approximate Life
Hot –applied	PVC/pitch polymer	Medium
	Polymer/bitumen	Medium
Cold-applied	Polysulphide	Medium
	Polyurethane	Medium
	Silicone	Medium
Compression	Polychloroprene	Longest

Either hot or cold-applied elastomeric materials or compression seals are suitable for general re-sealing but gun-grade cold-applied materials are probably the most appropriate for small quantities of material. At joints between concrete and bituminous pavements, only hot-applied polymer modified bituminous sealants, or pre-formed polymer-modified bituminous strips are suitable.

Joint seals may suffer from three common forms of defect. The most common is adhesion failure where the bond between the sealant and the sides of the sealing groove is lost. Cohesion failure may also occur, with the sealant splitting or tearing because it cannot accommodate the full range of movement. Extrusion is another type of defect that occurs when the sealant is pushed out of the sealing groove by compression forces.

For the seal to function properly it must adhere to the sides of the sealing groove. This calls for an appropriate primer, with the sides of the sealing groove scoured by abrasive blasting, clean, dry and not too cold at the time of application. Caution is required with spray-applied primers at high ambient temperatures because they can vaporise before adhering to the concrete. Inadequate primer adhesion to the joint faces is a common cause of premature failure of joint seals. As some sealants are mutually incompatible, all the old sealant should be removed before a new one is applied.

14.1.2 *Shallow spalling (Chart 2)*

Removing the old joint seal will reveal the extent of shallow spalling. The quality of the concrete can be confirmed by tapping with a steel rod - a hollow sound indicating cracked material and a ringing tone indicating intact concrete. Where possible, such repairs should be carried out using either cement mortar or fine concrete depending upon the depth; the practicable minimum is about 10 mm. Cement mortar should be used for repairs up to 20 mm deep and fine concrete for deeper repairs. Thin bonded repairs at joints must not be used for depths greater than the joint groove.

Using epoxy concrete, or other ‘concretes’ with thermal properties and strengths different from the existing concrete, is not recommended since further debonding or cracking of the existing concrete often follows. However, they can be used with care on small repairs less than 1 m long and less than 30 mm deep when there is insufficient time for cement mortar or fine concrete to cure.

Thin-bonded repairs to slab surfaces and joint grooves should be carried out in accordance with the procedure in Figure 14.1. Thorough preparation, attention to detail and good workmanship are essential. As sawing produces a polished surface that inhibits good bond, a groove should be chased out to provide a roughened vertical edge around the repair, against which the repair material can be properly bonded. Sawn grooves also tend to penetrate into the slab beyond the limits of the repair. However, a shallow delineating groove may be sawn to start with and subsequently chased out to the full depth. The success of thin-bonded joint repairs depends entirely upon a good bond. This is best achieved by compacting the repair material against a freshly scrabbled (roughened), clean surface and finishing it flush with the existing slab surface. The repair must not bridge the joint and care is required that the repair material does not find a route around the temporary joint.

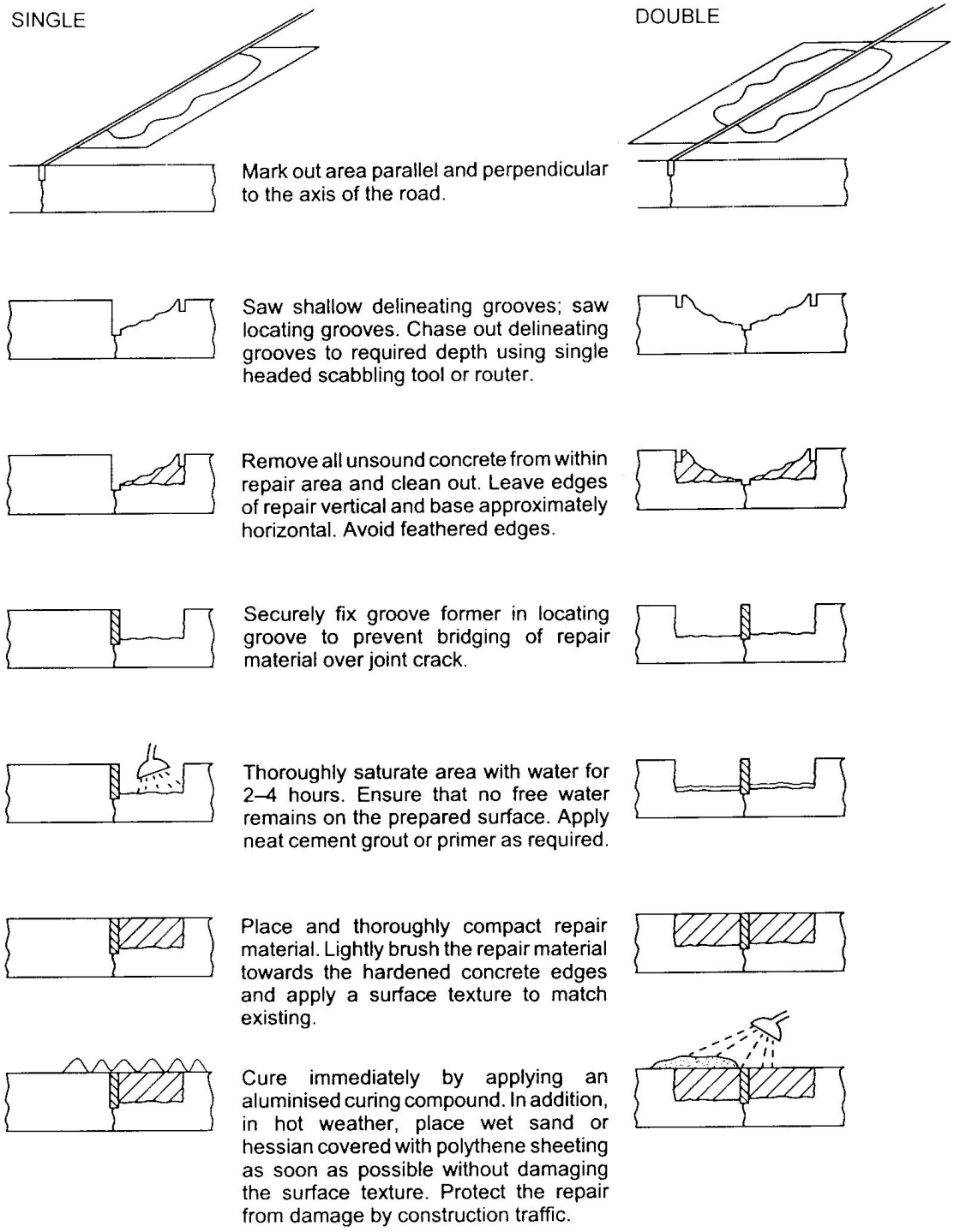


Figure 14-1 Procedure for thin bonded repair at joint

14.1.3 *Deep spalling (Charts 3 and 5)*

Deep spalling usually extends to at least half slab depth. Possible causes are summarised in Table 14.2. One cause, dowel bar restraint, may be due to misalignment and/or excessive bond along the bar (which must be free to move in one of the slabs). The only satisfactory remedy likely to achieve a long life is a full-depth bay-end replacement (Section 14.2.3).

Table 14-2 Deep spalling - causes and remedies

Type of defect	Cause	Remedy
Deep spalling at contraction and expansion joints	Dowel restraint Ingress of solids into the joint crack	Transverse full-depth repair
Deep spalling at bay corners	Dowel restraint Ingress of solids into the joint crack	Transverse full-depth repair

14.1.4 *Other structural cracks at joints (Chart 5)*

The main types of crack, their likely causes and appropriate remedies are summarised in Table 14.3.

Table 14-3 Structural cracks at joints: causes and remedies

Type of defect	Cause	Remedy
Transverse or diagonal cracks at transverse joint	Dowel restraint, gross misalignment Late sawing of joint groove Misaligned top and bottom crack inducers	Transverse full depth repair
Longitudinal crack at transverse joint	Compression failure Ingress of incompressible material into joint crack Edge restraint	Transverse or longitudinal full depth repair
Longitudinal cracks at longitudinal joints	Misaligned top and bottom crack inducers Omission of bottom crack inducer	Longitudinal full depth repair

14.2 Longitudinal and transverse cracks (Charts 6, 7, 8, 9)

Structural cracks are classified in terms of the unspalled width of the crack at the slab surface as indicated in Table 14.4.

Table 14-4 Classification of crack width

Crack definition	Width (mm)	Condition
Narrow	< 0.5	Full aggregate interlock and load transfer
Medium	0.5 – 1.5	Partial interlock and load transfer. May permit entry of water
Wide	> 1.5	No load transfer. Ingress of water and fine material

The most likely cause and appropriate treatment for structurally significant longitudinal and transverse cracks is shown in Table 14.5.

No cracks of any type are expected between the joints in unreinforced slabs. Although narrow transverse cracks may not need immediate treatment, they are likely to widen fairly quickly. At the very least, they should be inspected regularly. Medium and wide cracks in unreinforced slabs require sealing, a stitch repair, full-depth repair or slab replacement depending on their width and extent (see Charts 6 and 7).

Table 14-5 Longitudinal and transverse cracks: causes and remedies

Type of Defect	Cause	Remedy
Transverse Crack	Excessive bay length Late sawing of joint grooves Dowel bar restraint at joints Inadequate reinforcement overlap Sub-base restraint	Full slab width narrow cracks and medium cracks with good support – groove and seal Full slab medium width with poor support and wide cracks with good support – stitch repair. Full slab width wide cracks with poor support – full depth repair or slab replacement
Longitudinal Crack	Excessively wide bays Omission or displacement of bottom crack inducer at longitudinal crack Settlement	Full slab width narrow cracks and medium cracks with good support – groove and seal Full slab medium width with poor support and wide cracks with good support – stitch repair. Full slab width wide cracks with poor support – full depth repair or slab replacement

14.2.1 *Stitched crack repairs*

It is possible to repair cracks using a stitched crack repair. The two types - Type 1, a staple tie bar repair and Type 2, a diagonal tie bar repair, are shown in Figure 14.2. Such repairs are undertaken to convert the crack into a tied warping joint that will allow the slab to 'hinge' at that point, maintaining aggregate interlock and preventing it from widening.

For Type 1, slots 25-30 mm wide by 470 mm long at 600 mm centres and at right angles to the line of the crack are chased out to a depth such that the tie bars lie between one-third and one-half of the slab depth below the surface when bedded. Holes of 25-30 mm diameter and 50 mm deep are drilled at each end of the slot and the slots then cleaned with oil-free compressed air.

Once dry, the slots are primed and the staple tie bars placed onto beds of epoxy resin mortar and covered with the same material to a minimum depth of 30 mm. The sides of the slots are then cleaned of loose material and filled with thoroughly compacted resin or cementitious mortar. After the repair material has cured, a groove is sawn or routed along the line of the crack and sealed in the same manner as a transverse joint.

With Type 2 crack repairs, cross-stitching is employed as shown in the lower diagram of Figure 14.2 and about 25 degrees to the slab surface. These holes are spaced every 600 mm along the crack with alternate entry points on opposite sides. The entry points should be at a distance from the crack equal to the slab depth.

The length of bar depends on the slab depth and should be enough to allow 50 mm cover at the bottom of the slab. Deformed 12 mm grade 460 steel tie bars are used and notched at a point that will be 50 mm below the slab surface when the bars are fully inserted.

Each hole is filled with epoxy resin mortar so that, with the tie bar inserted, the mortar level reaches to 25 mm below the notch. Epoxy resin mortar is recommended for stitch repairs because the material must harden before crack movement can disrupt the repair. Once the mortar has set, the length of tie bar above the notch is broken off by twisting. Any bars that rotate after the mortar has hardened must be withdrawn and the hole re-drilled.

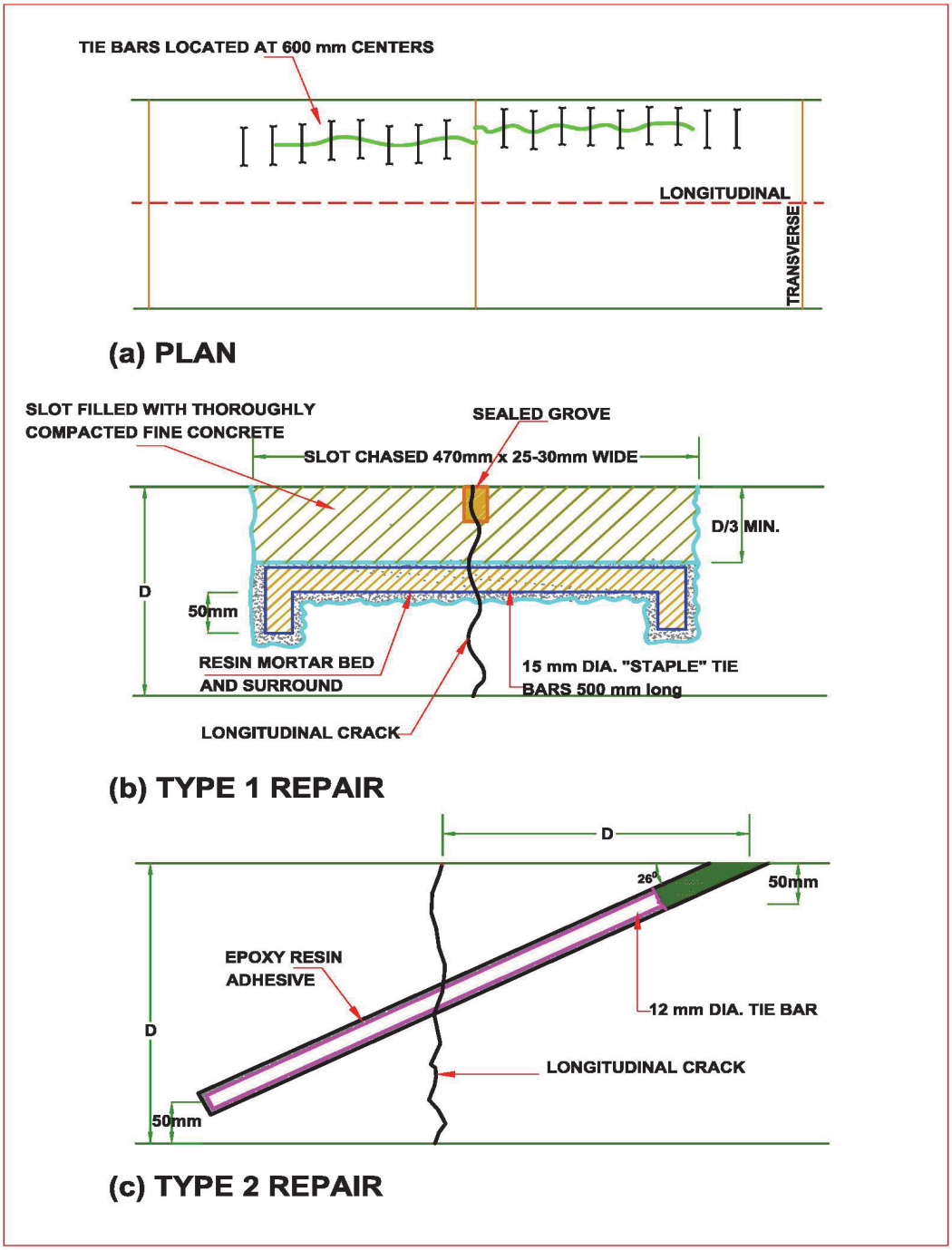


Figure 14-2 Stitched crack repair

14.2.2 *Slab replacement*

Before an affected slab is broken out, a full-depth saw-cut should be made around the perimeter of the repair to minimise damage to the surrounding slab. This should include the existing transverse and longitudinal joints, but saw cuts *must not* extend into adjacent bays. The concrete may then be sawn into smaller pieces before being broken up and removed. Concrete left in the corners of the repair after cutting must be broken out carefully to avoid undercutting the remaining slab.

Any necessary reinstatement of the sub-base should be done before new dowel and tie bars are fixed at the transverse and longitudinal joints. Any new sub-base material must be fully compacted, especially at corners; a heavy plate vibrator is required to compact granular or cement-bound sub-base material.

An existing cement-bound sub-base may be reinstated and regulated with sand/cement mortar, fine concrete or fine cold asphalt. If a granular sub-base is to be replaced with cement-bound material, action should be taken to avoid any potentially deleterious effect from the creation of a discontinuity in the under-slab drainage or to the slab support.

To avoid surface water ponding in the repair before concrete is placed, it should either be kept out by means of sandbags or provision should be made for it to drain away.

14.2.3 *Full-depth repairs*

Where there is an unbound sub-base, the length of a full-depth repair should be at least 2m so that sub-base compaction can be effective and the traffic load spread over a greater length, eliminating the punching effect on a shorter repair. Therefore longitudinal or mid-slab transverse cracks necessitate a slab replacement.

Where undertaken, full-width repairs constitute small slabs and so should be at least equivalent to the main slab in all respects (see Figures 14.3 and 14.4). It is advisable to reinforce such repairs and this *must* be done when the ratio of the longest to the shortest dimension is greater than two (as is usually the case). Either a square or long mesh reinforcement of appropriate weight is suitable. For the latter, the main bars must be parallel to the longest dimension. The quantity of reinforcement should be 500-800 mm^2 per metre width of concrete, depending on the traffic (see Figure 14.4).

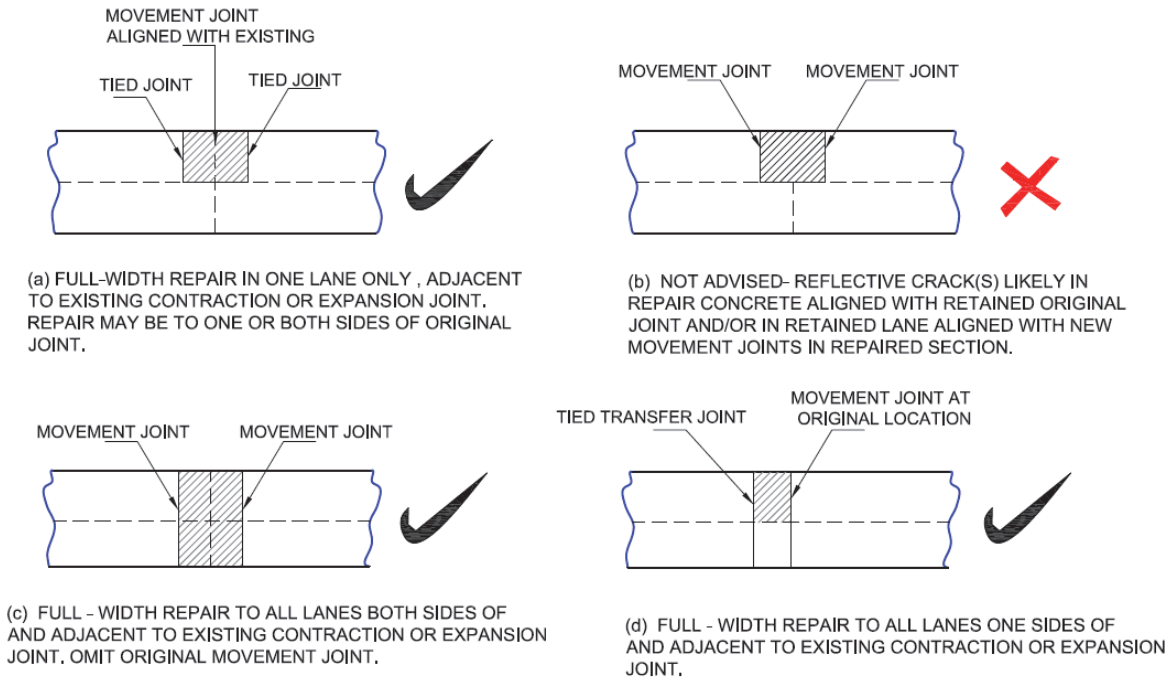
The function of any joints between new and existing concrete that may be introduced as a consequence of a full-depth repair must be considered. Failure to do so has frequently resulted in sympathetic cracking in the repair concrete or in the retained adjacent original concrete. Joint arrangements that have proved satisfactory are shown in Figure 14.3. Movement joints in this context are either transverse contraction joints or transverse expansion joints. New joints will usually be contraction joints unless the retained pavement adjoining the repair has been found to contain locked-up joints, in which case it may be appropriate to introduce an expansion joint in a full-depth repair as long as it spans the full carriageway width.

A tied transverse joint functions differently from a tied longitudinal joint. Tied transverse joints between an original slab and a full-depth repair are intended to give as near as possible a monolithic action between the repair concrete and the original slab. Because load transfer is likely to be impaired by the tied joint, shear transfer should be improved by

transverse tie bars of the same length and diameter as would be appropriate for dowel bars at a contraction joint. In this case the bars should be of high-yield deformed steel to take advantage of the increased bond available compared with plain mild-steel bars and the polymeric coating required for dowel bars in movement joints should be omitted. A detail suitable for tied transverse joints is shown on the left of Figure 14.4.

The movement joint on the right of the same figure is an expansion joint for which the dowel bars should be of plain round mild steel 25 mm in diameter for slabs less than 240 mm thick and 32 mm for slabs 240 mm or more. The detail for contraction joints is similar with the omission of the expansion filler board and dowel caps. For contraction joints, the bars are similar but the diameters may be reduced to 20 mm and 25 mm respectively.

With all full-depth repairs every effort should be made to prevent debris, such as slurry from sawing or other repair material, from entering any joint. Cracks and grooves should be cleaned using oil-free compressed air if necessary and taped over with adhesive masking tape. It is also essential to prevent slurry from the sawing operation - which could solidify and block drains - getting into the drainage system.



(a) Full-width repair in one lane only, adjacent to existing contraction or expansion joint. Repair may be to one or both sides of original joint.

(b) Not advised – reflective crack(s) likely in repair concrete aligning with retained original joint and/or in retained lane aligned with new movement joints in repaired section.

(c) Full-width repair to all lanes both sides of and adjacent to existing contraction or expansion joint. Omit original movement joint.

(d) Full-width repair to all lanes one side of and adjacent to existing contraction or expansion joint.

Figure 14-3 Full-depth repairs to unreinforced concrete pavements

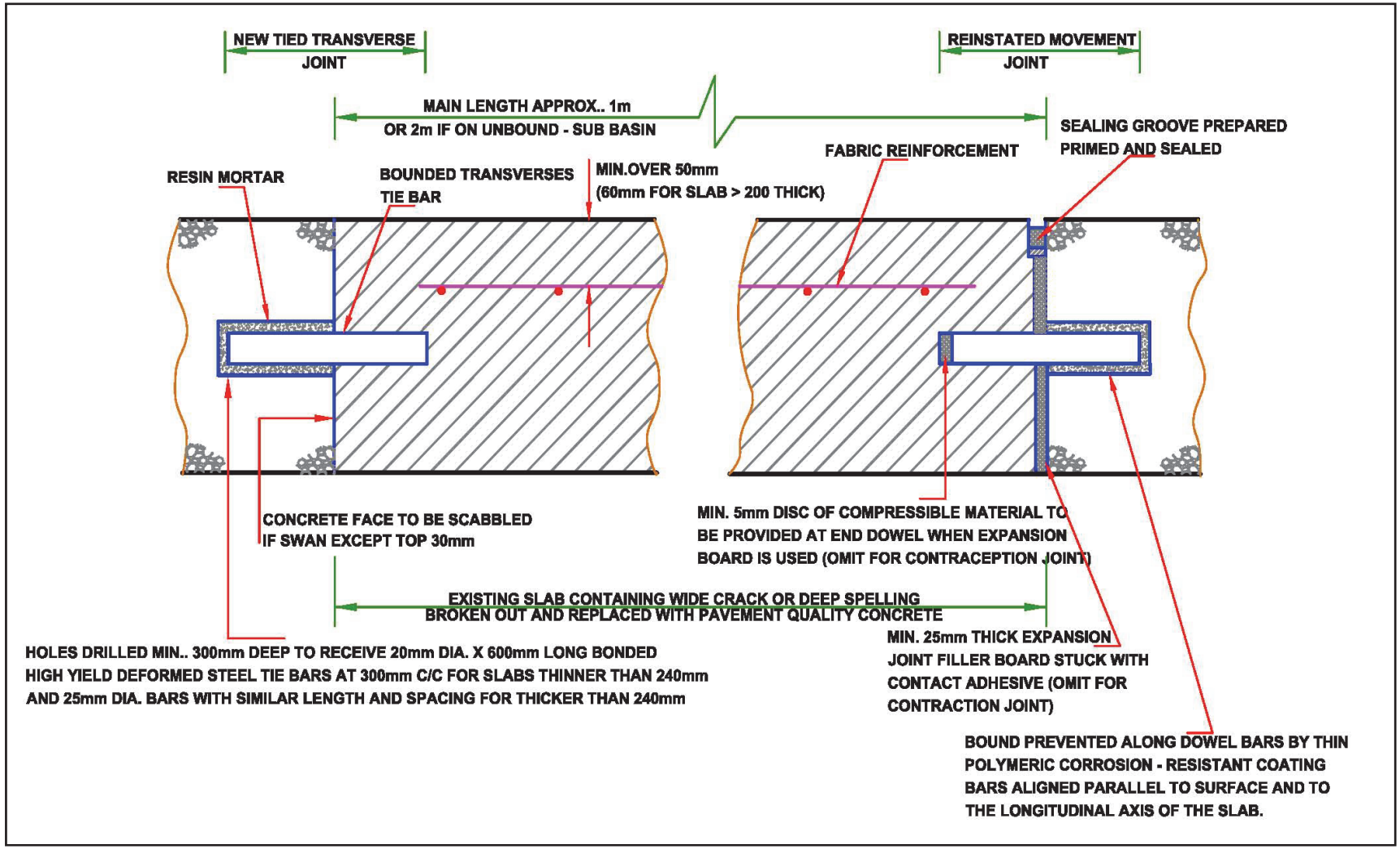


Figure 14-4 Longitudinal section through transverse full-depth repair adjacent to an existing transverse movement joint

14.3 Diagonal and corner cracks (Charts 8 and 9)

The term 'diagonal cracks' includes all multi-directional full-depth cracks that are neither generally transverse, nor longitudinal, nor across bay corners. Corner cracks include single full-depth cracks about 0.3-2m long across the bay corners. If they are not repaired they will lead to localised deterioration of the sub-base and perhaps subsequent mud pumping. The most likely causes and appropriate remedies are given in Table 14.6.

Table 14-6 Diagonal and corner cracks: causes and remedies

Type of defect	Cause	Remedy
Diagonal cracks	Settlement or heave of sub-base or subgrade	Narrow cracks will need to be sealed or remedied by means of a stitched crack repair
Corner cracks	Lack of load transfer at joints Dowel bar restraint at edge of slab Ingress of solids into joint at edge of slab Acute angles in non-rectangular slabs Loss of sub-base support	Transverse full depth repair or corner repair. The latter may not achieve long life.

If a full-width repair is inappropriate, a corner repair may be carried out. However, experience in the UK has shown that, although sometimes successful, corner repairs cannot be relied upon to achieve the life expected for a long-term repair.

14.4 Inadequate slab support

Vertical movement occurs either dynamically under passing traffic or permanently in the form of settlement of the slab or faulting at joints or cracks. Dynamic movements may be associated with mud-pumping which, unless remedied, is likely to lead to multiple cracks. Mud-pumping may also indicate poor pavement or sub-soil drainage; this should be corrected before any remedial work is undertaken. Seepage of water up through joints or along the edges of the slab may also indicate poor drainage.

Dynamic movement may be measured as deflections of the slab at joints or cracks under a static or dynamic load. A dynamic load may be applied by the Falling Weight Deflectometer (Appendix D). In each case, high absolute deflection or relative deflection across joints or cracks indicates poor support and possible voiding.

Settlement is most likely as a result of consolidation or compaction of the fill material in embankments, particularly in the back-fill behind structures or when the pavement is constructed on ground with a low bearing capacity.

'Faulting' in the form of permanent relative vertical movement at joints and wide cracks can occur in slabs where there is no effective load transfer in the form of dowel or tie bars at joints.

These defects, their likely causes and appropriate treatment are described in Table 14.7. Note that the remedy for the immediate problem may not remove the original cause, e.g.

ground softening due to water ingress. The cause must be understood and corrected before carrying out repairs.

Table 14-7 Inadequate slab support

Type of defect	Cause	Treatment
Dynamic movement at joints and cracks	Lack of support from sub-base. Lack of, or ineffective load transfer dowels or tie bars at joints	Pressure or vacuum grouting
Faulting at joints and cracks	Lack of, or ineffective load transfer dowels or tie bars at joints	Slab lifting in conjunction with pressure or vacuum grouting Grinding after pressure or vacuum grouting

14.4.1 *Slab lifting*

Raising the level of slabs by lifting is a controllable process in which the slab is connected to a frame straddling the bay and hydraulically jacked to the required level a few millimetres at a time. While the slab is still connected to the lifting frame, the void created underneath should be filled by pressure- or vacuum-grouting. When a substantial length of slab is lifted, it may be necessary to stitch tie bars across the longitudinal joint to stop it opening subsequently.

14.4.2 *Pressure-grouting*

Pressure-grouting is used either to fill small voids and stabilise dynamic movement of the slab, or to fill the voids created when slabs are raised to correct settlement or faulting at joints and cracks. As well as cementitious and resin grouts, a dry mix mortar may be used to fill voids, but it may be necessary to raise the slab initially to a slightly higher level than is actually required to allow for future compaction under traffic. Fluid grout is more suitable for filling smaller voids under the slab.

14.4.3 *Vacuum-grouting*

For vacuum-grouting, a low-viscosity resin grout is induced to flow into voids beneath the slab by applying a vacuum. Holes about 30 mm in diameter are drilled through the slab on a one metre square grid for vacuum suction and grout injection. The advantages of the process are that any water beneath the slab is drawn off before the grout is injected and the low viscosity of the grout enables small voids to be penetrated. There is also little danger of inadvertently filling service ducts.

Slab lifting, pressure- and vacuum-grouting may not produce a durable repair if the sub-base is of unbound material. In this case, the sub-base itself and/or the underlying formation may have been weakened by persistent water penetration through a defective joint seal. The slab may then settle further despite the voids having been filled with grout.

14.4.4 *Full-depth corner repair*

Full-depth corner repairs have been widely used and have often been regarded as long-term repairs. Experience indicates that a significant proportion fail through separation from the original concrete or local settlement well in advance of failure in the original pavement. A

full-width full-depth bay end repair as described Section 14.2.3 is always preferable. Full-depth corner repairs should not exceed the maximum dimensions given in Figure 14.5.

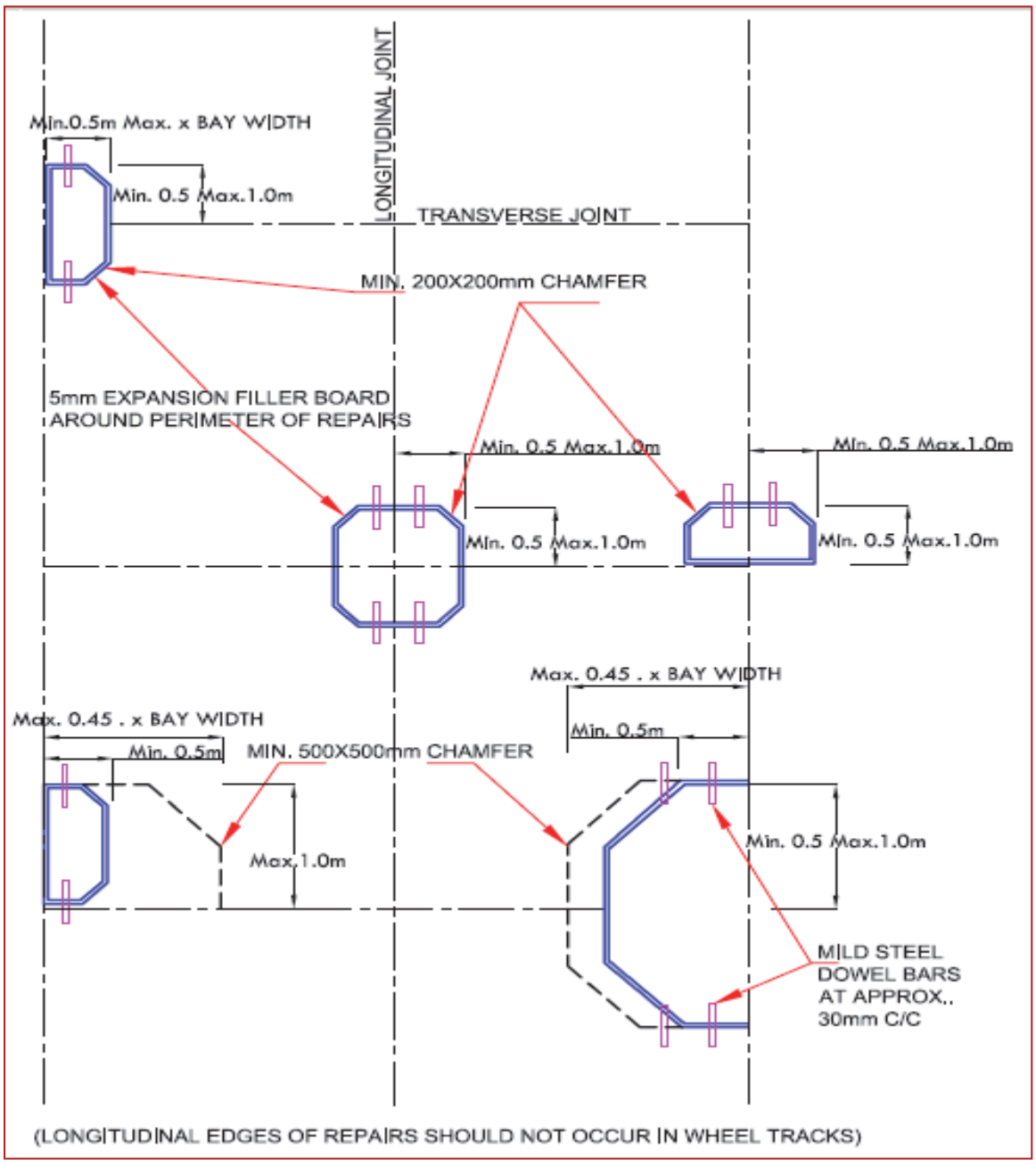


Figure 14-5 Full depth corner repairs

For corner repairs a chamfer, as large as possible, should be provided across the corner as shown in Figure 14.5. This reduces the risk of a crack subsequently developing across the slab from that point. It may therefore not be possible to extend the saw cuts around the corners of the repair through the full slab depth - necessitating careful breaking out to achieve the vertical face required in the corners. Particular care should be taken to avoid damaging the remaining top edges of the slab.

The initial thermal and shrinkage movements of the repair concrete must not be restricted in any way, nor should the repaired slab inhibit contraction or expansion movement in the existing slab. Hence it is recommended that no dowel or tie bars be provided in the edge parallel to the longitudinal axis of the slab, and that a 5 mm expansion filler board be installed around the perimeter of each repair.

14.5 Polymer modified repair material

There are various proprietary materials for filling cracks and making good surface defects. Most of them incorporate polymer-modified cement and/or bitumen as the binder. In addition to conventional sand or gravel aggregate, these repair materials may also incorporate glass fibres and rubber granules. In general, those with polymer-modified bitumen binders are applied hot and those with polymer-modified cement are applied cold. All are capable of sustaining strains significantly greater than conventional repairs using Portland cement. Depending on the particular formulation and repair depth, the surface may be opened to traffic between 10 minutes and 2 hours after laying. In all cases, it is important to comply strictly with the manufacturers' recommendations.

The advantages of polymer-modified materials over Portland cement repairs are normally claimed by the manufacturers to include:

i) short curing/setting period before re-opening to traffic.
ii) ductility enables the repair to withstand some substrate movement.
iii) some have low shrinkage characteristics.

Polymer-modified materials are available suitable for sealing cracks that have been routed to form a sealing groove that may be up to about 30 mm wide and 25 mm deep. Again, it is important to follow the manufacturers' recommendations: some advise the use of an elongated patch repair 300-400 mm wide over the crack and this may be preferred for cracks that have branched into two (cracks), or are difficult to follow with a mechanical router.

14.6 Partial depth cementitious repairs

Partial-depth cementitious repairs are used for spalling etc. that exceed the maximum depth for which thin-bonded repairs may be used and include surface repairs no deeper than one-third of the total slab depth. Such repairs have become possible with the advent of polymer-modified cements, particularly the rapid-hardening varieties.

Studies on partial-depth repairs showed that 80-100% of appropriately installed partial-depth patches with effective quality control performed well after 3-10 years. Less than 2% failed within 18 months. If partial-depth repairs are being considered, they must be preceded by the investigative techniques described in this manual to check that the defect is confined to the upper one-third of the slab depth.

Figure 14.6 shows an appropriate procedure for a partial-depth joint repair with polymer modified material. Here, a partial-depth repair is defined as deeper than the joint groove but not exceeding one-third of the slab depth. The finished surface of the repair should have a uniform surface texture and appearance and should be free from droppings, excess overlapping, damage by rain or frost, or other deposits.

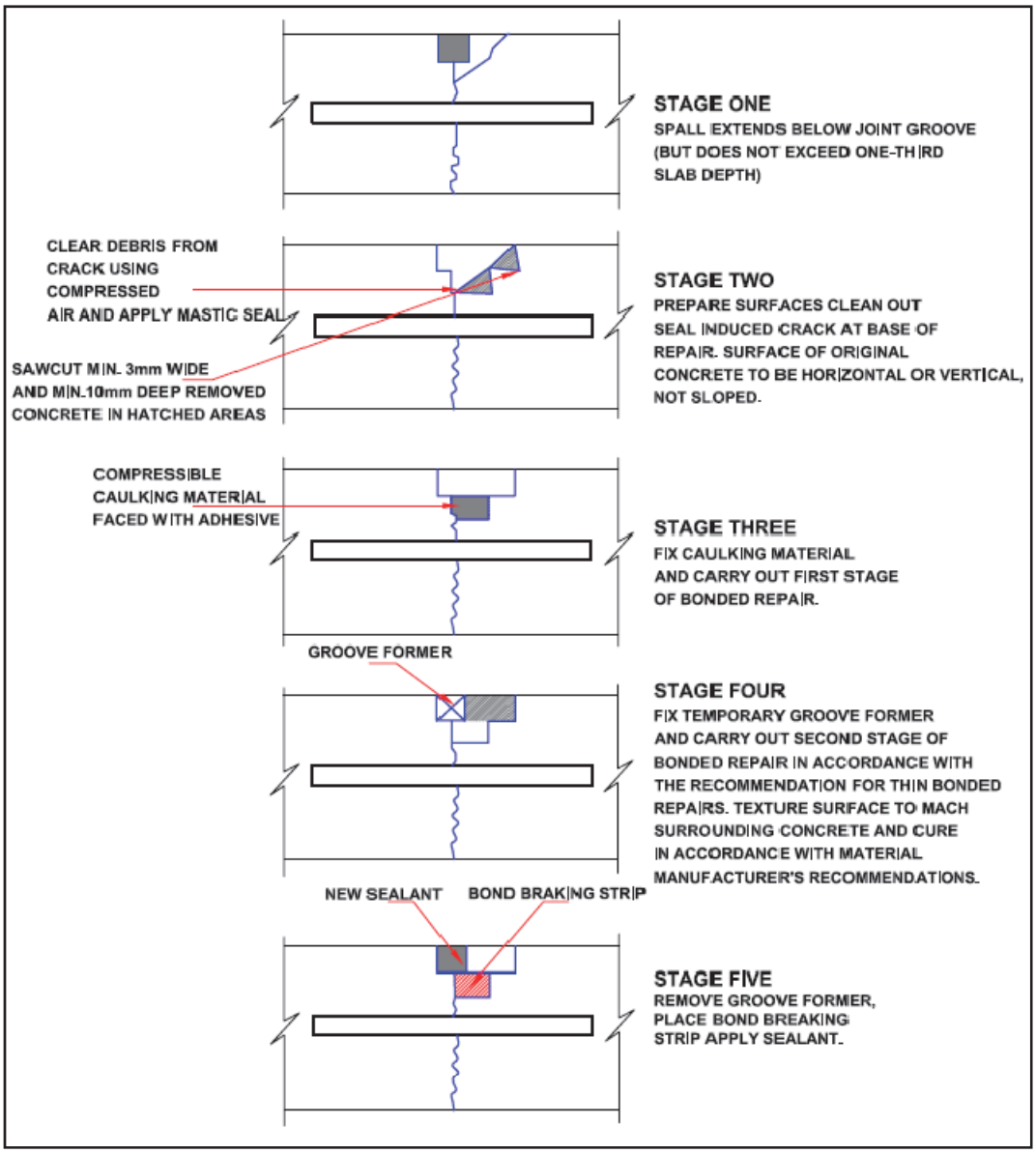


Figure 14-6 Partial-depth repair at a joint using polymer-modified material

15. OVERLAYS AND RECONSTRUCTION

15.1 Introduction

Overlays on unreinforced concrete roads are an alternative to reconstruction. HMA overlays are popular but concrete overlays should also be considered.

If HMA overlays over unreinforced concrete are to be successful then some measures need to be taken to control the rate and occurrence of reflection cracking. There are a number of alternatives;

a) Saw, cut and seal the HMA overlay at locations coinciding with the joints in the existing concrete pavement. All other cracks must be neutralised using the techniques described herein.

b) Use a thick HMA overlay. Reflection cracks take longer to propagate through thicker overlays. However this is not a reliable method on its own except if deflections are very low across all the slabs; a very unlikely scenario.

c) Include a bituminised granular interlayer between the old concrete and the new overlay. This method is reasonably reliable but it is almost equivalent to constructing a new road on top of the old one.

d) Crack and seat the existing concrete pavement prior to overlay. This technique reduces the sizes of the pieces of the old concrete layer and 'seats' them in the underlying sub-base thereby making a stable platform. The problem of reflection cracking is much reduced but the process requires a skilful contractor with the right equipment, which includes a very heavy roller. The method has been quite successful.

e) Rubblise the existing concrete pavement prior to overlay. This is a more extreme form of breaking up the old pavement, similar in principle to 'crack and seat'. It is a half way house between crack and seat and full recycling.

f) Full recycling. This is essentially reconstruction with the bonus that the existing material from the old concrete is recycled to be used as sub-base or road base in the new road.

None of these techniques are ideal because those that are reliable are expensive and require considerable skill by the contractor. The less expensive options are much less reliable. Hence, although the advantages of a rigid pavement are attractive, namely long life, the costs of rehabilitation need to be factored into the calculations and it is clear that good maintenance plays a vital role in achieving the benefits.

15.2 Assessment of the existing concrete pavement

Thin HMA overlays (≤ 100mm) are not effective in maintaining unreinforced concrete roads unless concrete slabs having the higher levels of crack severity and FWD deflection are either stitched or 'removed and replaced' prior to overlay (see (a) above). Where too many slabs need to be stitched or replaced, it becomes uneconomic to use an HMA overlay on its own and one of the reflection crack treatments listed above will also need to be used.

The development of reflection cracking in thin HMA overlays is controlled by the condition of the underlying unreinforced concrete pavement and its support (measured with the FWD or Benkelman beam) as shown in Tables 15.1 and 15.2. Therefore, prior to

constructing an HMA overlay, a visual survey and mid-slab deflection survey should be carried out. *Each* concrete slab should be visually assessed and, ideally, all slabs should be tested with the FWD. However, if resources are limited, testing can be restricted to those slabs having condition codes 3, 4 and 5 (See Table 15.2). All FWD deflection should be normalised to a load of 50kN.

Table 15-1 Classification of crack width

Crack definition	Width[1] (mm)	Condition
Narrow	< 0.5	Full aggregate interlock and load transfer
Medium	0.5 – 1.5	Partial interlock and load transfer. May permit entry of water
Wide	> 1.5	No load transfer. Ingress of water and fine material

Note 1 Width of crack where there is no spalling.

Table 15-2 Classification of crack severity

Code	Slab definition	Condition
1	Good	Slab which has no cracking
2	Fair	A slab which has some cracks but no cracks which are full slab width or length (either transverse or longitudinal)
3	Average	A slab which has cracks that include *one* full slab (width or length) crack of medium width but less than two full slab (width or length) cracks of medium width.
4	Poor	A slab that has cracks that include *two* full slab (width or length) cracks of medium width. Also a slab that has at least *one* full slab (width or length) crack of medium width plus minor asphalt repair (e.g. one corner patch etc)
5	Very poor	A slab that has multiple cracks with *three or more* full slab cracks (width or length of the slab) of medium width. Also a slab that has *two* full slab cracks (width or length of the slab) of medium width and substantial asphalt patching or lengths of sealed cracks.

Table 15.3 gives recommendations based on the severity of cracking and FWD central deflection which can be used to identify which slabs should be stitched or replaced prior to overlay. Technical recommendations for stitching and slab replacement are given in Sections 14.2.1 and 14.2.2 respectively.

Table 15-3 Recommendations for remedial works on existing concrete surface

Condition Code	FWD deflection d_1 (microns) @ 50kN		
	$d_1 < 300$	$300 < d_1 < 400$	$d_1 > 400$
1	Leave old concrete	Investigate[1]	Investigate[1]
2	Leave old concrete	Investigate[1]	Investigate[1]
3	Leave old concrete	Stitch cracks[2]	Remove and Replace
4	Stitch cracks[2]	Stitch cracks[2]	Remove and Replace
5	Remove and Replace	Remove and Replace	Remove and Replace

Notes 1. Unlikely combination unless pavement is relatively new. Investigate whether poor support is the cause of high FWD deflection.
2. Stitch full width/length cracks.
3. d_1 is the central FWD deflection.

15.3 HMA overlay thickness

A 50mm HMA overlay can be used to rehabilitate jointed unreinforced concrete pavements up to a design traffic of one million esa if the recommendations in Table 15.3 are followed. A 100mm HMA overlay can be used to rehabilitate jointed unreinforced concrete pavements up to a design traffic of 3.0 million esa. These are not very high traffic levels and the recommendations are conservative. However until the industry in Ethiopia has gained experience in building and maintaining concrete roads, it is best to remain on the side of safety.

15.4 Concrete overlays

Concrete overlays are normally at least 150 mm in depth and may be either 'jointed unreinforced' concrete, 'jointed reinforced' concrete or 'continuously reinforced' concrete. The existing pavement can provide a sound and strong sub-base that will not be prone to erosion hence the thickness design can be based on the design charts for new roads (see ERA *Pavement Design Manual).*

Jointed unreinforced and jointed reinforced concrete overlays should be unbonded. This does not constrain the designer to the same slab shape and size as the underlying pavement, but the design must ensure that debonding occurs by some positive means. When a jointed concrete overlay is used, plastic sheeting is the most appropriate way of ensuring the overlay is not bonded to the existing road. When continuously reinforced concrete (CRC) is used, the overlay should be uniformly bonded to the underlying layer.

15.5 Cracking and seating

Where the existing pavement is in a suitable condition, the existing jointed unreinforced concrete pavement should be cracked and seated prior to constructing an unbonded concrete overlay. Fine vertical transverse cracks are induced in the unreinforced concrete to create closely-spaced locations where thermal contraction can take place while retaining satisfactory load-carrying and load-transfer characteristics. When the concrete is cracked and seated, its load-spreading ability is reduced, depending on the remaining degree of interlock at the cracks and on the crack spacing. Consequently, there is a contradictory

requirement between the need to increase the number of cracks to help control reflective cracking and the need to retain aggregate interlock to assist load spreading and prevent the subgrade being overstressed.

The crack-and-seat process reduces the effective stiffness modulus of the concrete. This must be taken into account when designing the overlay. Crack spacings are typically 1 m, the recommended range being 0.75-2.0 m. It is useful to try to specify a spacing that divides equally into the existing slab dimension. The crack pattern on the surface of the concrete must be predominantly transverse, with longitudinal cracks being avoided. To help produce such crack patterns, the plant for the cracking process should have a guillotine action capable of delivering variable pre-set impact loads to the concrete surface. Before overlay, the cracked concrete should be seated with a pneumatic-tyred roller with a ballasted weight of at least 20 tonnes. Tests elsewhere have shown that six passes over every point of the cracked and seated concrete is sufficient to seat the pavement.

The overlay itself may be either a jointed unreinforced or jointed reinforced concrete overlay. However, continuously reinforced concrete (CRC) in particular offers good load-spreading properties that enable it to accommodate some localised variation in support from the underlying materials. Where the existing unreinforced concrete pavement is both particularly distressed and the joints exhibit poor load-transfer characteristics, a combination of crack-and-seating and a CRC overlay is strongly recommended.

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Appendix A Structural Number Approach

A.1 Introduction

The concept of structural number was first introduced as a result of the AASHO Road Test as a measure of overall pavement strength. It is essentially a measure of the total thickness of the road pavement weighted according to the 'strength' of each layer and calculated as follows:

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

where

- SN = structural number of the pavement,
- a_i = strength coefficient of the i^{th} layer,
- h_i = thickness of the i^{th} layer, in millimetres,
- m_i = 'drainage' coefficients that modify the layer strength coefficients of unbound materials if drainage is poor and/or climate is favourable or severe.

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

The individual layer strength coefficients are determined from the normal tests that are used to define the strength of the material in question e.g. CBR for granular materials, UCS for cemented materials etc. Table A.1 shows typical values.

To design the rehabilitation measures it is necessary to measure the structural number of the existing road and the strength of the subgrade. DCP and test pit data are used for this purpose.

If the DCP is used to measure the in situ strengths and the thicknesses of the layers of the existing pavement at the time of year that the pavement is at its weakest, no adjustments are required for the season of the year.

If the asphalt concrete surfacing on the existing road is in poor condition (e.g. badly cracked) its strength coefficient will be low and very variable. However it is unlikely that in this condition an overlay will be the preferred rehabilitation option and hence an accurate assessment of the strength coefficient of the surfacing for structural design of the overlaid road is not necessary. In situations where overlaying is the preferred option, an estimate of the strength coefficient of the asphalt surfacing is required. The bitumen towards the top of this layer is likely to be aged and fairly brittle, but this is compensated by the high road temperatures that occur in tropical regions. Its operating temperature will, however, also depend on the thickness of overlay and the local climatic conditions. Estimating its effective value is therefore difficult. Experience has indicated that a value of 0.35 is usually suitable but this can be varied from 0.25 up to 0.4 based on engineering judgement.

Table A1 Pavement layer strength coefficients

Layer	Layer Type	Condition	Coefficient
Surfacing	Surface dressing		$a_i = 0.1$
	New asphalt concrete [1,2] wearing	MR_{30} = 1500 MPa	$a_i = 0.30$
		MR_{30} = 2000 MPa	$a_i = 0.35$
		MR_{30} = 2500 MPa	$a_i = 0.40$
		$MR_{30} \geq 3000$ MPa	$a_i = 0.45$
Road base	Asphalt concrete	As above	As above
	Granular unbound	Default	$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
		GB 1 (CBR > 100%)	0.145
		GB 2 (CBR = 100%)	0.14
		GB 3 (CBR = 80%) With a stabilised layer underneath	0.135
		With an unbound granular layer underneath	0.13
		GB 4 (CBR = 65%) (4)	0.12
		GB 5 (CBR = 55%)(4)	0.107
		GB 6 (CBR = 45%)(4)	0.01
	Bitumen treated gravels and sands	Marshall stability = 2.5 MN	a = 0.135
		Marshall stability = 5.0 MN	a = 0.185
		Marshall stability = 7.5 MN	a = 0.23
	Cemented[3]	Equation	$a_i = 0.075 + 0.039\ UCS - 0.00088(UCS)^2$
		CB 1 (UCS = 3.0 – 6.0 MPa)	a = 0.18
		CB 2 (UCS = 1.5 – 3.0 MPa)	a = 0.13
Sub-base	Granular unbound	Equation	$a_j = -0.075 + 0.184(\log_{10} CBR) - 0.0444(\log_{10} CBR)^2$
		GS (CBR = 30%)	a = 0.105
		GC (CBR = 15%)	a = 0.08
	Cemented	CB 3 (UCS = 0.7 – 1.5 MPa)	a = 0.1

Notes:

1. See discussion above.
2. Unconfined Compressive Strength (UCS) is quoted in MPa at 14 days.
3. MR_{30} is the resilient modulus by the indirect tensile test at 30 °C.
4. Used for low volume roads (see LVR manual)

The drainage coefficients are effectively calibration factors for the moisture regime experienced by the road and are therefore related to *both* climate and drainage. They are obtained by comparing the performance of roads in a particular climatic region with the expected performance based on the AASHTO design equation with the coefficients set at unity. In the AASHTO design manual the quoted values range from 0.4 for extremely poor conditions up to 1.4 for very good conditions, but the usual working range is 0.8 to 1.2. However, for a well designed road the effects of its moisture regime or climate are primarily manifest in the strength of the subgrade, and considerable effort is spent in obtaining the most accurate value of this for design purposes. Poor drainage is unlikely to affect an entire road and the variable nature of internal drainage is best considered as part of the overall variability in performance that is captured in the standard deviation term S_0 in the design equation.

A.2 Modified Structural Number

The AASHO Road Test was constructed on a single subgrade, therefore the effect of different subgrades could not be estimated and the structural number could not include a subgrade contribution. To overcome this problem and to extend the concept to all subgrades, a subgrade contribution was derived as described by Hodges et al. (1975) and a modified structural number defined as follows:

$$SNC = SN + 3.51\ (\log_{10} CBR_s) - 0.85\ (\log_{10} CBR_s)^2 - 1.43$$

where

SNC = Modified structural number of the pavement
CBR_s = in-situ CBR of the subgrade

The modified structural number (SNC) has been used extensively and forms the basis for defining pavement strength in many pavement performance models.

A.3 Adjusted Structural Number

Many road pavements cannot be divided easily into distinct roadbase and sub-base layers with a well-defined and uniform subgrade. Hence, when calculating the structural number according to the equation above, the engineer has to judge which layers to define as roadbase, which as sub-base, and where to define the top of the subgrade. For many roads this has proven quite difficult. There are often several layers that could be considered either as sub-bases or part of the subgrade, especially where capping layers or selected fill have been used. The simple summation over all the apparent layers allows the engineer to obtain almost any value of structural number since the value will depend on where the engineer assumes that the sub-base(s) end and the subgrade begins. In the past this problem has been addressed by simply limiting the total depth of all the layers that are considered to be road pavement. However, this is somewhat arbitrary, has not been used universally, and has led to unacceptably large errors in some circumstances.

The problem arises because the contributions of each layer to the structural number are independent of depth. This cannot be correct since logic dictates that a layer that lies very deep within the subgrade can have little or no influence on the performance of the road. To eliminate the problem, a method of calculating the modified structural number has been devised in which the contributions of each layer to the overall structural number decrease with depth (Rolt and Parkman, 2000).

To distinguish the structural number derived from the original Modified Structural Number (SNC), the new structural number is called the Adjusted Structural Number (SNP). It is calculated as follows:

$$SNP = SNA + SNS + SNG$$

Where the component terms are calculated follows;

$$SNA = 0.0394 \sum_{i=1}^{n} a_i h_i$$

$$SNS = 0.0394 \sum_{j=1}^{m} a_j \left\{ \left(\frac{b_0 \exp(-b_3 z_j)}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_j)}{(b_2 + b_3)} \right) - \left(\frac{b_0 \exp(-b_3 z_{j-1})}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_{j-1})}{(b_2 + b_3)} \right) \right\}$$

$$SNG = (b_0 - b_1 \exp(-b_2 z_m))\,(\exp(-b_3 z_m))\,[3.51 \log_{10} CBR - 0.85(\log_{10} CBR)^2 - 1.43]$$

and

SNP	=	adjusted structural number of the pavement
SNA	=	contribution of surfacing and base layers
SNS	=	contribution of the sub-base and selected fill layers
SNG	=	contribution of the subgrade
n	=	number of base and surfacing layers (i = 1, n)
a_i	=	layer coefficient for base or surfacing layer i
h_i	=	thickness of base or surfacing layer i, in mm
m	=	number of sub-base and selected fill layers (j = 1, m)
a_j	=	layer coefficient for sub-base or selected fill layer j for season s
z	=	depth parameter measured from the top of the sub-base (underside of base), in mm
z_j	=	depth to the underside of the jth layer ($z_0 = 0$), in mm
CBR	=	in situ subgrade CBR

The values of the model coefficients b_0 to b_3 are given in Table A.2.

Table A.2 Adjusted structural number model coefficients

b_0	b_1	b_2	b_3
1.6	0.6	0.008	0.00207

It should be noted that for roads that have been built according to the designs in the ERA *Pavement Design Manual* (or any other manual for that matter) with well defined layers of uniform strength, the Adjusted Structural Number and the Modified Structural Number are essentially identical. The value of SNP is calculated when evaluating a pavement with many layers of varying strength.

A.4 Target Structural Numbers

When designing rehabilitation it is necessary to determine the existing effective SN and the required SN to carry the additional traffic. Tables A.3 and A.4 show the target values of SN and SNC for different subgrade conditions and for various pavement types calculated from the design charts in the ERA *Pavement Design Manual.*

When designing an overlay, the main load bearing layer will be the overlay itself and the existing old surfacing. Thus the designs will be based on Charts C, D or E depending on the existing structure.

Table A.3 Target Structural Numbers for different structures

Chart No	Subgrade	T1 <0.3	T2 0.3 - 0.7	T3 0.7 - 1.5	T4 1.5 - 3.0	T5 3 - 6	T6 6 - 10	T7 10 - 17	T8 17 - 30	T9 30 - 50	T10 50 - 80
Chart A	S1			2.90	3.10	3.40	3.65				
	S2			2.45	2.70	2.95	3.20				
	S3			2.15	2.35	2.65	2.75				
	S4			1.95	2.05	2.25	2.45				
	S5			1.60	1.75	1.90	2.05				
	S6			1.10	1.25	1.35	1.50				
Chart B	S1	2.60	2.75	2.85	3.00	3.25	3.45	3.55			
	S2	2.15	2.30	2.40	2.55	2.80	3.00	3.25			
	S3	1.85	1.90	2.05	2.25	2.50	2.85	2.95			
	S4	1.55	1.65	1.80	1.90	2.15	2.35	2.65			
	S5	1.40	1.40	1.55	1.65	1.80	1.90	2.15			
	S6	0.90	0.90	1.05	1.15	1.25	1.40	1.65			
Chart C	S1				4.50	4.95	5.15	5.60	5.90	6.40	6.75
	S2				4.15	4.55	4.75	5.15	5.45	5.95	6.35
	S3				3.70	4.10	4.30	4.55	4.85	5.35	5.70
	S4				3.30	3.65	3.90	4.15	4.45	4.80	5.20
	S5				2.75	3.15	3.35	3.60	3.90	4.25	4.65
	S6				2.65	3.00	3.25	3.45	3.75	4.15	4.50
Chart D	S1				4.25	4.50	4.85	5.25	5.70	6.05	6.40
	S2				3.95	4.15	4.50	4.95	5.35	5.70	6.10
	S3				3.65	3.85	4.25	4.65	5.05	5.45	5.80
	S4				3.20	3.40	3.75	4.05	4.45	4.80	5.20
	S5				3.05	3.25	3.60	3.90	4.35	4.55	4.90
	S6				3.05	3.25	3.60	3.90	4.35	4.55	4.90
Chart E	S1				4.30	4.65	5.05	5.40	5.80	6.05	6.50
	S2				4.00	4.30	4.65	5.00	5.25	5.65	6.05
	S3				3.65	3.90	4.20	4.45	4.85	5.10	5.55
	S4				3.15	3.45	3.70	4.00	4.40	4.65	5.05
	S5				2.95	3.25	3.50	3.80	4.05	4.35	4.75
	S6				2.55	2.85	3.10	3.40	3.65	3.95	4.35
Chart F	S1	2.60	2.70	2.80	2.95	3.15	3.25	3.40			
	S2	2.20	2.30	2.45	2.55	2.75	2.95	3.10			
	S3	1.95	1.95	2.10	2.20	2.40	2.60	2.75			
	S4	1.50	1.60	1.70	1.90	2.15	2.35	2.45			
	S5	1.30	1.30	1.40	1.60	1.85	1.95	2.10			
	S6	0.85	0.85	1.00	1.10	1.25	1.40	1.50			

Table A.4 Target Modified Structural Numbers for different structures

Chart No	Subgrade	T1	T2	T3	T4	T5	T6	T7	T8	T9	T10
		<0.3	0.3 - 0.7	0.7 - 1.5	1.5 - 3.0	3 - 6	6 - 10	10 - 17	17 - 30	30 - 50	50 - 80
Chart A	S1			2.45	2.65	2.95	3.20				
	S2			2.50	2.75	3.00	3.25				
	S3			2.75	2.95	3.25	3.35				
	S4			3.00	3.10	3.30	3.50				
	S5			3.10	3.25	3.40	3.55				
	S6			3.00	3.15	3.25	3.40				
Chart B	S1	2.15	2.30	2.40	2.55	2.80	3.00	3.10			
	S2	2.20	2.35	2.45	2.60	2.85	3.05	3.30			
	S3	2.45	2.50	2.65	2.85	3.10	3.45	3.55			
	S4	2.60	2.70	2.85	2.95	3.20	3.40	3.70			
	S5	2.90	2.90	3.05	3.15	3.30	3.40	3.65			
	S6	2.80	2.80	2.95	3.05	3.15	3.30	3.55			
Chart C	S1				4.05	4.50	4.70	5.15	5.45	5.95	6.30
	S2				4.20	4.60	4.80	5.20	5.50	6.00	6.40
	S3				4.30	4.70	4.90	5.15	5.45	5.95	6.30
	S4				4.35	4.70	4.95	5.20	5.50	5.85	6.25
	S5				4.25	4.65	4.85	5.10	5.40	5.75	6.15
	S6				4.55	4.90	5.15	5.35	5.65	6.05	6.40
Chart D	S1				3.80	4.05	4.40	4.80	5.25	5.60	5.95
	S2				4.00	4.20	4.55	5.00	5.40	5.75	6.15
	S3				4.25	4.45	4.85	5.25	5.65	6.05	6.40
	S4				4.25	4.45	4.80	5.10	5.50	5.85	6.25
	S5				4.55	4.75	5.10	5.40	5.85	6.05	6.40
	S6				4.95	5.15	5.50	5.80	6.25	6.45	6.80
Chart E	S1				3.85	4.20	4.60	4.95	5.35	5.60	6.05
	S2				4.05	4.35	4.70	5.05	5.30	5.70	6.10
	S3				4.25	4.50	4.80	5.05	5.45	5.70	6.15
	S4				4.20	4.50	4.75	5.05	5.45	5.70	6.10
	S5				4.45	4.75	5.00	5.30	5.55	5.85	6.25
	S6				4.45	4.75	5.00	5.30	5.55	5.85	6.25
Chart F	S1	2.15	2.25	2.35	2.50	2.70	2.80	2.95			
	S2	2.25	2.35	2.50	2.60	2.80	3.00	3.15			
	S3	2.55	2.55	2.70	2.80	3.00	3.20	3.35			
	S4	2.55	2.65	2.75	2.95	3.20	3.40	3.50			
	S5	2.80	2.80	2.90	3.10	3.35	3.45	3.60			
	S6	2.75	2.75	2.90	3.00	3.15	3.30	3.40			

Appendix B DCP Measurements

B.1 Introduction

The Dynamic Cone Penetrometer (DCP) is an instrument which can be used for the rapid measurement of the in situ strength of existing pavements constructed with unbound materials. Measurements can be made down to a depth of approximately 800mm or, when an extension rod is fitted, to a depth of 1200mm. Where pavement layers have different strengths, the boundaries between them can be identified and the thickness of each layer estimated.

DCP tests are particularly useful for identifying the cause of road deterioration when it is associated with one of the unbound pavement layers, e.g. shear failure of the roadbase or sub-base. A comparison between DCP test results from sub-sections that are just beginning to fail and those that are sound will quickly identify the pavement layer which is the cause of the problem.

It is usually convenient to convert the individual pavement layer thicknesses and strengths measured in the DCP test into a structural number as described in Appendix A.

If it is suspected that the road failures are related to the *overall* structural strength of the pavement, the Modified Structural Number of different sub-sections can be readily compared to identify the weakness.

B.2 DCP test procedure

The standard DCP uses an 8kg hammer dropping through a height of 575mm and a 60° cone having a maximum diameter of 20mm.

The instrument is assembled as shown in Figure B.1. It is supplied with two spanners and a tommy bar to ensure that the screwed joints are kept tight at all times. To assist in this the following joints should be secured with a non-hardening thread locking compound prior to use:

i) Handle/hammer shaft
ii) Coupling/hammer shaft
iii) Standard shaft/cone

The instrument is usually split at the joint between the standard shaft and the coupling for carriage and storage and therefore it is not usual to use locking compound at this joint. However it is important that this joint is checked regularly during use to ensure that it does not become loose. Operating the DCP with any loose joints will significantly reduce the life of the instrument.

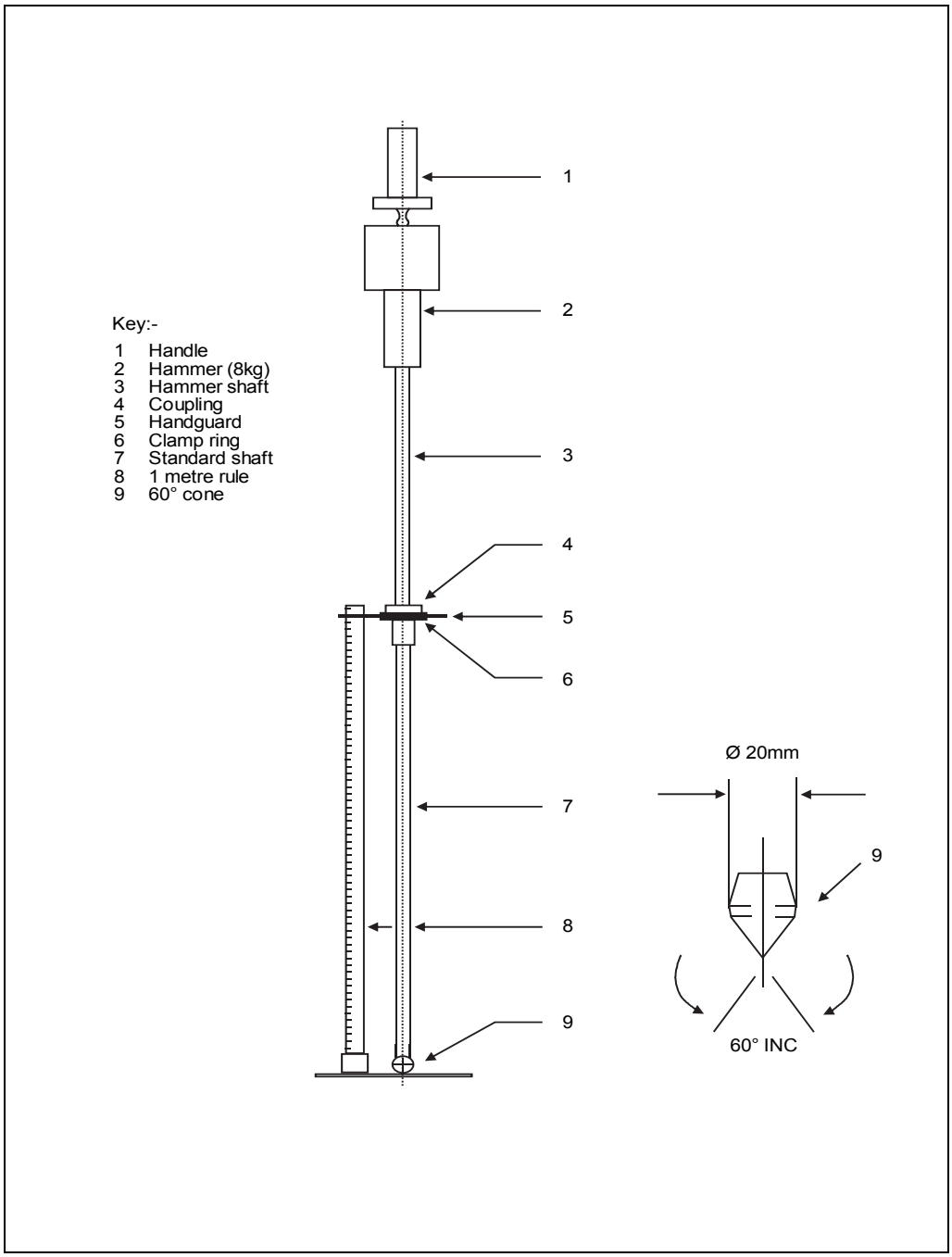


Figure B.1 Dynamic Cone Penetrometer

B.3 Operation

A safe working environment should be maintained at all times. Many organisations have on-site safety procedures which should be followed. Where there are no local safety procedures those in TRL's Overseas Road Note 2 are recommended.

After assembly, the first task is to record the zero reading of the instrument. This is done by standing the DCP on a hard surface, such as concrete, checking that it is vertical and then entering the zero reading in the appropriate place on the proforma (See Figure B.2).

The DCP needs three operators, one to hold the instrument, one to raise and drop the weight and a technician to record the readings. The instrument is held vertical and the weight raised to the handle. Care should be taken to ensure that the weight is touching the handle, but not lifting the instrument, before it is allowed to drop. The operator must let it fall freely and not partially lower it with his hands.

It is recommended that a reading should be taken at increments of penetration of about 10mm. However it is usually easier to take a reading after a set number of blows. It is therefore necessary to change the number of blows between readings, according to the strength of the layer being penetrated. For good quality granular bases readings every 5 or 10 blows are usually satisfactory but for weaker sub-base layers and subgrades readings every 1 or 2 blows may be appropriate. There is no disadvantage in taking too many readings, but if readings are taken too infrequently, weak spots may be missed and it will be more difficult to identify layer boundaries accurately, hence important information will be lost.

When the extended version of the DCP is used the instrument is driven into the pavement to a depth of 400-500mm before the extension shaft can be added. To do this the metre rule is detached from its base plate and the shaft is split to accept the extension shaft. After re-assembly a penetration reading is taken before the test is continued.

After completing the test the DCP is removed by tapping the weight upwards against the handle. Care should be taken when doing this; if it is done too vigorously the life of the instrument will be reduced.

The DCP can be driven through surface dressings but it is recommended that thick bituminous surfacings are cored prior to testing the lower layers. This should be done using as little lubricating water as possible to avoid wetting the layer below and obtaining an incorrect strength reading. Little difficulty is normally experienced with the penetration of most types of granular or lightly stabilised materials. It is more difficult to penetrate strongly stabilised layers, granular materials with large particles and very dense, high quality crushed stone. The TRL instrument has been designed for strong materials and therefore the operator should persevere with the test. Penetration rates as low as 0.5mm/blow are acceptable but if there is no measurable penetration after 20 consecutive blows then it can be assumed that the DCP will not penetrate the material. Under these circumstances a hole can be drilled through the layer using an electric or pneumatic drill, or by *dry* coring. The lower pavement layers can then be tested in the normal way. If only occasional difficulties are experienced in penetrating granular materials, it is worthwhile repeating any failed tests a short distance away from the original test point.

If, during the test, the DCP leans away from the vertical, no attempt should be made to correct it because contact between the shaft and the sides of the hole can give rise to an overestimate of subgrade strength. This is a result of friction on the rod caused by either tilted penetration through, or collapse of, any upper granular pavement layers. Where there is a substantial thickness of granular material, and when estimates of the actual subgrade strength are required (rather than relative values) it is recommended that a hole is drilled through the granular layer prior to testing the lower layers.

It is always advisable to check that side friction has not influenced the result of a DCP test. This is easily done by attempting to twist the shaft when the DCP is at full penetration. If

the shaft cannot be spun reasonably easily between the fingers then there is too much side friction and the test should be repeated. The erroneous test should be marked as such but not abandoned because the side friction problem may not have arisen for the first part of the test.

If the DCP is used extensively for hard materials, wear on the cone itself will be accelerated. The cone is a replaceable part and it is recommended by other authorities that it should be replaced when its diameter is reduced by 10 per cent. However, other causes of wear can also occur hence the cone should be inspected before every test.

B.4 Interpretation of results

The results of the DCP test are usually recorded on a field data sheet similar to that shown in Figure B.2. The results can then either be plotted by hand, as shown in Figure B.3, or processed by computer.

Relationships between DCP readings and CBR have been obtained by several research authorities (see Figure B.4). Agreement is generally good over most of the range but differences are apparent at low values of CBR in fine grained materials. It is expected that for such materials the relationship between DCP and CBR will depend on material state and therefore, if more precise values are needed it is advisable to calibrate the DCP for the material being evaluated.

DCP TEST

Site/Road: Date:
Test No:
Section No/Chainage:
Direction: Zero reading of DCP:
Wheel path: Started test at:

No. Blows	Σ Blows	mm	No. Blows	Σ Blows	mm	No. Blows	Σ Blows	mm

Figure B.2 DCP Test Field Sheet

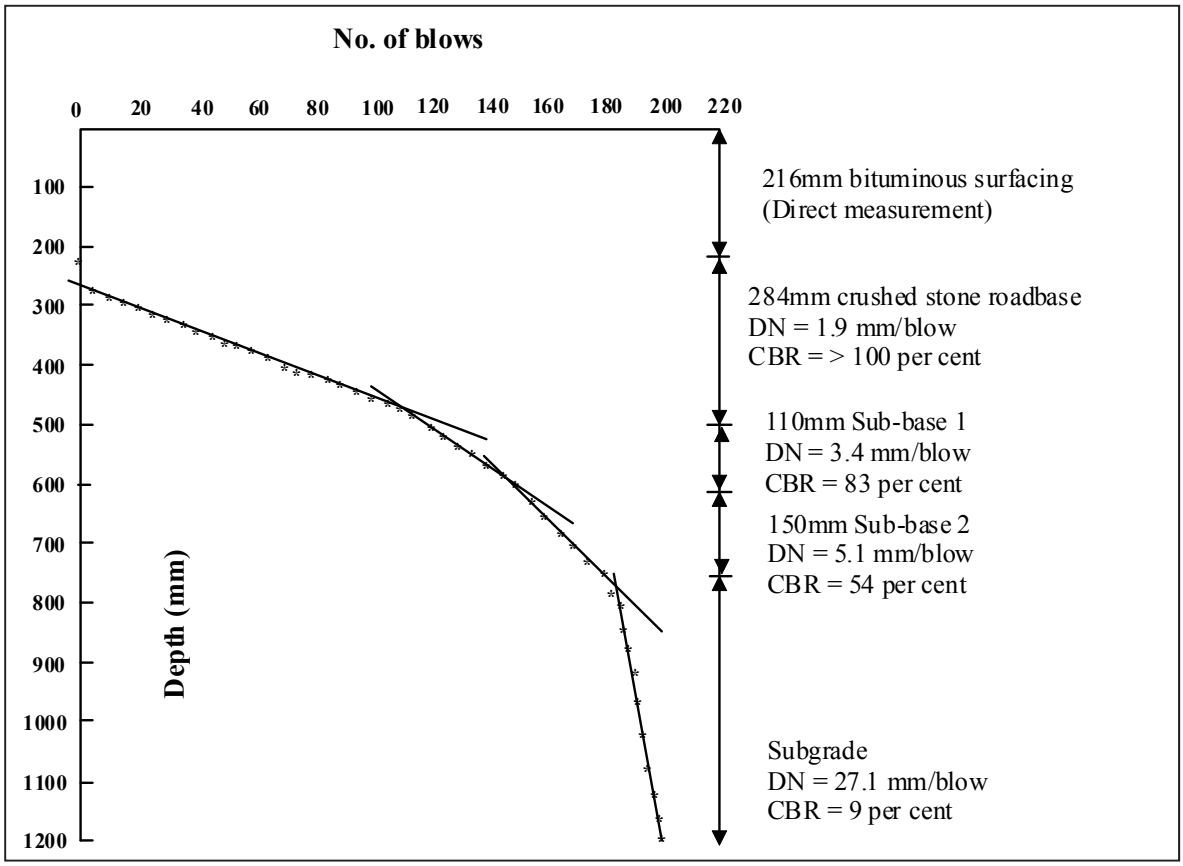
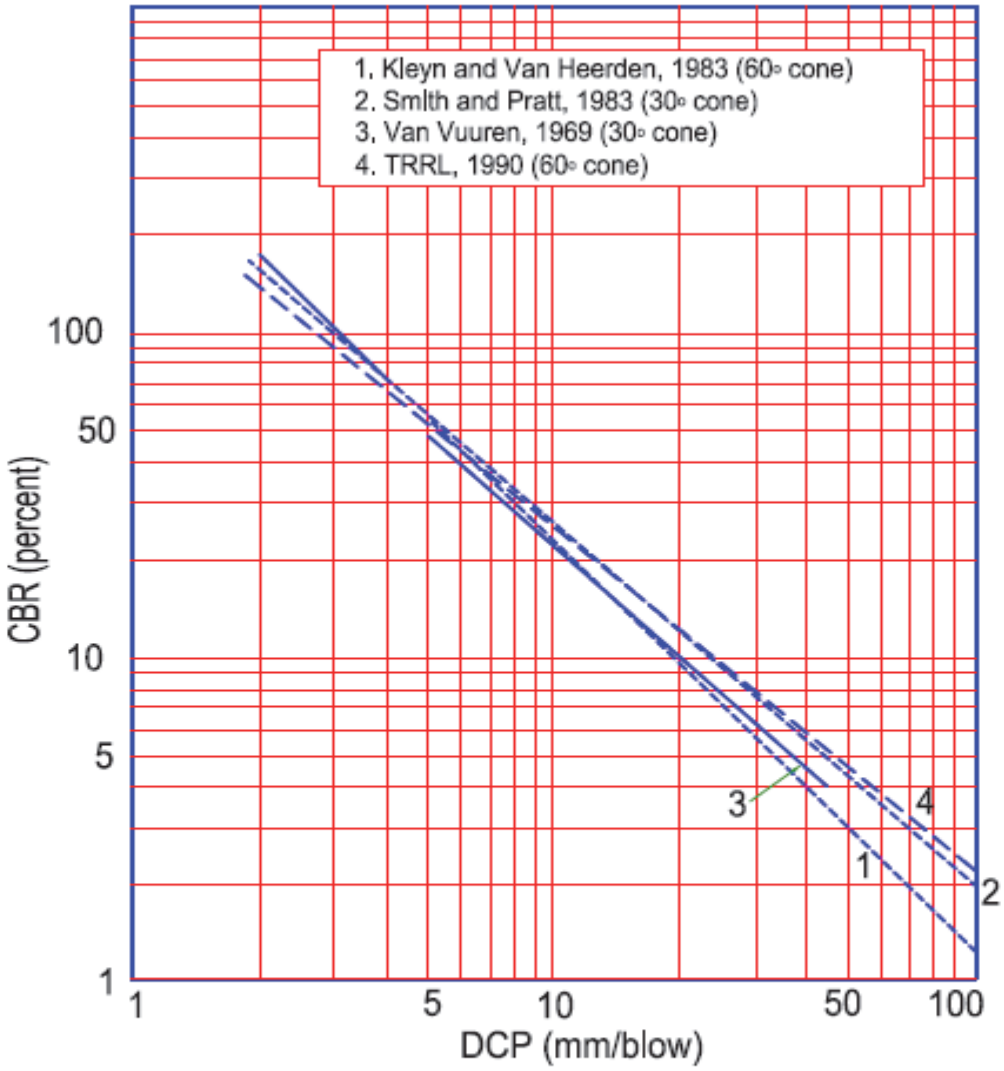


Figure B.3 Typical DCP test result



1. Log_{10} (CBR)=2,632-1,28Log_{10}(mm/blow)
2. Log_{10} (CBR)=2,555-1,145Log_{10}(mm/blow)
3. Log_{10} (CBR)=2.503-1.15Log_{10}(mm/blow)
4. Log_{10} (CBR)=2,48-1,057Log_{10}(mm/blow)

Figure B.4 DCP-CBR relationships

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Appendix C Roughness Measurements

C.1 Introduction

Vehicle operating costs increase as the roughness of the road pavement increases. Most road defects contribute in some way to increasing the roughness of the road pavement although, in its early stages, cracking may cause little or no change. However, without proper maintenance, the cracked surfacing deteriorates and the resulting potholes and subsequent patching cause a rapid increase in roughness. Surface texture and variability in rut depth also have a significant effect on the roughness of a road pavement.

The roughness of roads with similar pavement construction is a good measure of their relative pavement condition, but it does not identify the nature of the failures or their causes. However, if resources for the surface condition survey are limited, or if the sections of the road under investigation are very long, roughness and windscreen survey data can be used to establish those lengths of road having failures of differing severity. This allows representative lengths of road to be selected which can then be used to identify the cause or causes of deterioration.

The standard measure of road roughness is the International Roughness Index (IRI) which was developed during the International Road Roughness Experiment held in Brazil in 1982 (Sayers et al 1986). The IRI is a mathematical quarter car simulation of the motion of a vehicle at a speed of 80 km/hr over the measured profile and can be calculated directly from road levels measured at frequent intervals. The units of IRI are m/km. Devices for measuring levels are usually either slow and labour intensive or fast, automatic and expensive. Hence, the roughness of the road is usually measured using a Response Type Road Roughness Measuring System (RTRRMS) which must be periodically calibrated to allow the values of roughness to be reported in terms of IRI. An approximate guide to IRI is shown in Figure C.1.

The many methods for measuring road roughness in use throughout the world can be grouped into four generic classes on the basis of how accurately they measure the profile of the road and hence International Roughness Index (IRI).

Class 1 - Precision profiles
Class 2 - Other profilometric methods
Class 3 - IRI by correlation
Class 4 - Subjective ratings

Class 1 - Precision profiles

This class has the highest standard of accuracy. Class 1 methods are those which sample the vertical profile of the road at distances no greater than 250mm to an accuracy of 0.5mm for smooth roads. This profile is then used to compute the IRI. Class 1 methods are mainly used for the calibration and validation of other methods of roughness measurement. They can be used for relatively short sections where a high degree of accuracy is required but are not suitable for general roughness surveys. Examples of Class 1 methods include the rod and level survey, the TRL Profile Beam, the Face Dipstick and the ARRB Walking Profiler.

Class 2 - Other profilometric methods

This class includes all other methods in which the road profile is measured as the basis for direct computation of the IRI, but which are not capable of the accuracy and/or measurement interval specified for a Class 1 precision profile. This class includes most high-speed profilometers.

IRI range (mm/km)	Description
< 3.0	Ride comfortable over 120 km/h In range 1.3 – 1.8 undulations barely perceptible at 80 km/hr No depressions, potholes or corrugations noticeable; depressions < 2mm/3m High quality AC in range 1.4 – 2.3 High quality surface treatment 2.0 – 3.0
4 - 5	Ride comfortable up to 100 -120km/h At 80km/h moderately perceptible movements or large undulations may be felt Occasional depressions, patches or potholes (e.g. 5-15 mm/3m) or 10-20 mm/5m with frequency 1-2 per 50m or many shallow potholes (e.g. on surface treatment showing extensive fretting) Surface without corrugations or large undulations.
7 - 8	Ride comfortable up to 70 -90km/h, strongly perceptible movement and swaying. Usually associated with defects: frequent moderate and uneven depressions or patches (e.g. 15-20mm/3m or 20-40mm/5m with frequency 3-5 per 50m) or occasional potholes (e.g. 1-3 per 50m). Surface defects: strong undulations and corrugations
9 - 10	Ride comfortable up to 50 -60km/h, frequent sharp movements or swaying. Associated with severe defects: frequent deep and uneven depressions and patches (e.g. 20-40mm/3m or 40-80mm/5m with frequency 3-5 per 50m) or frequent potholes (e.g. 4-6 per 50m)
>11	Necessary to reduce speed below 50km/h. Many deep depressions, potholes and severe disintegration (e.g. 40-80 mm deep with frequency 8-16 per 50m)

Figure C.1 Road roughness estimation scale for paved roads with asphaltic concrete surfacing or surface dressing

Class 3 - IRI from correlation

Devices in this class measure roughness but need calibration to convert the data into units of IRI. The majority of road roughness data currently collected throughout the world are obtained with Response-Type Road Roughness Measuring Systems (RTRRMS). While these systems can take the form of towed trailers, such as the towed 5th wheel bump integrator, they more frequently involve instruments mounted in a survey vehicle. Examples of vehicle-mounted RTRRMS include the Bump Integrator unit, the NAASRA meter and the Mays meter. These instruments usually measure roughness in terms of the cumulative movement between the vehicle's axle and chassis when travelling along a road under standard conditions.

Also in this class is a low cost alternative, the Machine for Evaluating Roughness using Low-cost INstrumentation (or MERLIN for short) that can be used to both estimate IRI and also calibrate other RTRRMS. The MERLIN does not record the absolute profile but measures the mid-chord deviations over a predetermined base length for a section of road

and then relates a statistic from the frequency of those deviations to the IRI using a predetermined correlation.

The roughness values recorded by RTRRMS depend on the dynamics of the vehicle and the speed at which it is driven. The dynamic properties of each vehicle are unique and will also change with time, for example, as springs and shock absorbers wear. It is therefore essential that the roughness values obtained from a RTRRMS are converted to units of IRI by regularly calibrating it with a Class 1 or 2 device or the MERLIN.

Class 4 - Subjective rating

This class has the lowest standard of accuracy. It includes methods such as subjective evaluation involving rideability and visual assessment. This is illustrated in Figure C.1. It has been found that subjective estimates of IRI are prone to errors of up to 40 per cent for new observers and therefore this method should only be used when other methods are unavailable. Uncalibrated RTRRMS also fall into this category.

C.2 Operation of the Merlin

The MERLIN is suitable for measuring the roughness of short lengths of road such as experimental test sections or sections used to calibrate a vehicle mounted system (RTRRMS). Note that for long lengths of road a vehicle mounted system is more appropriate.

The MERLIN is shown in Figure C.2. It has a foot and a wheel, 1.8 metres apart, which rest on the road surface along the wheel path. A moveable probe rests on the road surface mid-way between the foot and the wheel. This probe measures the vertical distance, ‘y’, between the road surface and the centre point of an imaginary line joining the foot and the bottom of the wheel. The result is recorded on a data chart mounted on the machine. By recording measurements along the wheel path, a histogram of the ‘y’ values (amplified by a factor of approximately 10) can be built up on the chart. The width of this histogram can then be used to determine the IRI.

To determine the IRI, 200 measurements are usually made at regular intervals. For each measurement the position of the pointer on the chart, shown in Figure C.3, is marked by a cross in the box in line with the pointer and, to keep a count of the total number of measurements made, a cross is also put in the ‘tally box’ on the chart. When the 200 measurements have been made the position mid-way between the 10th and 11th crosses (counting in from one end of the distribution) is marked on the chart. The procedure is repeated for the other end of the distribution. The spacing between the two marks, D, is then measured in millimetres.

For earth, gravel, surfaced dressed and asphaltic concrete roads, the IRI can be determined using the following equation.

$$IRI = 0.593 + 0.0471\ D$$

This equation assumes that the MERLIN has a mechanical amplification factor of 10. In practice this may not be true because of small errors in manufacturing. Therefore before the MERLIN is used the amplification has to be checked and the value of D corrected. To do this the instrument is rested with the probe on a smooth surface and the position of the pointer carefully marked on the chart. The probe is then raised and a calibration block

approximately 6 mm thick placed under the probe. The new position of the pointer is marked. If the distance between the marks on the chart is S and the thickness of the block is T then measurements made on the chart should be multiplied by the scaling factor:

$$\text{Scaling factor} = \frac{10\ T}{S}$$

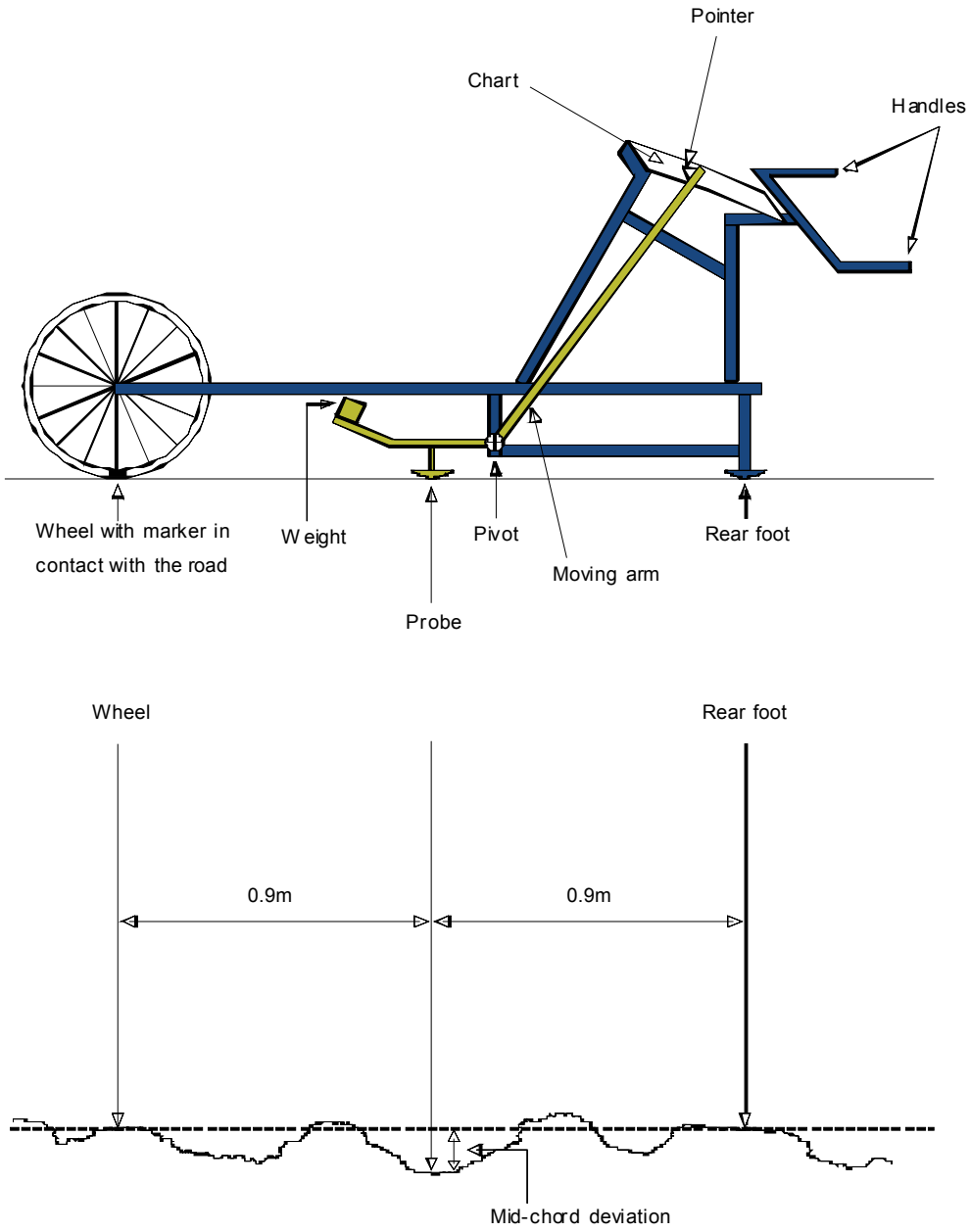


Fig C.2 Operation of the Merlin

The mechanical amplification factor (from actual probe movement to chart values) is usually about 10. When measuring a very rough road, this amplification may be too great and result in many readings going off the recording chart. To correct for this, the position of the probe can be moved to its alternative position on the MERLIN. Another set of holes to identify this location should already exist. At this alternative position, the amplification factor should be reduced to about 5, i.e. a greater number of readings will now be within

the chart boundary. A scaling factor should still be calculated and applied to the results, although the equation will need to be adjusted as follows:

$$\text{Scaling factor} = \frac{5\ T}{S}$$

C.2.1 Length of test section used in calibration

If 200 measurements (one at each wheel revolution) are taken using a MERLIN with a 680mm diameter wheel, then the length of the section surveyed will be 427 metres. For shorter or longer sections a different procedure will be required. The guiding principles are:

i) The test section should be a minimum of 200 metres long

ii) Approximately 200 readings must be taken per chart. With less than 200 readings the accuracy will decrease and with more the chart becomes cluttered. If the number of readings differs from 200, then the number of crosses counted in from each end of the distribution, to determine D, will also need to be changed. It should be 5% of the total crosses that is, 9 crosses for 180 readings, 11 for 220 readings etc.

iii) When taking measurements, a marker on the wheel should be used to indicate where to take the measurement, e.g. every time the marker is in contact with the road. This not only prevents errors due to any variation in radius of the wheel but also avoids operator bias.

iv) Regularly-spaced measurements should be taken over the full length of the test section. This gives the most representative result.

v) If taking repeat measurements along a section, try to avoid taking readings at the same points on different passes. E.g. start the second series of measurements half a metre from where the first series was started. For example, for a 210 metre test section the measurements could be made in two passes, taking one reading every revolution of the wheel, and off-setting the second pass by half a metre. Or the measurements can be made in one pass taking a measurement every half revolution of the wheel.

C.3 Road Roughness measurements using a RTRRMS

When roughness measurements are required on more than a few short sections of road, a RTRRMS is recommended. The main advantages of these types of system are their relative low cost and the high speed of data collection. The systems are capable of surveys at speeds up to 80 km/h so many hundreds of kilometres of road can be measured in a day.

The Bump Integrator (BI) Unit is a response-type road roughness measuring device that is mounted in a vehicle. The instrument measures the roughness in terms of the cumulative unidirectional movement between the rear axle and the chassis of a vehicle in motion. The BI system comprises a bump integrator unit, a counter unit with 2 displays, connection leads and an optional installation kit. The system is powered by the 12 volt battery of the vehicle.

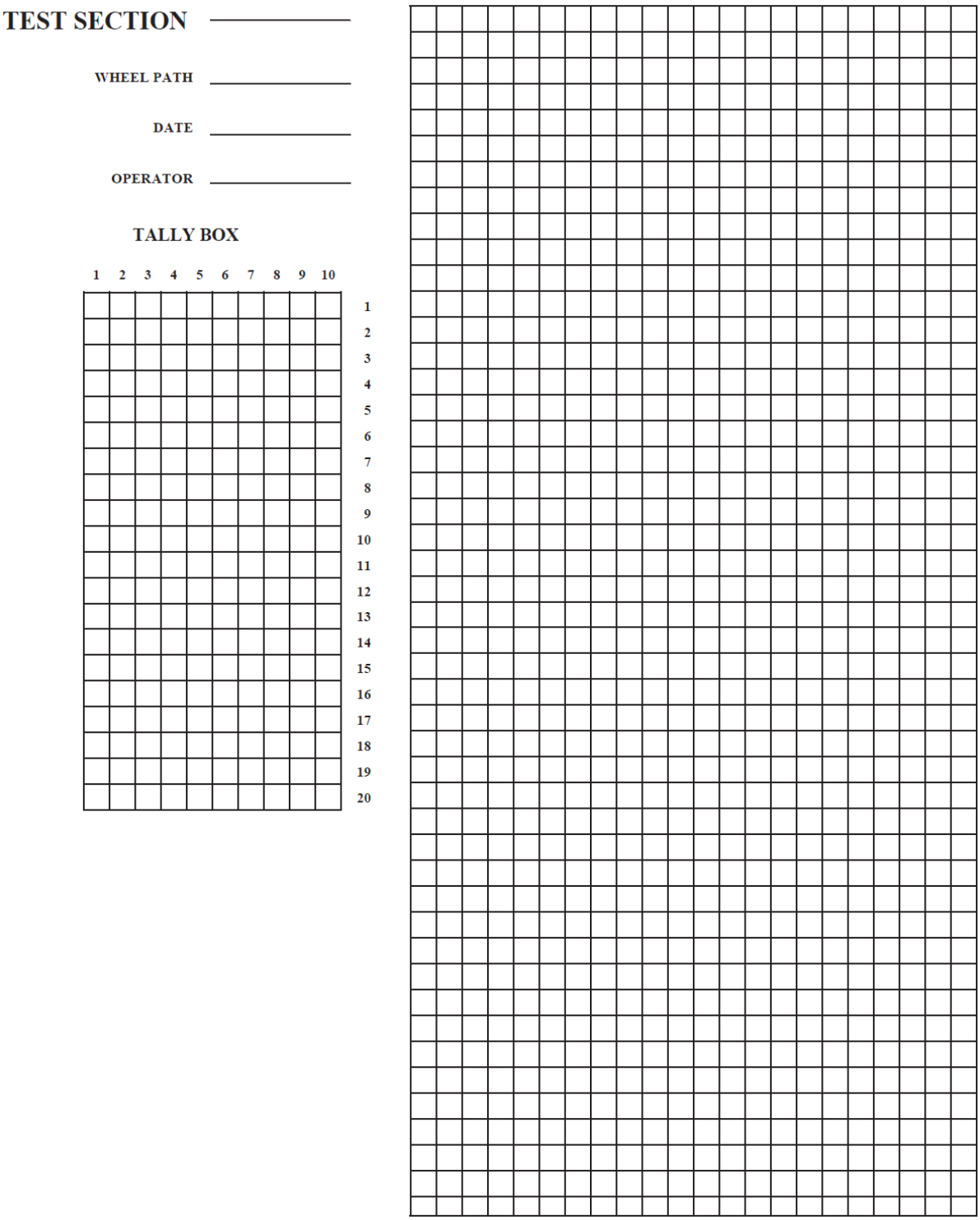


Figure C.3 Merlin data sheet

C.3.1 Fitting the BI unit

The BI unit is mounted in a rear-wheel drive vehicle as shown in Figure C.4. The unit is bolted to the rear floor pan of the vehicle directly above the centre of the rear axle. A 25mm hole needs to be cut in the floor pan and a bracket or hook fixed to the centre of the differential housing of the rear axle.

Before each survey, the flexible metal cord from the cylindrical drum of the BI unit is passed through the hole in the floor and hooked onto the bracket on the rear axle. This cord must not touch the sides of the hole. Tension in the cord is maintained by a return spring inside the drum of the BI unit. The BI unit measures the unidirectional movement, in centimetres, between the vehicle chassis and the axle as the vehicle is driven along the road. This is displayed on a counter box, usually fixed to the front passenger fascia.

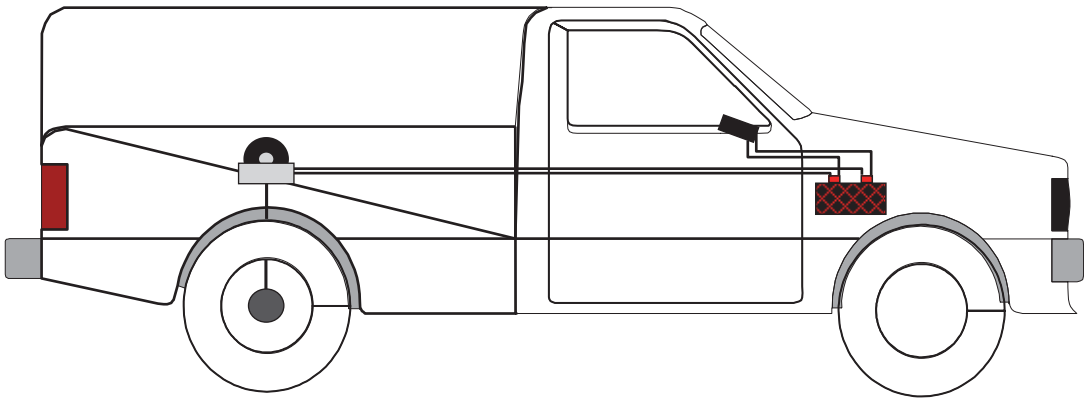


Figure C.4 Diagrammatical representation of the Integrator Unit fitted to a vehicle

C.3.2 Survey procedure

i) A safe working environment should be maintained at all times. Many organisations have on-site safety procedures which should be followed. As the vehicle may be moving slower than the majority of other traffic, it should be clearly signed and fitted with flashing lights.

ii) The vehicle should be well maintained and in good working order. The wheels should be properly balanced and the steering geometry correctly aligned. The tyres should not have flat spots or be unduly worn. Tyre pressures should be maintained precisely to the manufacturer's specifications. The load in the vehicle must be constant. Ideally the vehicle should contain only the driver and observer, and no other load should be carried.

iii) The engine and suspension system should be fully warmed-up before measurements commence. This can be achieved by driving the vehicle for at least 5km before measurements start.

iv) The tension cord from the BI unit to the axle should only be connected during the survey. At all other times the cord should be disconnected to stop unnecessary wear to the BI unit. When attaching the cord to the rear axle, the cord should be pre-tensioned by turning the BI pulley 2.5 turns anti-clockwise. The wire is then wound around the pulley 2 turns in the same direction as the arrow. Note: the pulley must

NOT be turned clockwise or suddenly released after being tensioned as the internal spring mechanism could be damaged.

v) When measurements are being taken the vehicle should normally be driven at constant speed, avoiding acceleration, deceleration and gear changes. This is necessary because the vehicle's response to a given profile varies with speed. To reduce reproducibility errors it is best to operate the RTRRMS at a standard speed of 80 km/h. However, if this speed is unsafe for reasons of traffic, pedestrians or restrictive road geometry, a lower speed of 50 or 32 km/h can be used. It is usual to use the same operating speed for all of the surveys. Calibration must be carried out at this operating speed.

vi) For general surveys, it is recommended that readings are recorded at half kilometre intervals. This distance should be measured with a precision odometer. The use of the vehicle odometer or kilometre posts is not recommended for survey purposes.

vii) There are two counters in the recording unit, connected by a changeover switch. This allows the observer to throw the switch at the end of each measurement interval so that the reading can be manually recorded while the other counter is working. The first counter can then be re-set to zero ready for the next changeover. Software is also available which automatically records the roughness data, vehicle speed and distances in spreadsheet form.

viii) The type of road surfacing should also be recorded to aid future analysis of the data. On completion of the survey, the wire cord should be disconnected from the rear axle.

ix) The counts measured by the BI are in units of cumulative centimetres of uni-directional movement of the rear axle. After the survey these should be converted to mm/km (the vehicle response roughness values, VR) using the following equation.

$$VR = \frac{\text{BI count} \times 10}{\text{Section length (km)}}$$

Where VR = Vehicle Response (mm/km)
BI = Number of counts per section (cm)

x) The vehicle response roughness values should then be converted to units of estimated IRI, E[IRI], using a calibration that is unique to the RTRRMS at that time. The results of a typical survey in terms of E[IRI] are shown in Figure C.5.

C.4 Calibration of a RTRRMS

The RTRRMS must be regularly calibrated against an instrument such as the MERLIN. This calibration should preferably be carried out before the survey and checked on ‘control’ sites during the survey period to ensure that the RTRRMS remains within calibration. The calibration of the RTRRMS will need to be re-checked before any subsequent surveys or after any part of the suspension of the vehicle is replaced.

The calibration exercise involves comparing the results from the RTRRMS and the calibration instrument over several short road sections. The relationship obtained by this comparison can then be used to convert RTRRMS survey results into units of E[IRI]. The recommended practice for roughness calibration is described below.

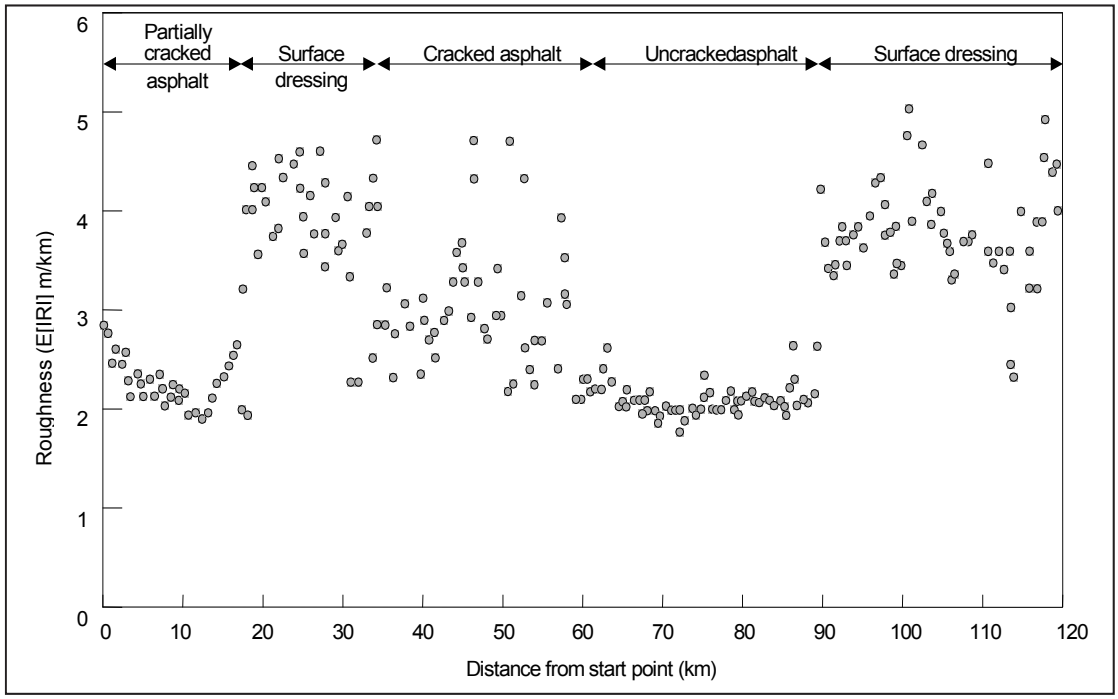


Figure C.5 An example of a roughness survey

i) A minimum of eight sections should be selected with varying roughness levels that span the range of roughness of the road network. The calibration sites should be on a similar type of road (i.e. paved or unpaved roads) to those being surveyed. The sections should have a minimum length of 200m and should be of uniform roughness over their length. In practice it may be difficult to find long homogeneous sections on very rough roads. In this case it is better to include a shorter section than to omit high roughness sites from the calibration. The sections should be straight and flat, with adequate run-up and slow-down lengths and should have no hazards such as junctions so that the vehicle can travel in a straight course at constant speed along the whole section.

ii) The roughness of each section should be measured by the RTRRMS at the same vehicle speed that is to be used for the general survey. The value of VR (mm/km) should be the mean value of at least three test runs.

iii) The calibration instrument should measure roughness in both wheel paths. The average of these IRI values (in m/km) is then plotted against the vehicle response for each of the test sections. The calibration equation for the RTRRMS is then derived by calculating the best fit line for the points. This relationship generally has a quadratic form but has also been found to be logarithmic depending upon the characteristics of the vehicles suspension and the levels of roughness over which the RTRRMS is being calibrated.

$$E[IRI] = a + b\,VR + c\,VR^2$$

Where E[IRI] = Estimated IRI (m/km)
VR = Vehicle Response (mm/km)
a, b and c = constants

The calibration equation can then be used to convert data from the RTRRMS into units of E[IRI]. An example of a calibration curve is shown in Figure C.6.

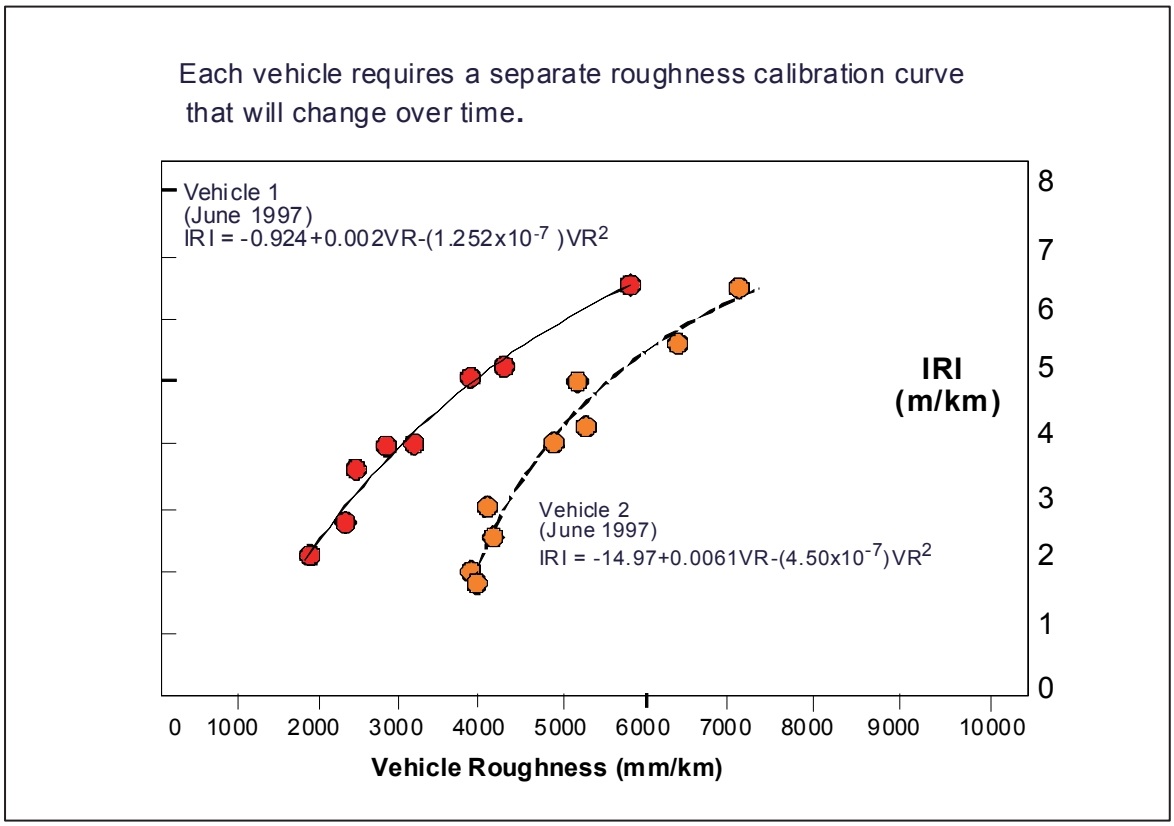


Figure C.6 Example of an RTRRMS calibration

Appendix D Deflection Measurements

D.1 Introduction

The structural integrity of a pavement can be quickly and efficiently assessed by applying a load to the pavement surface and measuring the resulting deflection. The numerous pavement deflection measurement techniques currently in use can be categorised according to the applied load characteristics. Measuring the pavement surface deflection under a static or slow moving load (Benkelman Beam, Deflectograph) represents the first generation approach. The next generation involved the application of a dynamic vibratory load (Road Rater and Dynaflect). The third generation deflection equipment (Falling Weight Deflectometer) simulates the effect of a moving wheel load by applying a dynamic impulse load. Future equipment will attempt to measure deflections caused by an actual wheel load moving at highway speeds.

The two most common techniques used to measure pavement deflections are the Benkelman Beam and the Falling Weight Deflectometer. This Appendix describes the equipment, procedures for their use and guidance on the interpretation of the results.

D.2 Deflection beam (Benkelman beam)

D.2.1 General

This is the least expensive instrument for measuring deflections, originally devised by A C Benkelman. It is a mechanical device that measures the maximum deflection of a road pavement under the dual rear wheels of a slowly moving loaded lorry. The beam consists of a slender pivoted beam, approximately 3.7m long, supported in a low frame that rests on the road. The frame is fitted with a dial gauge for registering the movement at one end of the pivoted beam, the other end of which rests on the surface of the road. It is shown in Figure D.1.

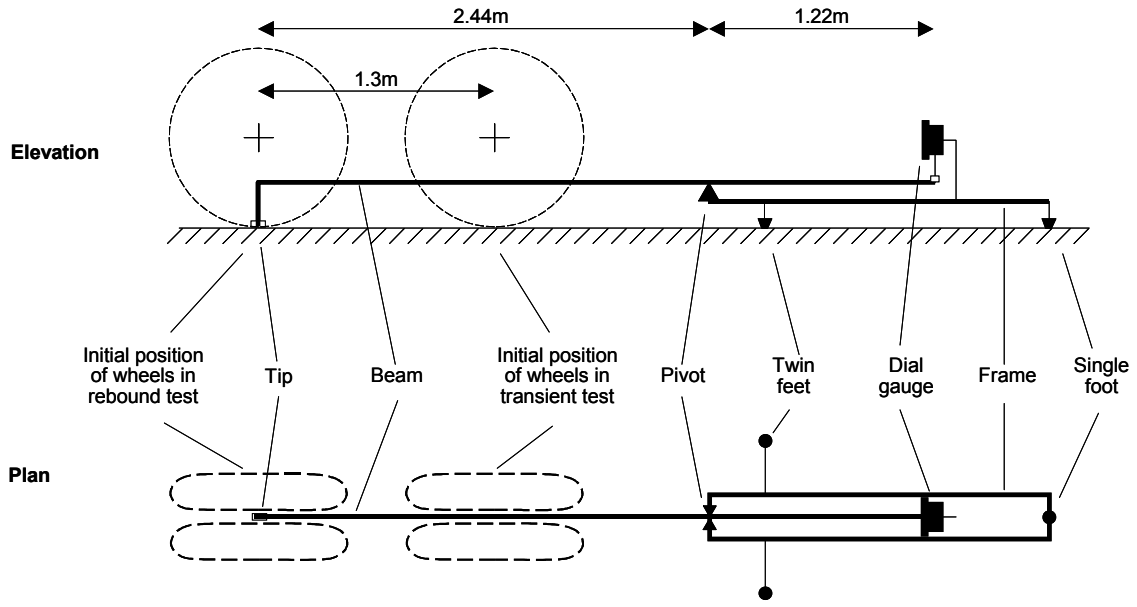


Figure D.1 Diagrammatic representation of the Benkelman beam

D.2.2 Deflection beam survey procedure

Deflection testing can be carried out by only taking measurements in the outer wheel path because this usually is the weakest since it has less lateral support and higher moisture contents in the lower granular layers as compared to the inner wheel path. However, if it is not evident that deflections measured in the outer wheel path are consistently higher than in the inner wheel path, deflection beam measurements should be made in both wheel paths of the slow lane on dual carriageways and in both lanes of a single carriageway road. Doing deflection tests in both wheel paths does not significantly increase testing time as both wheel paths are tested simultaneously.

Tests can be made at any frequency along the road, but when measurements are needed at closely spaced regular intervals (say 10 or 25 metres) it is generally more cost effective to use one of the more sophisticated deflection devices such as the FWD or Deflectograph.

When using manual deflection beam measurements, it is recommended that the following strategy is adopted.

i) Tests are carried out on a basic pattern of 50 or 100 metre spacing.

ii) Additional tests should be undertaken on any areas showing atypical surface distress.

iii) When a deflection value indicates the need for a significantly thicker overlay than is required for the adjacent section, the length of road involved should be determined by additional tests.

D.2.3 Timing of deflection surveys

In some cases the moisture content of the road pavement, especially the subgrade, changes seasonally. In these circumstances the tests should be carried out towards the end of the rainy season, when the road is at its weakest, and the deflection is at its greatest.

D.2.4 Details of test truck

The truck must have dual rear wheels and should be loaded to a standard rear axle load if possible. The axle load must in any case be recorded because load-related corrections to readings may be required. A number of authorities recommend the use of an 80 kN load on the rear axle (a standard axle), although TRL recommends the use of a 63.2 kN rear axle load. Over this range of loads the maximum deflection is usually linearly related to the applied load. Therefore deflection values can be measured at higher loads on structurally adequate pavements where over-stressing is not a danger, and then normalised to a standard load for comparison purposes. It is important that the test method and test conditions must be compatible with the deflection criteria and design procedures adopted.

D.2.5 Test method

There are two basic test methods commonly in use for the deflection beam. These are the *transient* deflection test and the *rebound* test.

D.2.6 Transient deflection test

In this test the tip of the beam is inserted between the dual rear-wheel assembly of the loaded truck. The dial gauge is set to zero and the truck then drives slowly forward. As the wheels approach the tip of the beam, the road surface deflects downwards (loading deflection) and the movement is registered by the dial gauge. As the wheels move away

from the tip of the beam, the road surface recovers (recovery deflection) and the dial gauge reading returns to approximately zero. The test procedure is summarised below.

i) Mark the point, in the outer wheel path, at which the deflection is to be measured and position the lorry so that the rear wheels are 1.3m behind the marked point.

ii) Insert the deflection beam between the twin rear wheels until its measuring tip rests on the marked point. If deflections are to be measured in both wheel paths, insert a second beam between the offside wheels. It is helpful in positioning the lorry and aligning the beams if a pointer is fixed to the lorry 1.3m in front of each pair of rear wheels.

iii) Adjust the foot screws on the frame of the beam to ensure that the frame is level transversely and that the pivoted arm is free to move. Adjust the dial gauge to zero and turn the buzzer on. Record the dial gauge reading which should be zero or some small positive or negative number.

iv) The maximum and final reading of the dial gauge should be recorded while the lorry is driven slowly forward to a point at least 5m in front of the marked point. The buzzer should remain on until the final reading is taken. Care must be taken to ensure that a wheel does not touch the beam. If it does, the test should be repeated.

v) The transient deflection is the average of the loading and recovery deflections. Because of the 2:1 ratio of the beam geometry over the pivot point (see Figure D.1) the transient deflection is calculated by either:

- adding the difference between initial and maximum dial gauge readings to the difference between maximum and final dial gauge readings, or,
- calculating the loading deflection (double the difference between the initial and maximum values) and the recovery deflection (double the difference between the maximum and final readings) and then calculating the mean of the two deflections.

Note that not all commercially available beams have a 2:1 ratio and this therefore needs to be taken into account. [At least one manufacturer has also supplied dial gauges that read the correct deflection (i.e. their scales already incorporate the scaling factor). Such dial gauges are dangerous because they could easily be used for another purpose and then they will give the wrong answer.]

At least two tests should be carried out at each chainage and the mean value used to represent the transient test result. If the results of the two tests do not fall within the repeatability limits described in Table D.1 then a third test should be carried out.

Table D.1: Repeatability of duplicate transient deflection tests

Mean deflection (mm)	Maximum permissible difference between the two tests (mm)
< 0.10	0.02
0.10 - 0.30	0.03
0.31 - 0.50	0.04
0.51 - 1.00	0.05
> 1.00	0.06

D.2.7 Rebound deflection test

This is probably the most commonly used method which, while not as comprehensive as the transient method, allows a greater production rate with less need for repeat measurements (e.g. due to the tyre touching the beam when guide pointers are not used on the lorry). Because the rebound deflection can be influenced by the length of time during which the loading wheels are stationary over the test point care must be taken over the exact procedure used. For example if it is being used with deflection criteria developed using rebound procedures, the deflection test procedure should be identical to that used during the development of the designs curves.

For the rebound deflection test, the dual wheels are positioned immediately above the test point and the measuring tip of the beam is placed on the test point and between the dual wheels. The beam is adjusted in the same way as for the transient test and, after the initial reading has been noted, the lorry is driven forward at creep speed until the wheels are far enough away to have no influence upon the deflection beam. The final dial gauge reading is recorded and the rebound deflection is twice the difference between the initial and final dial gauge readings.

Whichever method is adopted for the deflection beam measurements, the possible effect of plastic flow upon the results should be noted, although this is only likely to be significant for thicker and relatively fresh asphalts. When an asphalt surfacing material flows plastically, it squeezes upwards between the dual loading wheels of the deflection truck. In the transient deflection test this reduces the transient loading deflection because the upward movement of the material counteracts the downward movement of the pavement. The transient recovery deflection that is measured may be correct but further plastic movement of the raised surfacing material can occur during the time taken for the wheels to move from the test point to the final position. This causes an error in the recovery deflection reading. It is usually very clear from the test results when plastic flow occurs and testing should be stopped to avoid recording erroneous data.

In the rebound test greater plastic flow will be induced in susceptible materials because of the time the wheels remain stationary over the test point. When the truck is driven forward the road surface 'rebounds' but an indeterminate amount of recovery of the displaced surfacing material can occur. There is thus no clear indication from the simple rebound test when plastic flow occurs and this is the primary reason why it is not recommended by TRL for countries with tropical climates.

D.2.8 Analysis of deflection survey data

Deflection readings can be affected by a number of factors which should be taken into account before the results can be interpreted. These are the temperature of the road, plastic flow of the surfacing between the loading wheels, seasonal effects and the size of the deflection bowl.

D.2.8.1 Road temperature

The stiffness of asphalt surfacings will change with temperature and therefore the magnitude of deflection can also change. The temperature of the bituminous surfacing is recorded when the deflection measurement is taken, thus allowing the value of deflection to be corrected to a standard temperature. It is recommended that 35^{o}C, measured at a

depth of 40mm in the surfacing, is a suitable standard temperature. Fortunately, it is often found that little or no correction is required when the road surfacing is either old and age hardened, or relatively thin.

It is not possible to produce general correction curves to cover all roads, it is therefore necessary to establish the deflection/temperature relationship for each project. This is obtained by recording the change in deflection on a range of test points as the surfacing temperature rises from early morning to midday.

D.2.8.2 Seasonal effects

In areas where the moisture content of the subgrade changes seasonally, the deflection will also change. For overlay design purposes, it is usual to use values that are representative of the most adverse seasonal conditions and it is therefore normal practice to carry out surveys either immediately after the rainy season or towards its end. If this cannot be done, an attempt should be made to correct for the seasonal effect. However, this requires a considerable data bank of deflection results and rainfall records before reliable corrections can be made.

D.2.8.3 Size of deflection bowl

The size of the deflection bowl can occasionally be so large that the front feet of the deflection beam lie within the bowl at the beginning of the transient deflection test. If this happens, the loading and recovery deflection will differ. The simplest way to check whether the differences in loading and recovery deflection are caused by the size of the bowl is to place the tip of another beam close to the front feet of the measurement beam at the beginning of the transient test. This second beam can be used to measure any subsequent movement of the feet of the first beam as the truck moves forward. If feet movements larger than 0.06mm are observed only the recovery part of the deflection cycle should be used for estimating the value of transient deflection.

D.2.8.4 Data processing

After all measurements have been made, and any corrections applied to the raw data, it is then convenient to plot the deflection profile of the road for each lane (a performance chart). When measurements in both wheel paths have been made, only the larger deflection of either wheel path at each chainage is used. Any areas showing exceptionally high deflections that may need reconstruction or special treatment can then be identified.

The deflection profile is used to divide the road into homogeneous sections in such a way as to minimise variations in deflections within each section. The minimum length of these sections should be compatible with the frequency of thickness adjustments that can sensibly be made by the paving machine, whilst still maintaining satisfactory finished levels. When selecting the sections the topography, subgrade type, pavement construction and maintenance history should all be considered (see Chapter 7).

A number of statistical techniques can be used to divide deflection data into homogeneous sections. The recommended technique is the cumulative sum method, where plots of the cumulative sums of deviations from the mean deflection against chainage can be used to discern the sections.

D.3 Falling Weight Deflectometer (FWD)

D.3.1 General

The Falling Weight Deflectometer (FWD) simulates the effect of actual traffic-induced loads by dropping a constant weight onto the pavement surface from variable heights. A diagram is shown in Figure D.2. The FWD is generally placed on a semi-trailer and equipped with its own power source (generator / batteries). It weighs about 1 tonne (1.5 tonne for the larger airport version) and can comfortably travel on surfaced roads at 100 km/h. A distance measuring wheel is also attached to the semi-trailer and is used to locate the appropriate test chainages during surveys.

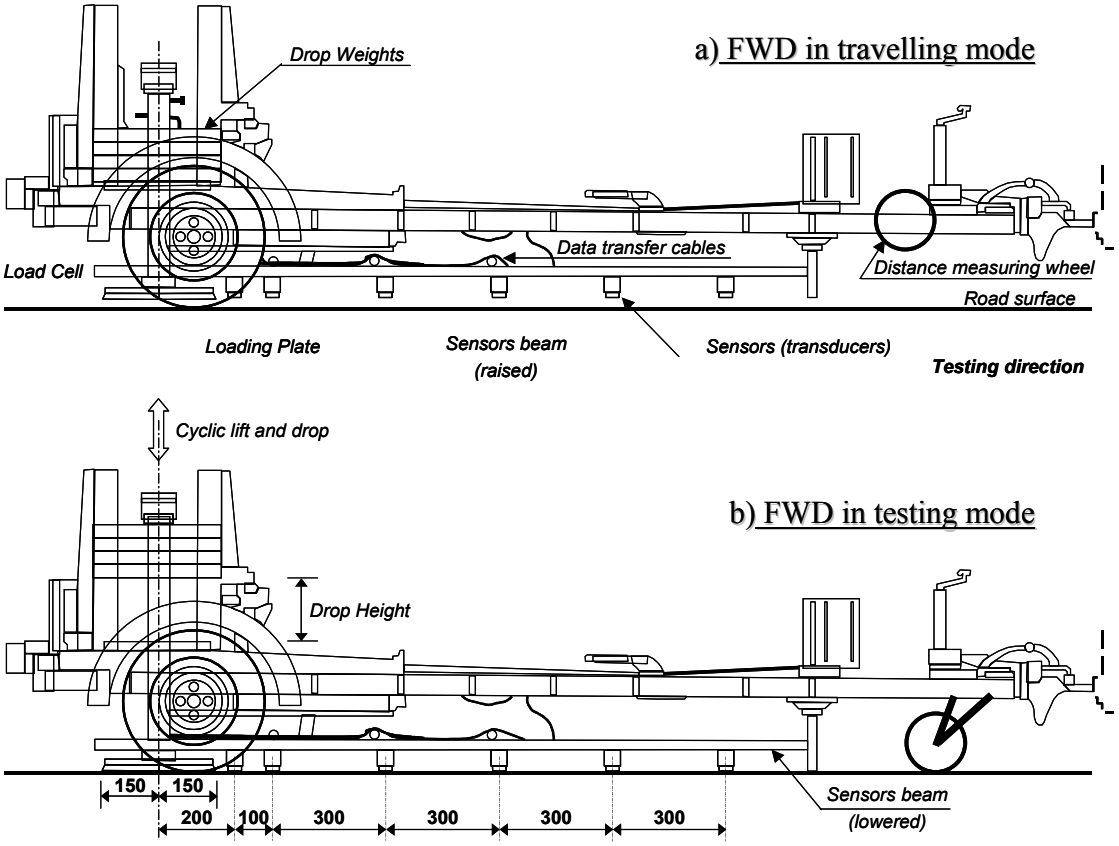


Figure D.2: Diagrammatic representation of the FWD

A number of detachable weights are locked on a hydraulic piston which facilitates their quick and precise lift. The weights are thereafter dropped from a predetermined height. A circular, flexible, loading plate (150 mm radius) ensures the smooth load transfer between the dropping weights and the potentially uneven pavement surface. A load cell, placed directly under the dropping weight, accurately measures the loading level. The resultant pavement surface deflections are measured by 7 sensors / transducers placed under a sensors beam at the offsets shown in Table D.2. Multiple data transfer cables, also attached to the sensors beam, ensure the communication between the load cell / sensors / FWD engines and the central computer.

D.3.2 Measurement procedure

Establishing a concise but clear and consistent testing reference system *prior to the commencement of testing* is critical. The reference system should include the following:

a) General information: Date, operator(s), FWD serial number, road ID (for network testing), measurement units (metric / imperial), test start and end chainages, test spacing (distance between adjacent test points); sensors spacing (depending on the pavement layer thicknesses);

b) Test point information and parameters: Number and sequence of drops (in terms of corresponding load levels); air, surface and in-depth temperatures; pavement cracking type, extent and magnitude; road profile (e.g. fill/cut, to reflect potential water ingress); change(s) in the pavement structure; and underground structures (e.g. culverts, pipes, which can significantly affect the deflection magnitude).

The number and sequence of drops can be set up differently in up to five series. The operator can apply any or all of these series at a test point. Generally, one series of two drops (4 tonne each) is usually applied for all test points.

When the road has an asphalt surfacing, the deflection may change as the temperature of the surfacing changes. Also, when the deflection bowl is to be used to estimate pavement layer moduli, the stiffness of the asphalt surfacing will need to be corrected to a standard temperature. It is therefore necessary to measure the temperature of the surfacing during testing. In temperate climates measurements taken hourly may be sufficient, however, in tropical climates the pavement temperature will rise quickly during mid-morning and can reach a temperature at which the asphalt surfacing is liable to plastic deformation during testing. This must be carefully monitored and temperature measurements at this critical time of the day may need to be taken every 15 or 20 minutes. Where the asphalt surfacing is less than 150mm thick the temperature should be measured at a depth of 40mm. When the surfacing exceeds 150mm, it is recommended that temperatures should be recorded at two depths, 40 and 100mm.

All relevant calibrations (see next section) must be undertaken as required. A large amount of deflection data could prove incorrect and, therefore, useless should the system malfunction at any time.

While moving between two adjacent points the sensors beam must be raised, irrespective of travelling speed. Once the FWD has stopped, the sensors beam is lowered together with the loading plate. The operator inputs the test point information and, automatically, the weights are raised and dropped from a “test” height for an in-built, on-the-spot system check. Once the operator is satisfied with the system pre-test data, the weights are automatically raised and dropped to and from the predetermined height(s) as many times as required. After each drop, the relevant data is sent to the computer which displays it. The operator can interrupt the automatic testing sequence at any time and restart and /or continue it manually (drop by drop) if necessary.

D.3.3 Calibrations

Three types of calibration of the sensors are done, namely absolute, reference and relative. Absolute calibration is done in the factory at the time of manufacture, while reference calibration is typically undertaken annually, also indoors, by designated agents. The absolute and reference calibration results should be recorded by the agents in calibration certificates and be available for inspection at all times.

The relative calibration is usually done monthly and/or at the start of every new project, in approximately 4 hours. During this calibration, the sensors are placed one on top of each other and subjected to a standard vertical load. If all the sensors are in good condition, their readings should be almost exactly equal.

The load cell should be tested at the start and end of each testing session by plotting, on the computer screen, its output curve, for a standard drop. This plotting option is available on most FWD equipment. If the load cell is in good condition, its output curve shall have a continuous sinusoidal shape.

Generally, no other calibration is required, even when the equipment has to travel on rough roads.

D.3.4 Output

The testing output is stored in specific text files in the computer and can be downloaded to an office computer for analysis.

D.3.5 Deflection bowl parameters

FWD deflection data may also be tabulated and plotted to show variation of pavement response along the road. Certain parts of the deflection bowl are influenced by the different pavement layers. With reference to Table D.2, the chosen deflection criteria are usually d_1, d_6 and d_1-d_4.

Table D.2 Recommended sensor positions

Flexible pavement	Distance from centre of load (mm)						
	1	2	3	4	5	6	7
Thick asphalt surfacing	0	300	600	900	1200[1]	1500	2100
Thin asphalt surfacing or seal	0	200[1]	300	600	900	1500	1800

Note 1. Where only 6 sensors are available these positions will be omitted.

The maximum deflection d_1 gives an indication of overall pavement performance whilst the deflection difference (d_1-d_4) relates to the condition of the bound pavement layers. Deflection d_6 is an indicator of subgrade condition. A typical deflection profile is shown in Figure D.3. Although the actual values of deflection will depend on the type and condition of the pavement layers, such plots show relative differences in their condition and give an indication of any structural weaknesses.

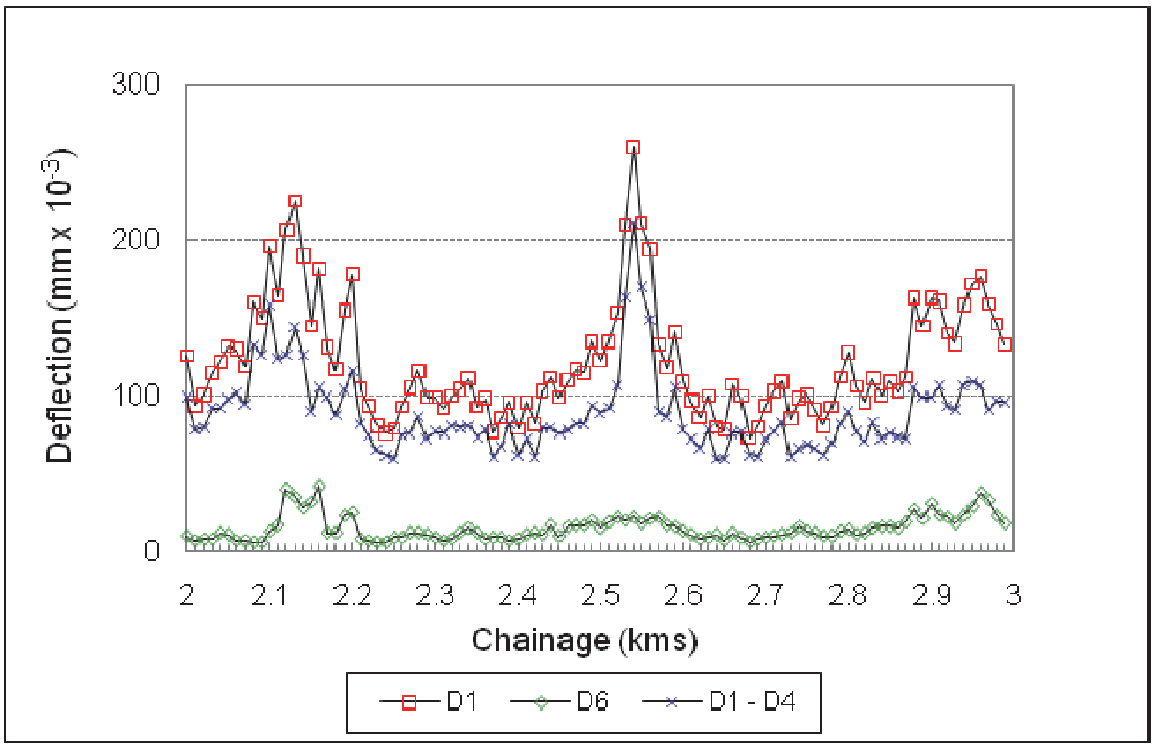


Figure D.3 FWD Deflection Profile

D.3.6 Back-calculation

Analysis of deflection bowl data is dependent on a suitable model to calculate the response of the pavement to the applied load. Most analysis programs are based on the assumption that the pavement behaves like a multi-layer structure made up of linearly elastic layers. Using such a model, it is possible to calculate the effective elastic modulus of each pavement layer from knowledge of the shape of the deflection bowl. This 'back-analysis' procedure requires accurate deflection data extending from the central maximum deflection out to deflection values at radial offsets of as much as 2.5 metres.

However, the linear elastic model is a very simple model of road pavements. Road materials display a variety of properties that do not comply with the assumptions of the model. For example, the elastic modulus of unbound materials is not a constant but depends on the stresses to which the material is subjected at each point in the structure, i.e. the materials are not linear. This is a particular problem with the subgrade because the modulus of the subgrade has a strong influence on the shape of the entire deflection bowl. Errors or inaccuracies in the assumptions here, give rise to errors in the calculations of the moduli of all other layers. *This is a very common problem and engineers should view the results of back calculation programs with great care.*

A further consideration is the capability of the programs to handle complex structures. The more layers that are present, the more difficult it is for the programs to identify a suitable unique solution. Overall, the acceptability of the results often depends much more on the skill of the analyst than the sophistication of the analysis program. Research during the Strategic Highway Research Programme in America has resulted in a set of rules and guidelines that can be used when estimating pavement layer moduli by back-calculation from deflection bowl data. These provide a reasonable basis for the back-analysis of road

pavements (FHWA-RD-01-113 (2002). *Back-calculation of layer parameters for LTPP test sections. Volume II: Layered elastic analysis for flexible and rigid pavements*).

D.4 Rigid pavements

Deflection tests are used to measure the degree of load transfer across joints to assess their condition. The Benkelman Beam should be positioned on the shoulder as shown in Figure D.4.

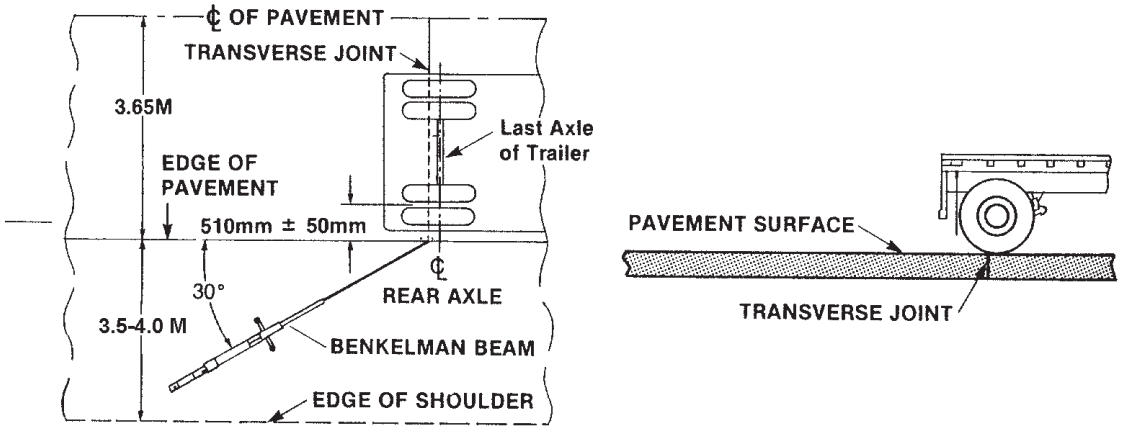


Figure D.4 Position of vehicle and Benkelman Beam for making static rebound deflection measurements on rigid pavements.

Measurements should be made on either side of the joint or crack to determine the deflection on either side of the joint. The ratio is a measure of the load transfer efficiency and should be taken during the most severe environmental circumstances, e.g. at the end of a wet season. A suggested minimum spacing is 100 m, with more closely spaced sample locations where cut and fill sections alternate rapidly.

Appendix E Skid Resistance

When the detailed surface condition survey indicates that the surfacing has poor texture or polished aggregate then a quantitative survey will usually be required. This survey can be dispensed with if the road is suffering from other failures that require the road to be resurfaced.

E.1 Texture Depth measurements

The texture depth of bituminous surfacings is measured by the sand patch test (BSI, 1990). There are also other relatively low cost instruments, such as the Mini-Texture Meter, which give continuous measurements of surface texture and are quicker and more convenient to use. However, the results from texture meters need to be calibrated against the sand patch test if they are to be compared with specifications. The sand patch test gives a single value of texture at one point and therefore a number of tests are needed to give a representative value for the road. This is done by selecting sections of road, 50 metres long, which cover the range of severity of the defect recorded during the detailed condition survey. A mean of ten tests, usually in the verge side wheel path, should be used to characterise each 50 metre section. Sections should also be chosen in hazardous areas such as the approaches to and crowns of bends. These values can then be compared to national standards, where these have been established, to identify the lengths of the road that need resurfacing.

E.1.1 Sand Patch test

The sand patch test is described in detail in BS 598 Part 105 (1990). The method is summarised below.

Apparatus

i) Plastic cylinder of 25 or 50ml volume. For surfacings having a texture depth of more than 1mm (e.g. surface dressings), 50ml of sand should be used. On AC surfacings where the texture will generally be less than 1mm the volume of sand should be reduced to 25ml or less.

ii) A spreader disc comprising a flat wooden disc 64mm in diameter with a hard rubber disc 1.5mm thick, stuck to one face. The reverse face is provided with a handle.

iii) Sand of a natural dry type, with rounded particle shape, complying with the grading given in Table E.1.

Table E.1 Grading of sand

BS test sieve (mm)	% by mass passing
0.600	100
0.300	90 - 100
0.150	0 - 15

Procedure

i) Dry the surface to be measured and, if necessary, sweep clean with a brush.

ii) Fill the cylinder with sand and, taking care not to compact it, and strike off the sand level with the top of the cylinder.

iii) Pour the sand into a heap on the surface to be tested, and spread the sand over the surface, working the disc with its face kept flat, in a circular motion so that the sand is spread into a circular patch. The patch should be of the largest diameter which results in the surface depressions just being filled with sand to a level of the peaks.

iv) Measure the diameter of the sand patch to the nearest 2mm at four diameters every 45^o and calculate the mean diameter (D) to the nearest 1mm.

v) Calculate the texture depth to the nearest 0.01mm from the following equation

$$\text{Texture depth (mm)} = \frac{1000.\ \text{Volume of sand (ml)}}{\text{Area of patch (mm}^2\text{)}}.$$

E.1.2 Standards

If national standards do not exist then the intervention values proposed in the UK may be used as a guide. These are given in Table E.2.

Table E.2 Texture depth standards in the UK

Definition	Texture depth (mm)	
	Bituminous	Concrete
Sound - no visible distress	>1	>0.5
Visible distress. (low level of concern) The distress is not serious and needs no action unless it extends over long lengths or can be related to excessive wet skidding accidents.	0.5 - 1.0	0.25 - 0.5
Extensive distress (warning level of concern). The distress is becoming serious and the incidence of accidents needs to be investigated.	<0.5	<0.25

E.2 Portable skid resistance tester

The micro-texture in terms of the ‘skid resistance' value (SRV) can be measured using the portable skid-resistance tester (RRL, 1969) (ASTM, E 303-93). There are other instruments available which measure skid resistance more rapidly (and more continuously), for example SCRIM (Sideways-force Coefficient Routine Investigation Machine) but these are relatively expensive.

A representative value of SRV can be obtained in a similar way to that described for texture depth, with the mean value of ten results being used to characterise a 50-metre section of road. These values can then be compared to national standards, where they have been established, to identify the lengths of the road in need of resurfacing.

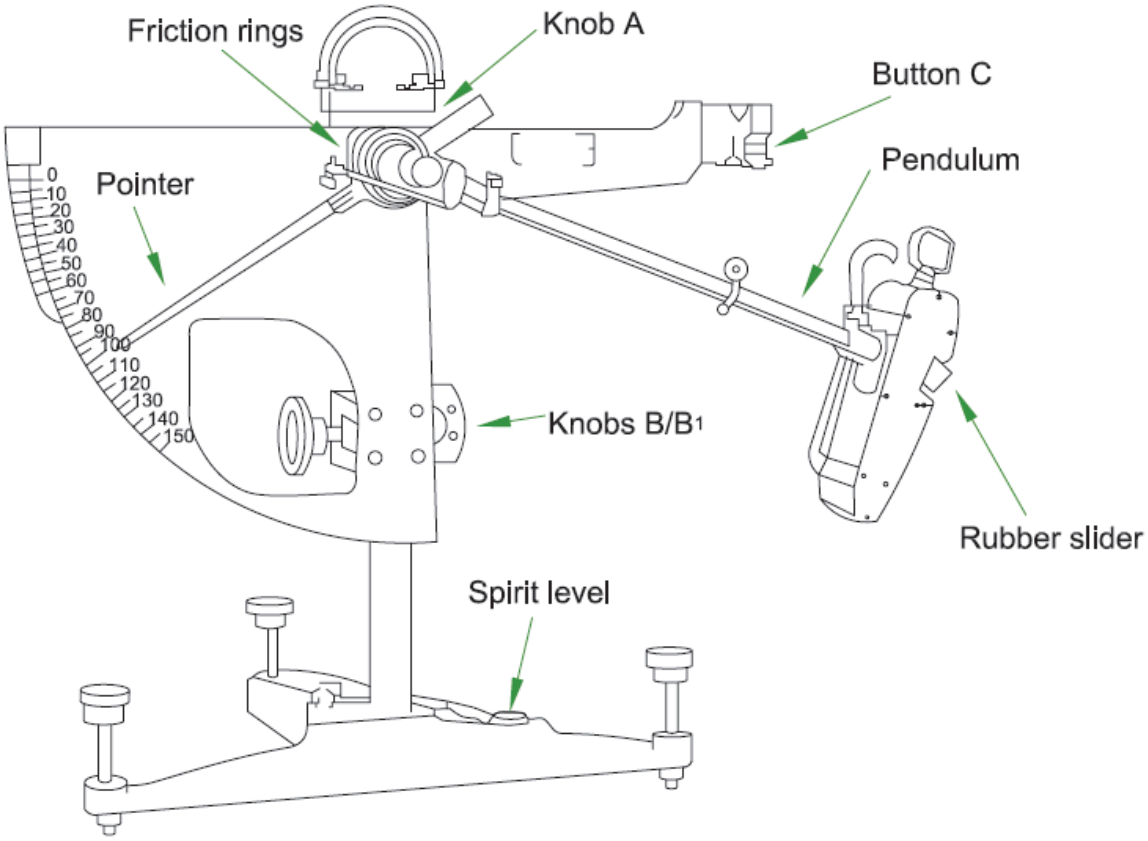


Figure E.1 Portable skid-resistance tester

E.2.1 Test method

The portable skid-resistance tester, shown in Figure E.1, was developed by the Transport Road Research Laboratory and is described in detail in Road Note 27 (TRRL, 1969). The testing procedure is summarised below.

Setting the tester

i) Set the base level using the in-built spirit level and the three levelling screws on the base-frame.

ii) Raise the head so that the pendulum arm swings clear of the surface. Movement of the head of the tester, which carries the swinging arm, graduated scale, pointer and release mechanism is controlled by a rack and pinion on the rear of the vertical column. After unclamping the locking knob A at the rear of the column, the head may be raised or lowered by turning either of the knobs B/B^1. When the required height is obtained, the head unit must be locked in position by using the clamping knob A.

iii) Check the zero reading. This is done by first raising the swinging arm to the horizontal release position, on the right-hand side of the tester. In this position it is automatically locked in the release catch. The pointer is then brought round to its stop in line with the pendulum arm. The pendulum is arm is released by pressing button C. The pointer is carried with the pendulum arm on the forward swing only. Catch the pendulum on its return swing, and note the pointer reading. Correct the zero setting as necessary by adjusting the friction rings.

iv) With the pendulum arm free, and hanging vertically, place the spacer, which is attached to the base of the vertical column, under the lifting-handle setting-screw to raise the slider. Lower the head of the tester, using knob B, until the slider just touches the road surface, and clamp in position with knob A. Remove the spacer.

v) Check the sliding length of the rubber slider over the surface under test, *by gently lowering the pendulum arm* until the slider just touches the surface first one side and then the other side of the vertical. The sliding length is the distance between the two extremities where the sliding edge of the rubber touches the test surface. To prevent undue wear of the slider when moving the pendulum arm through the arc of contact, the slider should be raised off the road surface by means of the lifting handle. If necessary, adjust to the correct length by raising or lowering the head slightly. When the apparatus is set correctly the sliding length should be between 125 and 127mm as indicated by the measure provided. Place the pendulum arm in its locked position. The apparatus is now ready for testing.

Operation of the tester

i) After ensuring that the road surface is free from loose grit, wet both the surface of the road and the slider.

ii) Bring the pointer round to its stop. Release the pendulum arm by pressing button C *and catch it on the return swing, before the slider strikes the road surface*. Record the indicated value.

iii) Return the arm and pointer to the locked position, keeping the slider clear of the road surface by means of the lifting handle. Repeat the process, spreading water over the contact area with the hand or brush between each swing. Record the mean of five successive swings, provided they do not differ by more than three units. If the range is greater than this, repeat swings until three successive readings are constant; record this value.

iv) Raise the head of the tester so that it swings clear of the surface again and check the free swing for any zero error.

v) Sliders should be renewed when the sliding edge becomes burred or rounded. One slider edge can usually be used for at least 100 tests (500 swings). New sliders should be roughened before use by swinging several times over a dry piece of road.

Temperature correction

The effect of temperature on rubber resilience makes it necessary to correct the measured value of skid resistance to a standard temperature. The road temperature is measured by recording the temperature of the water after the test using a digital thermometer and surface probe. It is recommended that in tropical climates the value should be corrected to a standard temperature of 35^{o}C using the following relation,

$$SRV_{35} = (100 + t)/135 \; . \; SRV_t$$

where
SRV_{35} = Skid resistance value at 35^{o}C
SRV_t = Measured skid resistance value
t = Temperature of test

At this standard temperature the corrected values will be 3-5 units lower than comparable surfaces in the UK, where results are corrected to 20^{o}C.

E.2.2 Standards

If national standards are not available for skid resistance then those recommended in the UK may be used as a guide. The present UK intervention levels are now specified in terms of the Sideway-Force Coefficient (SFC) as measured by SCRIM. If only the portable skid resistance tester is available, then previous UK standards, summarised in Table E.3 are suggested as a preliminary guide.

Table E.3 Suggested minimum skid resistance values

Type of site	Minimum SRV @ 20°C
Difficult sites such as: Roundabouts Bends with a radius less than 50m on unrestricted roads Gradients, 1 in 20 or steeper of lengths greater than 100m Approaches to traffic lights on unrestricted roads	65
Motorways, trunk and class 1 roads and heavily trafficked roads in urban areas (carrying more than 2000 vehicles per day)	55
All other sites	45

Appendix F Test Pits and Material Testing

When the results of the condition survey indicate that the properties of the asphalt surfacing could be the cause of differential performance between sub-sections then this should be confirmed by further testing. Sufficient 150mm diameter core samples need to be taken from each sub-section to ensure that representative values for the composition and properties of the asphalt surfacing are obtained. Prior to testing, the cores must be examined to establish the following

i) thickness of each bound layer;
ii) degree of bonding;
iii) occurrence of any stripping; and
iv) depth of cracking (if required).

Where only the thickness of the asphalt surfacing is to be measured, then 50-100mm diameter cores are satisfactory. Similar cores can be used for transverse core profiles which are used to establish whether rutting is the result of shear failure in the surfacing or in one of the lower unbound pavement layers.

When deflection measurements and DCP results indicate that either the thickness or properties of the lower pavement layers are the cause of the differential performance, then test pits are needed to obtain additional material information to confirm these results. These investigations are used both to provide an explanation for the present behaviour of the pavement and to provide information for its rehabilitation. Each test pit will provide information on the thickness of each pavement layer and properties of the material. These can then be compared to specified values.

F.1 Test pit procedure

F.1.1 Purpose

The purpose of carrying out a test pit investigation is to confirm the engineers understanding of the information from surface condition, deflection and DCP surveys. It is a time consuming and expensive operation, and for this reason the location of each test pit should be carefully selected to maximise the benefit of any data collected. The condition of the road pavement and the primary purpose for the pit investigation should be recorded on the Test Pit Log (see Figure F.1).

F.1.2 Labour, equipment and materials

Test pits can be excavated either by machine or manually. The choice will normally be determined by the availability of plant and the test pit programme. Machine operations are usually more productive but more costly than manual methods.

The following personnel are required;

i) Traffic controllers - a minimum of one at each end of the site (but see above);
ii) Labourers: 2 (if machine excavation) or 3 (if manual excavation);
iii) 1 machine operator if applicable;
iv) 1 driver for vehicle; and
v) 1 supervising technician.

Location Data

Date: | Done by: | Weather:

Road Number: | From: | To: | Section:

Chainage: | Position: | Pit Number:

Pavement Condition:

Purpose of Investigation:

Test Pit Data

Method of Pitting:

Depth (mm)	Layer Function	Material Description	Sample Depth (mm)	Tests Required	Remarks
500					
1000					
1500					

Notes for Completing Test Pit Data

Layer Function: S-Surfacing, R-Roadbase, SB-Sub-base, SF-Select Fill, SGR-Subgrade
Material Description: Subjective assessment of material type and properties
Sample Depth: Depth (range) at which any samples taken
Tests Required: Note any laboratory tests required
Remarks: Note any particular points of interest such as pavement or drainage condition, on site tests (moisture, density), evidence of groundwater etc.

Figure F.1 Test Pit Log

Equipment and materials requirements are as follows;

i) 1 backhoe (for machine excavation);
ii) 1 jack hammer with generator (to assist with manual excavation);
iii) 1 pick;
iv) 1 or 2 spades (a fence post hole digger can also be useful);
v) 1 tamper or plate compactor for backfilling test pit;
vi) material to backfill and seal test pit : gravel, cement for stabilising gravel, water and cold mix for resurfacing;
vii) 1 broom to tidy area on completion;
viii) 1 chisel is often useful to assist with inspecting the wall of the test pit;
ix) equipment necessary to complete any required on-site testing;
x) 1 tape measure and thin steel bar to span pit (to assist with depth measurements);
xi) sample bags and containers, with some means of labelling each;
xii) test pit log forms and clipboard; and
xiii) sample log book.

F.1.3 Sampling and testing

Before commencing the survey in the field, the engineer should be clear as to the information required from each test pit. This will depend on the results of previous surveys, the materials specifications in use and an understanding of the pavement behaviour. Some field testing might be necessary as well as subsequent laboratory testing of samples extracted from the pit. Table F.1 summarises the various tests that may be required and references the relevant standards with which the tests should comply. Not all these tests will be necessary and the engineer must decide on those which are required.

Table F.1. Possible information from test pit investigation

Property	Possible Tests[1]	Field or Lab[2]	Original BS Standard[3]	New designation	Remarks
Particle size distribution	Sieve analysis	Lab	BS1377:Part 2:1990	BS EN 14688	Initial visual assessment on site.
Plasticity	Plastic and Liquid Limits, Plasticity Index	Lab	BS1377:Part 2:1990	BS EN 14688	Initial visual assessment on site.
	Linear Shrinkage	Lab	BS1377:Part 2:1990	BS EN 14688	Correlated to PI
Particle Shape[4]	Elongation Index	Lab	BS812:Part 105:1990	BS EN 933-3	
	Flakiness Index	Lab	BS812:Part 105:1990	BS EN 933-3	
Particle Strength[4]	Aggregate Crushing Value	Lab	BS812:Part 110:1990	BS EN 1097-2	Los Angeles Abrasion Value given in ASTM C 131-96 and C 535-96
	10% Fines Value	Lab	BS812:Part 111:1990	BS EN 1097-2	
	Aggregate Impact Value	Lab	BS812:Part 112:1990	BS EN 1097-2	
Particle Durability[4]	Aggregate Abrasion	Lab	BS812:Part 113:1990	BS EN 1097-8	Los Angeles Abrasion Value given in ASTM C 131-96 and C 535-96
	Accelerated Polishing	Lab	ASTM D 3319-90		
Particle Soundness	Sulphate test	Lab	BS812:Part 121:1989	BS EN 1367-2	
Particle Density	Particle density	Lab	BS1377:Part 2:1990	BS EN 14688	For soils
	Particle density	Lab	BS812:Part2:1975	BS EN 1097-6	For aggregates
Moisture Content	Oven dry[7]	Lab	BS1377:Part 2:1990	BS EN 14688	Recommended method
	'Speedy'	Field	Suppliers instructions		
	Nuclear Density Meter	Field	Suppliers instructions		Hazardous radioactive material
Moisture Density Relationship	Tests at various levels of compaction	Lab	BS1377:Part 4:1990	BS EN 14688	
Layer[5] Density	Sand Replacement Method	Field	BS1377:Part 9:1990	BS EN 14688	
	Core Cutter Method	Lab	BS1377:Part 9:1990	BS EN 14688	
	Nuclear Density Meter	Field	Suppliers instructions		Hazardous radioactive material
Bearing Capacity	DCP	Field	See Appendix F		
	California Bearing Ratio	Lab or Field	BS1377:Part 4:1990	BS EN 14688	

Property	Possible Tests[1]	Field or Lab[2]	Original BS Standard[3]	New designation	Remarks
Shear Strength[6]	Vane test	Field	BS1377:Part 9:1990	BS EN 14688	
	Various load tests	Lab	BS1377:Part 8/9:1990	BS EN 14688	

Notes

1. In some cases, the possible tests listed for a given property are alternatives. In other cases all the tests listed for a given property might be required. The engineer must decide for which properties information is required and then design a suitable testing programme.
2. Field tests require testing at the site and possibly further analysis in the laboratory. Laboratory tests require only sampling in the field. All sampling should be carried out in accordance with the general guidance of BS1377 or BS812, whichever is applicable, as well as any specific requirements for each test.
3. British Standards (BS) are quoted where applicable. Where no British Standard is available, an alternative is quoted. BS standards are now superseded by Euro Codes.
4. These tests will only be required for surfacing or base materials.
5. The layer must consist of homogeneous material for these tests.
6. These tests will only be required where a slope stability or settlement problem is being evaluated and will only apply to subgrade materials.
7. The oven drying method is recommended since it provides a fundamental measure of the moisture content. Both the `Speedy' and the Nuclear Density Meter methods require accurate calibration and validation, since they derive the moisture content by indirect analysis, but they have the advantage of providing instant results. Validation should always be made with reference to the oven dry method.

F.1.4 Procedure

A safe working environment should be maintained at all times. Many organisations will have on-site safety procedures which should be followed. Where there are no local safety procedures those described in Overseas Road Note 2 are recommended.

Once it has been decided what testing is to be carried out and the location of the trial pits has been confirmed, the following procedure should be adopted.

i) Set up traffic control.

ii) Accurately locate position of test pit and record this on the Pavement Test Pit Log (see Figure F.1). Usually, the position of a pit will be apparent after completion due to the patched surface. However, if long term monitoring is required, a permanent location marker should be placed at the roadside. Record any relevant details such as surrounding drainage features, road condition and weather.

iii) Define the edge of the test pit and remove surfacing. The required size of pit will depend on the sample sizes necessary for the selected tests, but it can be increased later if found to be too small. Usually an area of about 0.8m by 0.8m will be sufficient for manual excavation, and the minimum working area required for a backhoe operation will be sufficient for machine excavations. The edge of the pit can be cut with a jack hammer or pick and the surfacing 'peeled' off, taking care not to disturb the surface of the roadbase. The average thickness of surfacing should be recorded.

iv) If density tests are to be performed, a smooth, clean and even surface is required. It is important for the accuracy of the test that the layer is homogeneous. For the sand replacement method, no prior knowledge is required of the layer thickness since this becomes obvious as the hole is excavated. If a nuclear density meter is used, the thickness of the layer can be estimated from the DCP results to determine the depth of testing.

v) On completion of any required density testing, the layer can be removed over the extent of the trial pit, a visual assessment made of the material and samples taken for laboratory testing. Care should be taken not to disturb the adjacent lower layer. The thickness of the layer and the depth at which samples are taken should be measured. All information should be recorded on the Pavement Test Pit Log.

vi) Continue to sample, test and excavate each pavement layer following the procedure above. Once it has been decided that there is no need to excavate further, the total depth of pit should be recorded along with any other information such as appearance of water in any of the layers.

vii) All samples should be clearly labelled and proposed tests for the pit materials should be logged in a sample log book to avoid later confusion in the laboratory.

viii) The pit should be backfilled in layers with suitable material which should be properly compacted. It is often good practice to stabilise the upper layer with cement accepting that full compaction will not be achieved. A bituminous cold mix can be used to patch the backfilled pit.

ix) The site should be cleared and left in a tidy and safe condition for traffic.

Appendix G Distress in Rigid Pavements

It is sometimes necessary to carry out a full survey of a rigid (concrete) pavement and to record the severity of the different forms of distress. This Appendix describes all the forms of distress that occur on rigid pavements and suggests methods of measuring and recording their extent and severity. A standard field survey form similar to that shown in Figure 3.2 has not been devised but will be required when concrete pavements become more common in Ethiopia.

G.1 Blow-up

Most blow-ups occur during the spring and hot summer at a transverse joint or wide crack. Infiltration of incompressible materials into the joint or crack during cold periods results in high compressive stresses in hot periods. When this compressive pressure becomes too great, a localized upward movement of the slab or shattering occurs at the joint or crack. Blow-ups are accelerated due to a spalling away of the slab at the bottom, creating reduced joint contact area. The presence of "D" cracking or freeze-thaw damage also weakens the concrete near the joint, resulting in increased spalling and blow-up potential.

Severity Levels:

L - Blow-up has occurred, but only causes some bounce of the vehicle which creates no discomfort.

M - Blow-up causes a significant bounce of the vehicle which creates some discomfort. Temporary patching may have been placed because of the blow-up.

H - Blow-up causes excessive bounce of the vehicle, which creates substantial discomfort and/or a safety hazard and/or vehicle damage, requiring a reduction

How to Measure:

Blow-ups are measured by counting the number existing in each uniform section. Severity level is determined by riding in a mid- to full-sized car weighing approximately 13-17 kN over the uniform section at the posted speed limit. The number is not as important as the fact that initial blow-ups signal a problem that needs to be investigated.

G.2 Corner break

A corner break is a crack that intersects the joints at a distance less than 1.8 m on each side, measured from the corner of the slab. A corner break extends vertically through the entire slab thickness. It should not be confused with a corner spall, which intersects the joint at an angle through the slab and is typically within 0.3 m from the slab corner. Heavy repeated loads, combined with pumping, poor load transfer across the joint, and thermal curling and moisture warping stresses, result in corner breaks.

Severity Levels:

L - Crack is tight (hairline). Well-sealed cracks are considered tight. No faulting or break-up of broken corner exists. Crack is not spalled.

M - Crack is working and spalled at medium severity, but break-up of broken corner has not occurred. Faulting of crack or joint is less than 13 mm. Temporary patching may have been placed because of corner break.

H - Crack is spalled at high severity, the corner piece has broken into two or more pieces, or faulting of crack or joint is more than 13 mm.

How to Measure:

Corner breaks are measured by counting the number that exists in the uniform section. Different levels of severity should be counted and recorded separately. Corner breaks adjacent to a patch will be counted as "patch adjacent slab deterioration".

G.3 Depression

Depressions in concrete pavements are localized settled areas. There is generally significant slab cracking in these areas, due to uneven settlement. The depressions can be located by stains caused by oil droppings from vehicles and by riding over the pavement. Depressions can be caused by settlement or consolidation of the foundation soil, or can be 'built-in' during construction. They are frequently found near culverts. This is usually caused by poor compaction of soil around the culvert during construction. Depressions cause slab cracking, roughness, and hydroplaning when filled with water of sufficient depth.

Severity Levels:

L - Depression causes a distinct bounce of vehicle which creates no discomfort.

M - Depression causes significant bounce of the vehicle, which creates some discomfort.

H - Depression causes excessive bounce of the vehicle, which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

Depressions are measured by counting the number that exists in each uniform section. Each depression is rated according to its level of severity. Severity level is determined by riding in a mid- to full-sized car weighing approximately 13-17 kN over the uniform section at the posted speed limit.

G.4 Durability (D) cracking

'D' cracking is a series of closely-spaced, crescent-shaped hairline cracks that appear at a PCC pavement slab surface adjacent and roughly parallel to transverse and longitudinal joints, transverse and longitudinal cracks, and the free edges of pavement slab. The fine surface cracks often curve around the intersection of longitudinal joints/cracks and transverse joints/cracks. These surface cracks often contain calcium hydroxide residue, which causes a dark colouring of the crack and immediate surrounding area. This may eventually lead to disintegration of the concrete within 0.3 to 0.6 m or more of the joint or crack, particularly in the wheel paths. D cracking is caused by freeze-thaw expansive pressures of certain types of coarse aggregates and typically begins at the bottom of the slab, which disintegrates first. Concrete durability problems caused by reactive aggregates are rated under 'Reactive Aggregate Distress'.

Severity Levels:

L - The characteristic pattern of closely-spaced fine cracks has developed near joints, cracks, and/or free edges; however, the width of the affected area is generally

<30 cm wide at the centre of the lane in transverse cracks and joints. The crack pattern may fan out at the intersection of transverse cracks/joints with longitudinal cracks/joints. No joint/crack spalling has occurred, and no patches have been placed for 'D' cracking.

M - The characteristic pattern of closely-spaced cracks has developed near the crack, joint, or free edge and: (1) is generally wider than 30 cm at the centre of the lane in transverse cracks and/or joints; or (2) low- or medium-severity joint/crack or corner spalling has developed in the affected area; or (3) temporary patches have been placed due to "D" cracking-induced spalling.

H - The pattern of fine cracks has developed near joints or cracks and (1) a high severity level of spalling at joints/cracks exists and considerable material is loose in the affected area; or (2) the crack pattern has developed generally over the entire slab area between cracks and/or joints.

How to Measure:

'D' cracking is measured by counting the number of joints or cracks (including longitudinal) affected. Different severity levels are counted and recorded separately. 'D' cracking adjacent to a patch is rated as patch-adjacent slab deterioration. 'D' cracking should not be counted if the fine crack pattern has not developed near cracks, joints, and free edges. Popouts and discoloration of joints, cracks, and free edges may occur without 'D' cracking.

G.5 Faulting of transverse joints and cracks

Faulting is the difference of elevation across a joint or crack. Faulting is caused in part by a build-up of loose materials under the approach slab near the joint or crack, as well as depression of the leave slab. The build-up of eroded or infiltrated materials is caused by pumping from under the leave slab and shoulder (free moisture under pressure) due to heavy loadings. The warp and/or curl upward of the slab near the joint or crack due to moisture and/or temperature gradient contributes to the pumping condition. Lack of load transfer contributes greatly to faulting.

Severity Levels:

Severity is determined by the average faulting over the joints within the sample unit.

How to Measure:

Faulting is determined by measuring the difference in elevation of slabs at transverse joints for the slabs in the sample unit. Faulting of cracks is measured as a guide to determine the distress level of the crack. Faulting is measured 30 cm in from the outside (right) slab edge on all lanes except the innermost passing lane. Faulting is measured 30 cm in from the inside (left) slab edge on the inner passing lane. If temporary patching prevents measurement, proceed on to the next joint. Sign convention: + when approach slab is higher than departure slab, – when the opposite occurs. Faulting never occurs in the opposite direction.

G.6 Joint load transfer system associated deterioration (second stage cracking)

This distress develops as a transverse crack a short distance from a transverse joint at the end of joint dowels. This usually occurs when the dowel system fails to function properly

due to extensive corrosion or misalignment. It may also be caused by a combination of smaller diameter dowels and heavy traffic loadings.

Severity Levels:

L - Hairline (tight) crack with no spalling or faulting or well-sealed crack with no visible faulting or spalling.

M - Any of the following conditions exist: the crack has opened to a width less than 25 mm; the crack has faulted less than 13 mm; the crack may have spalled to a low- or medium-severity level; the area between the crack and joint has started to break up, but pieces have not been dislodged to the point that a tire damage or safety hazard is present; or temporary patches have been placed due to this joint deterioration.

H - Any of the following conditions exist: a crack with width of opening greater than 25 mm; a crack with a high-severity level of spalling; a crack faulted 13 mm or more; or the area between the crack and joint has broken up and pieces have been dislodged to the point that a tire damage or safety hazard is present.

How to Measure:

The number of joints with each severity level is counted in the uniform section.

G.7 Joint seal damage of transverse joints

Joint seal damage exists when incompressible materials and/or water can infiltrate into the joints. This infiltration can result in pumping, spalling, and blow-ups. A joint sealant bonded to the edges of the slabs protects the joints from accumulation of incompressible materials and also reduces the amount of water seeping into the pavement structure. Typical types of joint seal damage are: (1) stripping of joint sealant, (2) extrusion of joint sealant, (3) weed growth, (4) hardening of the filler (oxidation), (5) loss of bond to the slab edges, and (6) lack or absence of sealant in the joint.

Severity Levels:

L - Joint sealant is in good condition throughout the section with only a minor amount of any of the above types of damage present. Little water and no incompressible material can infiltrate through the joint.

M - Joint sealant is in fair condition over the entire surveyed section, with one or more of the above types of damage occurring to a moderate degree. Water can infiltrate the joint fairly easily; some compressible material can infiltrate the joint. Sealant needs replacement within 1 to 3 years.

H - Joint sealant is in poor condition over most of the sample unit, with one or more of the above types of damage occurring to a severe degree. Water and incompressible material can freely infiltrate the joint. Sealant needs immediate replacement.

How to Measure:

Joint sealant damage of transverse joints is rated based on the overall condition of the sealant over the entire sample unit.

G.8 Lane/shoulder drop-off or heave

Lane/shoulder drop-off or heave occurs when there is a difference in elevation between the traffic lane and shoulder. Typically, the outside shoulder settles due to consolidation or a

settlement of the underlying granular or subgrade material or pumping of the underlying material. Heave of the shoulder may occur due to frost action or swelling soils. Drop-off of granular or soil shoulder is generally caused from blowing away of shoulder material from passing trucks.

Severity Levels:

Severity level is determined by computing the mean difference in elevation between the traffic lane and shoulder.

How to Measure:

Lane/shoulder drop-off or heave is measured in the sample unit at all joints when joint spacing is > 15 m, at every third joint when spacing is < 15 m. It is also measured at mid-slab in each slab measured at the joint. The mean difference in elevation is computed from the data and used to determine severity level. Measurements at joints are made 0.3 m from the transverse joint on the departure slab only on the outer lane/shoulder.

G.9 Lane/shoulder joint separation

Lane/shoulder joint separation is the widening of the joint between the traffic lane and the shoulder, generally due to movement in the shoulder. If the joint is tightly closed or well-sealed so that water cannot easily infiltrate, then lane/shoulder joint separation is not considered a distress.

Severity Levels:

No severity level is recorded if the joint is tightly sealed.

L - Some opening, but less than or equal to 3 mm.

M - More than 3 mm, but equal or less than 10 mm opening.

H - More than 10 mm opening. Gravel or sod shoulders are rated as high.

How to Measure:

Lane/shoulder joint separation is measured and recorded in mm near transverse joints and at mid-slab. The mean separation is used to determine the severity level.

G.10 Longitudinal cracks

Longitudinal cracks occur generally parallel to the centreline of the pavement. They are often caused by improper construction of longitudinal joints or by a combination of heavy load repetition, loss of foundation support, and thermal and moisture gradient stresses.

Severity Levels:

L - Hairline (tight) crack with no spalling or faulting, or a well-sealed crack with no visible faulting or spalling.

M - Working crack with a moderate or less severity spalling and/or faulting less than 12 mm.

H - A crack with width greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

Cracks are measured in meters for each level of distress. The length and average severity of each crack should be identified and recorded.

G.11 Longitudinal joint faulting

Longitudinal joint faulting is a difference in elevation of two traffic lanes measured at the longitudinal joint. It is caused primarily by heavy truck traffic and settlement of the foundation.

Severity Levels:

Severity level is determined by measuring the maximum fault.

How to Measure:

Where the longitudinal joint has faulted, the length of the affected area and the maximum joint faulting is recorded.

G.12 Patch deterioration (including replaced slabs)

A patch is an area where a portion of the original slab, or the entire original slab, has been removed and replaced with a permanent type of material (e.g. concrete or hot-mixed asphalt). *Only permanent patches should be considered.*

Severity Levels:

L - Patch has little or no deterioration. Some low severity spalling of the patch edges may exist. Faulting across the slab-patch joints must be less than 6 mm. Patch is rated low severity even if it is in excellent condition.

M - Patch has cracked (low severity level) and/or some spalling of medium-severity level exists around the edges. Minor rutting may be present. Faulting of 6-19 mm exists. Temporary patches may have been placed because of permanent patch deterioration.

H - Patch has deteriorated by spalling, rutting, or cracking within the patch to a condition which requires replacement.

How to Measure:

The number of patches within each uniform section is recorded. Patches at different severity levels are counted and recorded separately. Additionally, the approximate area (in square meters) of each patch and type (i.e., PCC or asphalt) is recorded. All patches are rated L, M, or H.

G.13 Patch adjacent slab deterioration

Deterioration of the original concrete slab adjacent to a permanent patch is given the above name. This may be in the form of spalling of the slab at the slab/patch joint, “D” cracking of the slab adjacent to the patch, a corner break in the adjacent slab, or a second permanent patch placed adjacent to the original patch.

Severity Levels:

Severity levels are the same as that described for the particular distress found. A second permanent patch, placed adjacent to a previously-placed permanent patch, is rated here as

medium severity. Temporary patches placed because of this deterioration are also rated here as medium severity.

How to Measure:

The number of permanent patches with distress in the original slab adjacent to the patch at each severity level is recorded separately. Additionally, the type of patch (AC or PCC) and distress is recorded separately.

G.14 Popouts

A popout is a small piece of concrete that breaks loose from the surface due to freeze-thaw action, expansive aggregates, and/or nondurable materials. Popouts may be indicative of unsound aggregates and "D" cracking. Popouts typically range from approximately 25 mm to 10 cm in diameter and from 10 to 50 mm deep.

Severity Levels:

No degrees of severity are defined for popouts. The average popout density must exceed approximately one popout per square meter over the entire slab area before they are counted as a distress.

How to Measure:

The density of popouts can be determined by counting the number of popouts per square meter of surface in areas having typical amounts.

G.15 Pumping and water bleeding:

Pumping is the movement of material by water pressure beneath the slab when it is deflected under a heavy moving wheel load. Sometimes the pumped material moves around beneath the slab, but often it is ejected through joints and/or cracks (particularly along the longitudinal lane/shoulder joint with an asphalt shoulder). Beneath the slab there is typically particle movement counter to the direction of traffic across a joint or crack that results in a build up of loose materials under the approach slab near the joint or crack. Often some fine materials are pumped out, leaving a thin layer of relatively loose clean sand and gravel beneath the slab, along with voids causing loss of support. Pumping occurs even in pavement sections containing stabilized sub-bases.

Water bleeding occurs when water seeps out of joints and/or cracks. Often it drains out over the shoulder in low areas.

Severity Levels:

L - Water is forced out of a joint or crack when trucks pass over the joints or cracks; water is forced out of the lane/shoulder longitudinal joint when trucks pass along the joint; or water bleeding exists. No fines can be seen on the surface of the traffic lanes or shoulder.

M - A small amount of pumped material can be observed near some of the joints or cracks on the surface on the traffic lane or shoulder. Blow holes may exist.

H - A significant amount of pumped materials exist on the pavement surface of the traffic lane or shoulder along the joints or cracks.

How to Measure:

If pumping or water bleeding exists anywhere in the sample unit, it is counted as occurring at highest severity level, as defined above.

G.16 Aggregate distresses

Reactive aggregates either expand in alkaline environments or develop prominent siliceous reaction rims in concrete. This may be an alkali-silica reaction or an alkali-carbonate reaction. As expansion occurs, the cement matrix is disrupted and cracks. It appears as a map-cracked area; however, the cracks may go deeper into the concrete than in normal map cracking. This may affect most of the slab or it may first appear at joints and cracks.

Severity Levels:

Only one level of severity is defined. If alkali-aggregate cracking occurs anywhere in the slab, it is counted. If the reaction has caused spalling or map cracking, these are also counted.

How to Measure:

Reactive-aggregate distress is measured in square feet or square meters.

G.17 Scaling and map cracking or crazing

Scaling is the deterioration of the upper 3 to 13 cm of the concrete slab surface. Map cracking or crazing is a series of fine cracks that extend only into the upper surface of the slab surface. Map cracking or crazing is usually caused by over-finishing of the slab and may lead to scaling of the surface. Scaling can also be caused by reinforcing steel being too close to the surface.

Severity Levels:

L - Crazing or map cracking exists; the surface is in good condition with no scaling.

M/H -Scaling exists.

How to Measure:

Scaling and map cracking or crazing are measured by area of slab in square meters.

G.18 Spalling (transverse and longitudinal joint/crack):

Spalling of cracks and joints is the cracking, breaking or chipping (or fraying) of the slab edges within 0.6 m of the joint/crack. A spall usually does not extend vertically through the whole slab thickness but extends to intersect the joint at an angle. Spalling usually results from (1) excessive stresses at the joint or crack caused by infiltration of incompressible materials and subsequent expansion, (2) disintegration of the concrete from freeze-thaw action of "D" cracking, (3) weak concrete at the joint (caused by honeycombing), (4) poorly designed or constructed load transfer device (misalignment, corrosion), and/or (5) heavy repeated traffic loads.

Severity Levels:

L - The spall or fray does not extend more than 8 cm on either side of the joint or crack. No temporary patching has been placed to repair the spall.

M - The spall or fray extends more than 8 cm on either side of the joint or crack. Some pieces may be loose and/or missing, but the spalled area does not present a tyre damage or safety hazard. Temporary patching may have been placed because of spalling.

H - The joint is severely spalled or frayed to the extent that a tyre damage or safety hazard exists.

How to Measure:

Spalling is measured by counting and recording separately the number of joints with each severity level. If more than one level of severity exists along a joint, it will be recorded as containing the highest severity level present. Although the definition and severity levels are the same, spalling of cracks should not be recorded. *The spalling of cracks is included in rating severity levels of cracks*. Spalling of transverse and longitudinal joints is recorded separately. Spalling of the slab edge adjacent to a permanent patch is recorded as patch adjacent slab deterioration. If spalling is caused by "D" cracking, it is counted as both spalling and "D" cracking at appropriate severity levels.

G.19 Spalling (corner):

Corner spalling is the ravelling or breakdown of the slab within approximately 0.3 m of the corner. However, corner spalls with both edges less than 8 cm long are not recorded. A corner spall differs from a corner break in that the spall usually angles downward at about 45° to intersect the joint, while a break extends vertically through the slab. Corner spalling can be caused by freeze-thaw deterioration, "D" cracking, and other factors.

Severity Levels:

L - Spall is not broken into pieces and not loose.

M - One of the following conditions exists: Spall is broken into pieces; cracks are spalled; some or all pieces are loose or absent but do not present tyre damage or safety hazard; or spall is patched.

H - Pieces of the spall are missing to the extent that the hole presents a tyre damage or safety hazard.

How to Measure:

Corner spalling is measured by counting and recording separately the number of corners spalled at each severity level within the sample unit.

G.20 Swell

A swell is an upward movement or heave of the slab surface, sometimes resulting in a sharp wave. The swell is usually accompanied by slab cracking. It is usually caused by frost heave in the subgrade or by an expansive soil. Swells can often be identified by oil droppings on the surface as well as riding over the pavement in a vehicle.

Severity Levels:

L - Swell causes distinct bounce of the vehicle which causes no discomfort.

M - Swell causes significant bounce of the vehicle which creates some discomfort.

H - Swell causes excessive bounce of the vehicle which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

The number of swells within the uniform section are counted and recorded by severity level. Severity levels are determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit.

G.21 Transverse and diagonal cracks

Linear cracks are caused by one or a combination of the following: heavy load repetition, thermal and moisture gradient stresses, and drying shrinkage stresses. Medium- or high-severity cracks are working cracks and are considered major structural distresses. They may sometimes be due to deep-seated differential settlement problems. (NOTE: Hairline cracks that are less than 1.8 m long are not rated.)

Severity Levels:

L - Hairline (tight) crack with no spalling or faulting, a well-sealed crack with no visible faulting or spalling.

M - Working crack with low- to medium-severity level of spalling, and/or faulting less than 13 mm; temporary patching may be present.

H - A crack with width of greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

The number and severity level of each crack should be identified and recorded. If the crack does not have the same severity level along the entire length, the crack is rated at the highest severity level present. Cracks in patches are recorded under patch deterioration.

Appendix H Recycling Pavement Materials

All pavement layers in a road that is to be rehabilitated can, in principle, be recycled and used in the new pavement. Most commonly, an unbound layer of gravel or processed aggregate may be scarified and recompacted or stabilised with cement or lime to form a sub-base or roadbase for the new pavement. The design and processing of such materials differs little from the processing of completely new layers and is described in ERA's *Pavement Design Manual.* The recycling of asphalt layers or concrete layers is more complicated and is described in this Appendix.

H.1 Introduction to asphalt recycling

The use of thick bituminous surfacings is increasing as traffic loads increase. When they become worn out the recycling of such materials can reduce costs and damage to the environment by reducing the exploitation of local natural resources. This is particularly true for areas where there is a shortage of road-building aggregate or where there are no indigenous oil reserves.

It is possible to ensure that there are benefits both for client and contractor from recycling operations but recycling is unlikely to become widespread until certain conditions are met. These include:

i) sufficient potential for recycling to justify the purchase of specialist plant;
ii) pressure from government agencies to implement recycling;
iii) suitable specifications and/or working methodologies for contractual use; and
iv) good understanding of material properties and methods of modification and application.

The asphalts that are likely to be available for recycling are those that have become severely age-hardened and cracked or surfacings that have failed through plastic deformation. Of these, the former is more common, with plastic deformation failures occurring locally where traffic is slow moving such as at junctions and on climbing lanes. Aged asphalt is brittle, containing very hard bitumen. In contrast, asphalt that has deformed plastically is likely to contain bitumen that has suffered very little age hardening. These two types of material present different problems for recovery, stockpiling and re-use.

H.2 Methods of recycling

Reclaimed Asphalt Pavement (RAP), or millings, are recycled in three main ways. They are crushed and used as granular materials for fill or lower pavement layers or re-used in a bituminous material, either by cold mix or hot mix recycling. These recycling processes can be carried out either in-place or at a central plant. The greatest cost savings will be obtained when RAP is used to produce good quality bitumen bound material and its use as unbound material should be regarded as the minimum target for recycling.

The decision to recycle asphalt and the quality that can be achieved is determined by a number of factors which include the following:

i) availability of suitable recycling plant;
ii) thickness of the existing bituminous layer;

iii) the effect on traffic management, e.g. can deviations be constructed or must the carriageway be partially open to traffic;
iv) level of quality control that can be achieved in the recycling process; and
v) variability in the properties of the existing material.

Standard 'cutting-out' and crushing equipment can be very effective for producing well-graded RAP from brittle age-hardened asphalt. This material, modified if necessary by the addition of fresh aggregate, may be suitable for use in any pavement layer. However, it is recommended that RAP is not used to manufacture bituminous wearing courses unless it can be demonstrated that the high degree of uniformity and the close tolerances required for this critical layer can be achieved. The wider tolerances allowed for bituminous roadbase and, to a lesser extent, for binder courses, make these layers more suitable for incorporating RAP. The uniformity and quality of the RAP and the type of recycling plant will determine the percentage of RAP that can be used in the mixes. Typically this will range from 20 to 50 per cent.

When RAP is to be used in a pavement layer, good quality control of the RAP stockpiles is vital for the manufacture of consistent HMA. This may require a considerable amount of testing. The presence of old multiple surface dressings may also be acceptable if they have age-hardened. However, where there has been heavy patching or the quality of the seal is variable, recycling may be limited to *in situ* pulverisation and stabilisation.

Powerful pulverisers are available which make it possible to carry out cold *in situ* recycling with fresh materials being incorporated as necessary to produce a layer of the required quality.

H.3 Method of sampling existing asphalt

A feasibility study is necessary to assess the variability of the existing material and to establish that a suitable mix design can be achieved and can be manufactured with the available plant. Samples must be cut from the existing asphalt for analysis. A balance must be found between costly and time consuming testing and the need for sufficient samples to determine material variability. The sampling pattern should take account of visually obvious variability such as:

i) contaminated 'oil lanes';
ii) wheel paths that look 'rich' in bitumen, indicating a dense material in which bitumen hardening may not be as severe as elsewhere in the pavement;
iii) material which looks rich in bitumen and may have deformed plastically; and
iv) cracking or fretting indicating that appreciable bitumen hardening has occurred.

Identification of road lengths with apparently uniform appearance is used to establish short representative sections which can be tested. Based on these sections, a suitable pattern of testing can be established. The intention should be to stockpile separately severely age-hardened materials (typically with penetration values of recovered bitumen of less than 20) from less hardened materials and to discard badly contaminated material from 'oil lanes'. Detailed assessment of stockpile management should be finalised after a desk study has been carried out to show how the various RAP materials can be combined with fresh aggregate to produce acceptable mixes.

The following tests should be carried out to determine material properties:

i) particle size distribution;
ii) bitumen content;
iii) viscosity of recovered bitumen; and
iv) an assessment of crushability.

Recovering bitumen from RAP to determine the penetration of the existing bitumen presents a problem for many authorities. Unless it is clear that the existing bitumen is severely age hardened it is likely that a carefully selected batch of cores will have to be sent to a qualified testing house to have these tests carried out.

H.4 Methods of obtaining RAP

RAP can be obtained by milling or it can be cut from the road in lumps which must be crushed. An assessment of the likelihood of obtaining a well-crushed material with the available plant must be made, preferably at the feasibility stage. Milling is particularly useful where traffic access must be retained during the removal of damaged asphalt. Either method is suitable when the road is closed to traffic during the rehabilitation work.

H.4.1 Asphalt millings

Asphalt millings are obtained by planing in a layer by layer fashion using a mobile plant. They are usually consistent in their lump-size distribution and can normally be used as granular material as won or with minimum screening to remove any over-size material.

H.4.2 Crushed asphalt

Crushed asphalt is commonly obtained by using horizontal impact crushers or hammer mill impact crushers. Jaw/roll combination crushers are not suitable for processing RAP which contains ‘soft’ bitumen because ‘pancaking’ can occur on warm days and the material will remain agglomerated.

H.4.3 Granulated asphalt

Granulated asphalt is produced in a specialised plant, known as a granulator, or in milling/grinding units. These units are not crushers and are designed only to break the bitumen-asphalt bond.

H.5 Stockpiling RAP

The stockpiling of RAP is a very important part of the recycling process. The full benefits of comprehensive testing of the *in situ* asphalt layers can easily be lost if equally meticulous control of the stockpiling process is not put in place. Depending upon the variability found during testing, it may be necessary to build separate stockpiles of materials taken from different sections of the road.

The tendency for RAP to agglomerate is affected by both the hardness of the bitumen in the RAP and the ambient temperature. The most effective method of stockpiling must be established by trial and error. Experience has shown that RAP in large piles does not tend to agglomerate. A 250-300mm crust may form at the surface of the stockpile and this should be scalped off and reprocessed prior to recycling. Higher stockpiles should, therefore, provide more usable RAP.

RAP can hold up to 7-8% moisture which seriously reduces the amount of material that can be hot mixed, raises fuel costs and limits productivity. Although covering a stockpile with a waterproof sheet will prevent rainfall from entering the stockpile, condensation may occur within the stockpile. Ideally RAP for hot mixing should be stored under a roof in an open sided building.

Stockpiled RAP destined to be used as unbound granular material may be watered to prevent agglomeration of particles in warm weather and this also aids compaction on site.

H.6 Use of RAP as unbound granular material

Age-hardened asphalt can be recycled as an unbound granular material. It may be produced as millings, crushed asphalt from lumps or as granulated asphalt. The RAP can be mixed with fresh aggregate to produce a particle size distribution appropriate to the layer in which it will be used. The harder the bitumen in the RAP, the easier it will be to crush, handle and re-compact in the new layer. For example, bitumen in RAP with a penetration value of less than about 15 will behave in a brittle manner.

In contrast, asphalt which has failed by plastic deformation will have suffered little or no bitumen hardening in the wheel paths. In the 'oil lane' the bitumen may have softened over time whilst material outside the wheel paths, or oil lane, may have significantly age-hardened. This type of material is difficult to process and the best results can be expected by selective milling and stockpiling before re-blending and adding fresh aggregate in a purpose-made hot-mix recycling plant.

H.6.1 Outline of UK specification for use of RAP as capping layer

A capping layer is only used in the construction of a new pavement when the *in situ* subgrade CBR is less than 5 per cent.

The UK specification requires milled or granular RAP to meet the grading requirements given in Table H.1. The layer can contain 100 per cent RAP provided the bitumen content is less than 10 per cent. The recycled material may be laid to a maximum compacted thickness of 200mm provided the required density is obtained.

Table H.1 Grading requirements for RAP for use in capping layers

BS sieve size (mm)	Percent passing sieve size
125	100
90	80-100
75	65-100
37.5	45-100
10	15-60
5	10-45
0.6	0-25
0.063	0-12

A minimum density of 95 per cent of the maximum dry density obtained in the British Standard (Heavy) Compaction Test, 4.5 kg rammer, or in the British Standard Vibrating Hammer Test (BS 1377, Part 4, 1990), is a suitable specification.

Other unbound granular material can be added to the RAP to give a material with reduced effective bitumen content.

A higher quality material can be obtained by limiting the maximum effective particle size to 37.5mm by screening out and re-crushing oversized material.

H.6.2 Outline of UK specification for use of RAP as sub base

In principle, milled or crushed RAP can be used in the sub-base of a road pavement. The quality of the aggregate in the RAP should meet or exceed normal requirements for these layers. If good quality aggregate is added to modify the particle size distribution, then a compacted layer of the blended material should be of acceptable quality provided that the bitumen in the RAP is hard enough not to hinder compaction and that the finished layer is sufficiently dense. RAP for sub-base should conform to the particle size distribution given in Table H.2.

Representative samples of the RAP are compacted to determine the optimum moisture content using the procedure described in BS 5835-1: 1980. The laboratory compaction equipment specified in the British Standard includes a special mould, a loading frame and a vibrating hammer.

Table H.2 Range of lump-size grading of RAP for use in sub-base

BS sieve size (mm)	Percent passing sieve size
75	100
37.5	85-100
20	60-90
10	30-70
5	15-45
0.6	0-22
0.075	0-10

Note 1: The lump size distribution shall be determined either by the washing and sieving method or by the dry sieving method of BS 812: Part 103: 1985 (see note 2)

Note 2: The planings should be oven dried (prior to sieving) at a temperature of 45 to 50°C. Sieving shall be carried out at a temperature of 20 ± 5°C to reduce the tendency of the bitumen to soften and particles to adhere to each other. The temperature range for sieving can be higher when the RAP is age-hardened.

Where the required laboratory equipment is not available it may be possible to modify the compaction method using the British Standard Vibrating Hammer Test (BS 1377, Part 4, 1990) in conjunction with a mould that will allow drainage during compaction. It would be necessary to prove that this change in methodology is satisfactory. Adoption of the Trafficking Trial procedure could help to achieve this.

Material is laid at a moisture content between the optimum and 2 per cent below optimum and compacted without drying or segregation.

The UK specification allows for a Trafficking Trial. RAP, at the correct moisture content, is laid on a prepared trial area constructed to specified standards and trafficked with a loaded truck. After the equivalent of 1000 standard axles has been applied to a single track,

the mean deformation in the two wheel paths is measured. For the material to be approved the mean deformation must be less than 30mm.

H.6.3 Use of RAP as granular roadbase

If it can be shown that an unbound material containing RAP meets the specifications for grading, density and CBR which are normally applied to fresh materials then it is normally acceptable to use the RAP as roadbase. A limiting factor is the hardness of the bitumen in the RAP; 'softer' bitumen in agglomerations of bitumen and fines may prevent the achievement of the required density. In these circumstances the proportion of RAP used in the new layer must be restricted to a level at which thorough compaction can be achieved.

The roadbase is an important load bearing layer and it is therefore advisable to restrict the general use of RAP in this layer until experience of its performance has been acquired. Inclusion of RAP in a lower roadbase layer or under an AC surfacing on the more lightly trafficked roads provides an appropriate method of acquiring this experience.

H.7 Cold mix recycling

Cold mix recycling can be done at partial or full depth in an asphalt pavement with mixing carried out in-place or off-site at a central plant. The process preserves aggregate and bitumen, air quality problems are minimised and energy requirements are low. The existing pavement layers are reprocessed with the addition of fresh aggregate if required. During the reprocessing operation, hydraulic stabiliser, such as Portland cement or emulsified or foamed bitumen, is mixed in to produce a new material with the required properties. Cold mix recycling is outside the scope of this manual. Reference should be made to appropriate manuals for detailed recommendations.

H.8 Plant hot mix recycling

Hot-mix recycling is normally carried out off-site at a central plant. Asphalt containing tar should not be recycled because of the high risk of generating carcinogenic material.

H.8.1 RAP feed to plant

To avoid blockages that will substantially reduce output, RAP should be metered into the plant through cold feed bins having the following characteristics:

i) The sides should be steeper than those of an aggregate feed bin.
ii) The bottom of the bin may be longer and wider than that of an aggregate feed bin.
iii) The bottom of the bin may slope downwards, to match an angled feed belt, and the end wall is sometimes left open.
iv) Vibrators should not be used.
v) RAP should be delivered slowly into the cold feed bin from the front-end loader.
vi) The level in the bin should be kept fairly low. This means that the bin must be fed more frequently than is necessary for a normal aggregate cold feed bin.
vii) Material should not be left in the cold feed bin for more than one hour. It is more economical to run out the contents of the bin than to clear it later.

H.8.2 Batch plant recycling

Because cold aggregate travels towards the heating flame in this type of plant, the introduction of RAP would result in excessive smoke and other problems. The technique of conductive heat transfer, which involves the super heating of fresh aggregate and adding cold RAP via the elevator or directly into the weigh hopper minimises the likelihood of air pollution. The percentage of RAP that can be used depends upon the following factors:

i) The temperature to which the virgin aggregate is heated.
ii) The temperature and moisture content of the RAP.
iii) The required temperature of the final mix.

Under ideal conditions, batch plant recycling can blend up to 40 per cent RAP with superheated fresh aggregate but 15 to 25 per cent is more typical.

H.8.3 Batch mixers with a separate heating drum (parallel drum)

In this system RAP is heated in a separate drum to about 130ºC. Fresh aggregate is separately heated to a high temperature and both materials are weighed to produce the required blend in the mixing unit. The final temperature of the blend is about 160º C. Preheating allows 50 per cent of RAP to be used in the blend, or even more if a consistent quality of output can be guaranteed.

Preheating the RAP allows the production of a more uniform mix and better control of mix temperature and this is the preferred method of recycling. However, development continues and other types of plant specifically designed for recycling bituminous materials are becoming available.

H.9 Evaluation and design - plant hot-mix recycling

H.9.1 Variability of RAP

RAP is usually either material which has failed by plastic deformation, and contains mostly relatively soft bitumen, or badly cracked asphalt containing very hard bitumen. It is therefore important to determine the variations in properties of the bitumen in RAP and how this will be taken into account in the mix design process.

H.10 Bitumen rejuvenators

Rejuvenators have been used to change the properties of bitumen in RAP to make it similar to new bitumen. However, although such agents can change aged bitumen to obtain the required viscosity, different agents produce binders with different temperature susceptibilities. It has also been found that there can be problems relating to the compatibility between aged bitumen and the rejuvenating agent; hence trials are recommended before full scale use.

H.11 Blending with a soft bitumen

If a softer bitumen is added with the intention of bringing the blended bitumen within specification, the penetration (P) of the fresh bitumen can be calculated using equation H.1.

$$Log\ P = (A.\log Pa + B.\log Pb)/100 \quad \text{Equation H.1}$$

where,
P = specified penetration of final blend.
Pa = penetration of RAP bitumen.
Pb = penetration of virgin bitumen.
A = percentage of RAP bitumen in the final blend.
B = percentage of virgin bitumen in the final blend. In this relationship the 'blend' is the total quantity of bitumen only, i.e. A+B = 100.

H.11.1 Limitations of bitumen blending

Bitumen in RAP recovered from cracked asphalt will typically have a penetration of less than 15 and satisfactory blending of the new and old bitumen cannot be expected. For example, to obtain a final penetration of 80 in a blend of 60 per cent of fresh bitumen and 40 per cent RAP bitumen in which the bitumen had hardened to a penetration of 15 would require the use of fresh bitumen with a penetration of approximately 200. It is highly likely that some fresh aggregate would only be coated with the soft fresh bitumen and this could play a dominant role in mix performance with a risk of failure through plastic deformation.

The most reliable method of obtaining a robust design with brittle asphalt is, therefore, to regard the bitumen in hardened RAP as being part of the aggregate structure and to use a 60/70 or 80/100 penetration grade bitumen, rather than a soft binder. This will prevent the possibility of plastic deformation in the new mix.

In the case of RAP from areas of plastic deformation, the effect of the softer existing bitumen can be taken into account during the mix design process. Testing of laboratory and plant mix asphalt is required to ensure that requirements for volumetric design and Marshall properties are met. The Marshall procedures outlined in Appendix C of the ERA *Pavement Design Manual Volume 1* should be followed. Additional information from a performance test such as the wheel tracking test is also be very helpful in this evaluation.

The percentage of RAP that can be used is controlled by the mixing temperature that can be achieved in the blended material. The temperature must be high enough to ensure that the fresh bitumen is at a suitable viscosity for mixing.

H.12 Mix design

The most common design procedure is that proposed by the Asphalt Institute (1986). In order to meet mix consistency and design tolerances it is recommended that RAP be used to produce binder course or roadbase mixes for which suitable specifications have been given in the *Pavement Design Manual Volume*. These recycled materials must be sealed or surfaced with a new bituminous wearing course.

Initial assessments of the suitability of materials for recycling may necessarily be based on the results of tests carried out on completely 'broken down' cores. In practice RAP is obtained with heavy equipment producing blocks of material for crushing or by use of a milling machine. Therefore the actual grading of the RAP must be taken into account when completing the final mix designs. The need for further fine adjustment may be indicated after the handling and compaction characteristics of the new mix have been assessed in

pre-construction trials because further breakdown of RAP is likely to have occurred during plant mixing.

The recycled mix must meet the normal requirements for volumetric composition, i.e. be designed to 4 or 5 per cent VIM and retain at least 3 per cent VIM after secondary compaction by traffic as appropriate (see Appendix C of the *Pavement Design Manual Volume 1*). It is not possible to test roadbase or binder course mixes by the Marshall method if they contain aggregate particles larger than 25mm. If the proportion of material greater than 25mm is small then the guidance given in Appendix C can be adopted and the resultant mix evaluated in field trials.

Where it is not possible to use the Marshall test because of aggregate size, the Percentage Refusal Test (BSI, 1989) can be used to ensure that a suitable balance between composition and minimum VIM, after compaction, is obtained (see the *Pavement Design Manual Volume 1*). The Percentage Refusal Density Test should be used to check the density of the laid material.

Aggregate used in the RAP may be known to give good Marshall test results when used in a new AC material. If the fresh aggregate also comes from the same or a similar rock source, and meets the normal requirements for aggregate soundness, strength and durability, then compositional tests may be sufficient for the design of asphalt which will perform well under a new asphalt wearing course. However, wherever possible, performance tests such as Indirect Tensile or wheel tracking tests should be used to ensure that a satisfactory mix can be produced.

H.13 Asphalt recycling case studies

Two feasibility studies for recycling bituminous surfacing materials are described below. Only core samples could be obtained for testing. Coring locations were established on 1km long sections which were representative of the remainder of the road. Both sections were visually reasonably uniform. A longitudinal and transverse sampling pattern was adopted as shown in Table H.3. Site details are given in Table H.4.

Structural evaluations should be carried out as part of the feasibility study to ensure that an appropriate method of pavement rehabilitation is selected. However, the absence of deformation on Site 1 indicated that the pavement was strong and investigations at Site 2 showed that the cement stabilised roadbase had not deformed and that failure was confined to the asphalt layers.

Table H.3 Locations of core sampling

Chainage	Between verge side wheel path and road edge	Verge side wheel path	Centre line of lane	Off side wheel path
0	✓	✓	✓	✓
100		✓		
200	✓	✓	✓	✓
300		✓		
400	✓	✓	✓	✓
500		✓		
600	✓	✓	✓	✓
700		✓		
800	✓	✓	✓	✓
900		✓		
1000	✓	✓	✓	✓

Table H.4 Details of road sites

Site	Traffic category	Range of rut depths (mm)	Cracking	Comments
1	Very heavy	0-4	Severe	Failure by cracking of the asphalt surfacing
2	Very heavy	40-70	None	Climbing lane. Failure by plastic deformation of the asphalt surfacing

H.13.1 Testing of the core samples

The cores of RAP were warmed and broken down. The aggregate particle size distributions of the RAP were determined after removal of the bitumen binder from representative samples. Other samples were left in fine 'lump' condition, typical of a fine-milled material, for inclusion in recycled mixes. In practice the effective 'lump size' of RAP depends upon the method of recovery and the degree of breakdown which occurs during mixing.

H.13.2 Case study 1

Analyses of core samples are shown in Table H.5. The mean bitumen content and aggregate grading for the wearing course and binder course were significantly different, as would be expected, and the variability of bitumen content and penetration within each layer was low.

In practice the wearing course and binder course could be stockpiled separately or as a combined material. Milling would enable separate stockpiling but if simple breaking-out equipment were to be used then the two materials would probably be recovered in large lumps which would have to be crushed and mixed to give a single RAP material.

In this case study the penetration of the bitumen in both layers was less than 15 and the RAP was brittle. It is likely that full-scale recovery of RAP would result in lumps

containing material from both layers and the core samples were, therefore, mixed together for testing in the laboratory.

Table H.5 Summary of layer composition: Case Study 1

BS sieve (mm)	Per cent passing sieve size			
	Wearing course		Binder course	
	Mean	Range	Mean	Range
28	100	-	97	87-100
20	100	99-100	87	80-96
14	95	92-98	75	66-85
10	87	82-91	63	52-76
6.3	78	70-84	50	40-61
5	68	62-77	43	33-53
3.35	55	50-65	38	29-47
2.36	46	42-53	33	26-46
1.18	34	32-37	26	21-32
0.6	27	26-29	22	18-27
0.3	22	20-23	19	16-23
0.212	19	18-21	17	14-21
0.15	18	17-19	15	12-19
0.075	15	14-16	13	10-16
Bitumen (%)	5.4	4.9-5.7	3.6	3.1-4.4
Penetration (0.1mm)	13	6-24	9	5-15

Use as granular material

Crushing and stockpiling RAP from this site would be relatively easy if the two layers of brittle asphalt were to be combined. An ‘all-in’ particle size distribution would easily meet the requirements given in Table H.2 for sub-base.

Thorough pre-crushing of the RAP would make it easier to place and compact the material and vibratory rollers should also be effective in breaking down any agglomerations. Fresh aggregate could, if necessary, be blended with the RAP to modify the particle size distribution. The selection of an effective blend of materials must be determined after sufficient field compaction trials have been carried out to ensure that the normal requirements for the density and strength of a sub-base have been achieved.

Hot mix recycling

During hot mix recycling, agglomerated asphalt remaining in the pre-crushed RAP tends to breakdown and the effective particle size distribution of the RAP will be similar to that used in the laboratory trials described below.

Reference to historical data for fresh aggregate stockpiles used on a local road contract showed that up to 47 per cent of RAP could be blended with these materials to produce a grading meeting a roadbase specification.

As indicated in Table H.5, the penetration of recovered bitumen ranges from 6 to 24 in the wearing course and from 5 to 15 in the binder course. It is very unlikely that a rejuvenator

would suitably modify the bitumen and it would be better to treat the existing bitumen as part of the aggregate particles and to add new 60/70 penetration bitumen.

A blend of 40 per cent RAP and fresh aggregate was designed to conform with a Superpave mix having a nominal maximum aggregate size of 25mm as shown in Table H.6. It can be seen that the particle size distribution passes below the restricted zone. The particle size distribution also conforms to the requirements of the Asphalt Institute for a nominal 25mm mix and the resultant mix should, therefore, have the potential to be very stable.

Table H.6 An example of blending fresh aggregate and RAP: Case Study 1

Sieve (mm)	Superpave particle size distribution limits			Blend 40% RAP and fresh aggregate
	Control points		Restricted zone	
	Min	Max		
37.5	100	-	-	100
25	90	100	-	97
19		90	-	87
4.75			39.5 – 39.5	33
2.36	19	45	26.8 – 30.8	23
1.18			18.1 – 24.1	17
0.6			13.6 – 17.6	13
0.3			11.4 – 11.4	10
0.075	1	7	-	7

Mix design was based on the bitumen content which gave 3 per cent VIM at refusal density using a vibrating hammer. The design bitumen content was found to be 2.8 per cent of fresh 60/70 penetration grade bitumen. Samples having a diameter of 150mm were made in a gyratory compactor to give approximately 7 per cent VIM, or 96 per cent of refusal density, for Indirect Tensile Tests (ITT) and wheel tracking tests.

The results of the performance tests summarised in Table H.7 show that the mix should be very stable under traffic. Wheel tracking tests, in particular, show that the UK specification for 'very heavily stressed sites' (see Appendix F of the ERA *Pavement Design Manual Volume 1*) are easily met.

Table H.7 Laboratory performance test results for a recycled mix: Case Study 1

No. of samples	VIM (%)	% of refusal density	Wheel tracking rate (mm/hr at 60^0C)	ITT (GPa)	
				At 20^0C	At 30^0C
6 per test	6.5-7.3	95.3-96.0	0.17-0.42	5.7-8.0	2.5-3.6

H.13.3 Case Study 2

Plastic deformation of up to 70mm had developed at this site. The appearance of the asphalt exposed at the sides of the cores was very uniform throughout the depth of the material and no individual layers could be identified. Asphalt thicknesses are summarised in Table H.8.

Because of the large deformation it is convenient to refer to the position of material in relation to the top of the stabilised roadbase. The cores were sawn into 50mm slices and bitumen content, bitumen penetration and particle size distributions were carried out on the slices. Material in layer 3 represented the top 50mm of the road surfacing. A summary of the results are given in Tables H.9 and H.10.

The results show that layer 1 (immediately above the roadbase) had a slightly coarser particle size distribution and a mean bitumen content that was 0.4 and 0.2 per cent lower than for layers 2 and 3 respectively. Large variations were found in the penetration of bitumen recovered from the three layers. Theoretical penetrations of fully blended RAP bitumen and fresh 65 and 100 penetration bitumens calculated using equation H.1 are summarised in Table H.11.

Table H.8 Thickness of cores: Case Study 2

Chainage	Core Nos.	Core thicknesses			
		Near road edge	Verge side wheelpath	Centre of lane	Off-side
0	1		145		
100	2-5	190	107	175	115
200	6		106		
300	7-10	135	143	145	112
400	11		93		
500	12-15	157	113	175	78
600	16		132		
700	17-20	148	155	120	150
800	21		95		
900	22-25	145	105	140	85
1000	26		107		

Table H.9 VIM and bitumen content: Case Study 2

Layer	Height above roadbase (mm)	No. of cores analysed	No. with VIM < 3%	Bitumen content (%)		
				Mean	SD	Range
1	0-50	16	11	4.1	0.3	3.2-4.6
2	51-100	16	16	4.5	0.4	3.6-5.0
3	101-150	16	10	4.3	0.4	3.7-5.0

Bitumen in layer 1 had a much lower penetration than that in layers 2 and 3 indicating that consistent stockpiles of RAP would be obtained if this layer was treated separately. If 30 per cent of RAP from layer 1 was recycled with fresh aggregate and 100 penetration grade bitumen then substitution of these values in equation H.1 indicates that the resultant penetration would be between 53 and 83 with a median value of 66 if full blending of the bitumens occurred.

A satisfactory mix could probably be made using 30 per cent of well mixed RAP from layers 2 and 3 blended with fresh aggregate and 65 penetration grade bitumen. Although it is very unlikely that uniform blending of the bitumens would be achieved the theoretical penetration of the blended bitumens, before ageing in the mixing process, would range from approximately 49 to 81 with a mean value of 66.

Table H.10 Aggregate particle size distribution: Case Study 2

BS sieve (mm)	Per cent passing sieve size					
	Layer 1		Layer 2		Layer 3	
	Mean	Range	Mean	Range	Mean	Range
28	97	91-100	99	96-100	100	-
20	84	70-92	98	86-97	98	94-100
14	70	56-87	80	73-87	89	82-96
10	60	44-74	70	64-75	75	67-85
6.3	51	35-63	60	53-65	60	53-73
5	45	31-58	53	46-60	54	47-65
3.35	40	27-51	47	42-54	48	40-54
2.36	34	23-44	42	37-46	42	35-47
1.18	26	18-34	33	26-46	34	27-38
0.6	22	15-28	28	21-35	29	24-33
0.3	17	12-23	23	16-29	24	18-28
0.212	14	10-19	20	13-25	21	15-25
0.15	11	7-15	16	9-21	18	12-22
0.075	7	5-11	11	5-15	13	7-17

Table H.11 Penetration of RAP bitumen blended with fresh bitumen: Case Study 2

Layer	Penetration of RAP bitumen		Ratio of RAP to fresh aggregate	Penetration of fresh bitumen	Penetration of bitumen after mixing	
	Median	Range			Median	Range
3 Top 50mm	87	30-135	30:70	65	67	52-81
			50:50		69	52-93
2 Middle 50 mm	68	25-160	30:70		64	49-81
			50:50		65	40-94
1 Bottom 50mm	27	12-58	30:70		49	39-61
			50:50		41	28-59
3 Top 50mm	87	30-135	30:70	100	90	70-109
			50:50		86	55-115
2 Middle 50 mm	68	25-160	30:70		87	66-109
			50:50		81	50-116
1 Bottom 50mm	27	12-58	30:70		66	53-83
			50:50		50	35-73

Reclaiming the existing asphalt

It is necessary to use a milling machine to produce as fine a material grading as possible because the bitumen in the RAP is soft and simple crushing during hot weather would probably not be possible. Hot mixing would take advantage of the 'soft' bitumen and the addition of fresh 60/70 or 80/100 penetration grade bitumen should ensure that mixing with new aggregate would produce a mix with acceptable uniformity.

A laboratory design for a roadbase mix was carried out using fresh aggregates. A blend of 50 per cent RAP and fresh aggregate was designed to conform with the Asphalt Institute requirements for a mix having a nominal maximum aggregate size of 25mm as shown in Table H.12.

The design bitumen content was found to be 3.5 per cent of which 1.3 per cent was fresh 60/70 penetration grade bitumen. Samples having a diameter of 150mm were made in a gyratory compactor to give approximately 7 per cent VIM, or 96 per cent refusal density, for Indirect Tensile Tests (ITT) and wheel tracking tests.

The results of the performance tests on the recycled mix are shown in Table H.13. Wheel tracking rates were low and well within the UK specification for very heavily stressed sites whilst the ITT results were marginal for a mix containing 60/70 penetration grade bitumen. It is considered that the wheel tracking test gives the better indication of stability and it was concluded that the mix should be stable under traffic.

Table H.12 An example of blending fresh aggregate and RAP: Case Study 2

Sieve size (mm)	Passing sieve size (%)	
	Asphalt Institute grading for 25mm nominal maximum size aggregate	Blend 50:50 RAP and fresh aggregate
37.5	100	100
25	90-100	93
12.5	56-80	66
4.75	29-59	34
2.36	19-45	22
0.3	5-17	12
0.075	1-7	6

Table H.13 Laboratory performance test results for a recycled mix: Case Study 2

No. of samples	VIM (%)	% of refusal density	Wheel tracking rate (mm/hr at 60^0C)	ITT (GPa)	
				At 20^0C	At 30^0C
6 per test	7.0	96.0	0.12-0.46	-	1 - 2

H.14 Recycling of Portland Cement Concrete

In most cases where reconstruction is justified, the existing concrete can be recycled. Concrete pavement recycling involves breaking up the old pavement on grade, loading and hauling the material to a crushing plant, and processing it at the plant to produce recycled concrete aggregate (RCA). The product of this process is an aggregate that can be used in place of virgin aggregate in any component of the pavement structure where aggregate is used, including:

1. Untreated, dense-graded aggregate roadbase
2. Cement- and asphalt-stabilized roadbases
3. Lean concrete roadbase
4. Portland cement concrete surfacing
5. Asphalt concrete surfacing
6. Fill
7. Filter material
8. Drainage layer or edge drains.

Recycling of PCC pavements is not limited to those pavements that contain sound aggregate. Pavements containing reactive aggregate can be recycled into new concrete using fly ash to control expansion of the reactive aggregate. Badly D-cracked pavements have also been successfully recycled into new pavements. The common practice in recycling D-cracked pavement is to limit the maximum size of the recycled concrete aggregate, so that any aggregate particles that contain sizable voids are fractured before being used in the new concrete mix.

One limitation of small aggregate size is poor aggregate interlock at joints and cracks, especially on JRCP and CRCP. Additional large virgin aggregate may need to be added so that sufficient aggregate interlock will develop.

A PCC pavement that has an AC overlay can be recycled, but the two layers must be recycled separately. Asphalt concrete should not be recycled for aggregate in a PCC mix, because the asphalt cement will inhibit entrainment of air in the concrete mix.

Whether or not a particular pavement should be recycled depends on numerous factors, including availability and cost of virgin aggregate, cost of disposing of old pavement material if it is not recycled, approximate cost of recycling, policy toward recycling, and the extent of local contractors' experience with recycling. However, PCC pavement recycling has proven to be both economical and environmentally advantageous. Laboratory and field studies have shown that a high quality concrete, with improved durability, can be produced using recycled concrete aggregate. Significant savings in material transportation and disposal costs are possible through recycling, particularly in urban areas.

PCC pavement recycling can be divided into two groups:

1. surface recycling
2. central plant recycling

Both surface and central plant recycling techniques have been utilized on roadways containing Portland cement concrete. However, surface recycling techniques applied to

concrete pavements is most often considered a pavement removal operation. The removed pavement can be recycled.

H.14.1 Surface Recycling

Surface recycling techniques involve the use of cold milling or cold planing techniques that are capable of economically removing up to approximately two inches of concrete in a single pass. Traffic can operate for extended periods of time on the milled surface, or an asphalt concrete overlay can be placed.

The sequence of operation involving surface recycling is as follows:

1. Establishing desirable grade line.
2. Milling, grinding, or planing the pavement to the desired depth.
3. Clean-up involving rotary broom and vacuum equipment.
4. Disposal or recycling of the millings.

Surface recycling operations involving overlays use the sequence of operations as described above, with the addition of a tack coat and asphalt concrete overlay.

Pavement milling operations are suitable for the removal of localized severe surface undulations caused by swelling clays, etc. Milling is also suitable for removal of pavement prior to overlay along gutters, at bridge approaches, and other areas where a feathered edge of asphalt concrete or Portland cement concrete is likely to abrade; and for improved drainage, surface texture, and skid resistance.

An added advantage of surface recycling is the increase in bond strength between a milled Portland cement concrete and an overlay as compared to a normal overlay operation.

H.14.2 Central Plant Recycling

The pavement removal and crushing operations are performed with conventional construction and demolition equipment or specially designed equipment. The old concrete pavement is normally broken such that the size of the resulting slab is small enough to be received by the primary crusher. Additional reduction in slab size can be performed at the crushing location. Central plant sizing can be performed with conventional, fixed and portable crushing equipment; however, reinforcing steel may be a problem and may have to be removed at one or more of six processing locations:

1. On the grade during the loading operation,
2. During the locating operation for crushing if stockpiling occurs prior to crushing,
3. At the entry to the primary crushing,
4. On the belt after primary crushing,
5. On the belt after final crushing, or
6. In the stockpile prior to remixing.

Equipment recently developed pulverizes the concrete to smaller sizes on grade and thus more complete steel removal is possible on grade.

Blending and mixing operations in the central plant are standard operations, as are the techniques utilized for the placing and curing of recycled materials. Gradation adjustments (particularly the addition of natural sands) are often made to improve workability.

H.14.3 Structural Design

The structural design of PCC pavement using recycled materials can be done, in principle, using the guidelines given in *the ERA Pavement Design Manual, Volume II*.

In essence, recycled material is viewed no differently from new or virgin material in either new or rehabilitated pavement systems. Therefore the most important structural rehabilitation parameter for recycled material is the characterization of the load carrying ability (lab and/or field) of the material.

Literature review indicates that the structural coefficients of recycled materials do reflect the extreme variation of reused materials and recycling processes utilized. However, it is equally apparent that recycled materials have the capacity to be equivalent, or greater, in load spreading capabilities than the original material. With good control on the recycling process, and based on laboratory test data, an appropriate structural coefficient can be evaluated to be used in the design.

Similarly, the properties of concrete made with recycled aggregate show some variations compared to the concrete made with virgin aggregate, as described below.

H.14.4 Concrete Properties

The following is a comparison of the properties of PCC made with recycled concrete aggregate and concrete made with virgin aggregate:

1. The compressive strength of recycled concrete is between 60 and 100 percent of the compressive strength of conventional concrete at the same water-cement ratio. Water-reducing admixtures can be used to obtain higher strength concrete at the same cement content without compromising workability.
2. The static modulus of elasticity of recycled concrete is between 60 and 100 percent of the modulus of conventional concrete at the same water-cement ratio.
3. The flexural strength of recycled concrete is between 80 and 100 percent of the flexural strength of conventional concrete at the same water-cement ratio.
4. Recycled concrete has a higher ratio of flexural strength to compressive strength than conventional concrete.
5. Low-strength recycled concrete can be recycled into higher-strength concrete through proper mix design.
6. The durability of concrete made with aggregate susceptible to D-cracking can be substantially increased by limiting the top size of the aggregate.
7. The volume response to moisture and temperature changes of recycled concrete is not significantly different from that of normal concrete.

The fact that concrete containing recycled concrete aggregate has a higher ratio of flexural strength to compressive strength than conventional concrete should be considered if compressive strength is to be used as the criterion for acceptance or for timing the early opening of reconstruction projects.

All of the above comments about the performance of recycled concrete aggregates in concrete surfaces apply to its use in lean concrete roadbase courses. Recycled concrete aggregate was used in lean concrete roadbases long before it was used in concrete surfaces because not enough was known about its performance in surface courses.

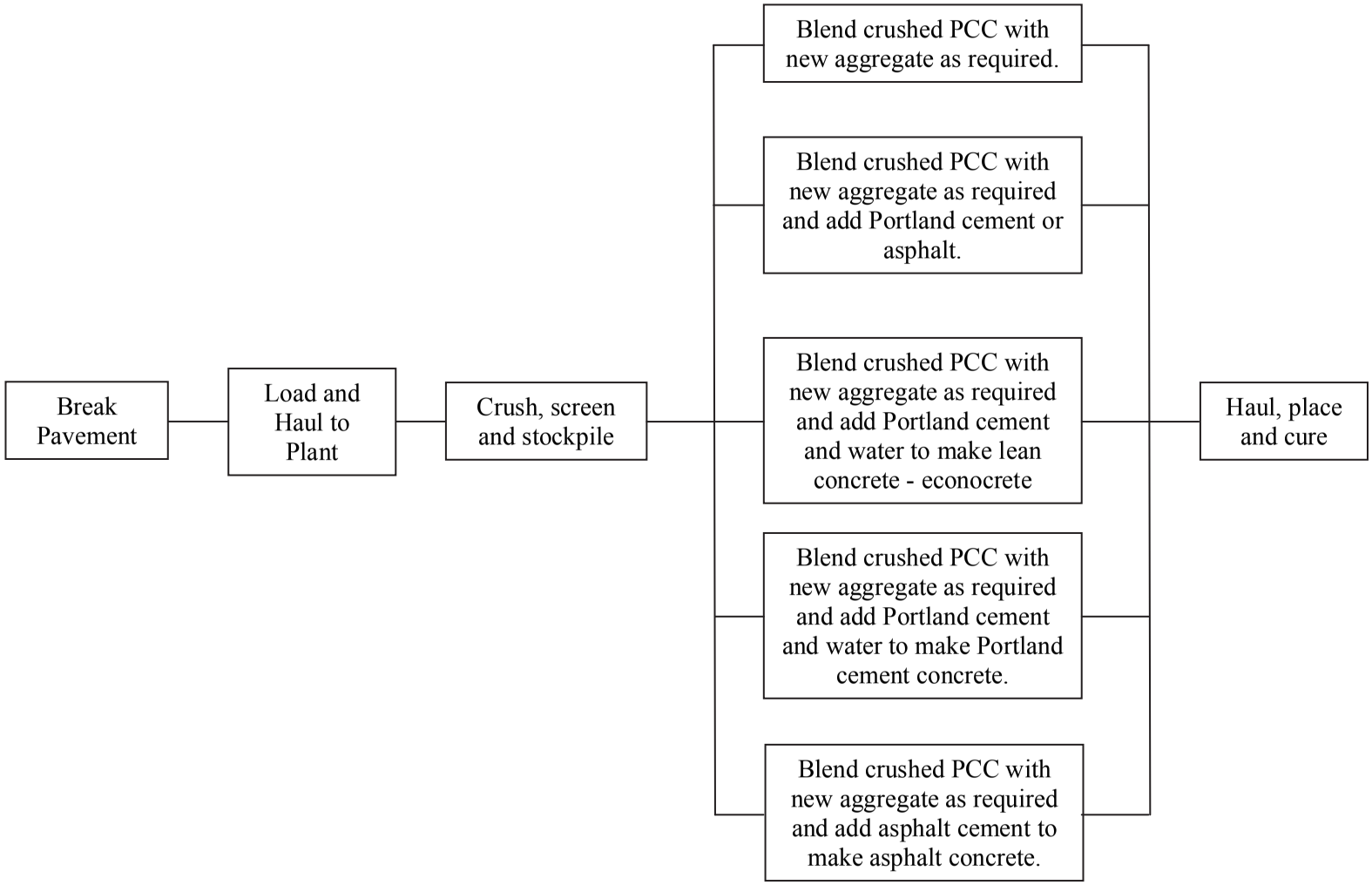


Figure H.2 Portland Cement Concrete Pavement Recycling